GEOTECHNICAL ENGINEERING INVESTIGATION REPORT
C-4016 NEW ALLIED SCIENCE BUILDING
CONTRA COSTA COLLEGE
2600 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA
KLEINFELDER PROJECT No.: 20181569.001A

October 17, 2017

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Contra Costa Community College District (District)
2600 Mission Bell Drive
San Pablo, California 94806
c/o Mr. Ron Johnson
ronj@csipm.com

SUBJECT: Geotechnical Engineering Investigation Report
C-4016 New Allied Science Building
Contra Costa College
2600 Mission Bell Drive
San Pablo, California

Dear Mr. Johnson:

Kleinfelder is pleased to present this geotechnical engineering investigation report for the planned new Allied Science building at Contra Costa College in San Pablo, California. The project site is currently occupied by the Liberal Arts and Health Sciences buildings, which are abandoned and earmarked for demolition.

The purpose of our geotechnical engineering investigation was to explore and characterize the subsurface conditions and provide mitigation measures for the identified geologic seismic hazards in addition to recommendations for grading, foundations, drainage, and construction considerations. It is our opinion that the project is feasible from a geotechnical engineering standpoint provided our recommendations are incorporated into the final plans and specifications of the project. The proposed new science building may be supported on a shallow foundation system. Based on the results of our field investigation and the current conceptual design, varying materials, from weathered claystone, to undocumented clay fill which are unsuitable materials, to sandy lean clay, clayey sand, and clayey sand with gravel, are expected at the foundation and lower floor slab bearing levels; therefore, over-excavation is recommended in order to provide a more uniform support for the proposed foundation and lower floor slab. Our geotechnical recommendations are provided in this report.

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.

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

Sincerely,

KLEINFELDER, INC.

REGISTERED PROFESSIONAL ENGINEER
EDWARD C. MAK
NO. GE 2212
Exp. 3/31/18
GEOTECHNICAL
STATE OF CALIFORNIA

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Geotechnical Engineer

Reviewed by

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Principal Geotechnical Engineer

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1 INTRODUCTION

This report presents the results of our geotechnical engineering investigation performed for the planned new Allied Science building at Contra Costa College in San Pablo, California. The approximate location of the school campus is shown on the Site Vicinity Map (Figure 1), and the approximate limit of the planned new science building is shown on the Site Plan (Figure 2).

We understand that the campus plans to demolish the existing abandoned Liberal Arts and Health Sciences buildings and construct a new 3-story building with an approximate footprint of up to about 20,000 square feet. No information on the design and construction of the existing Liberal Arts and Health Sciences building was provided to us. The foundation type and size of the existing building are unknown. There is a retaining wall adjacent to the existing service road on the northeast side, and outside, of the existing building. The foundation type and size of the retaining wall are also unknown to us at this time. Based on conceptual drawings that were provided to us, the first, second, and third floors of the new building will be near the Lower Plaza elevation of 72 feet, the Upper Plaza elevation of 92 feet, and the Upper Campus elevation of 114 feet, respectively. For a lower floor elevation of 72 feet it is anticipated that cuts up to about 9 feet may be required for the construction of the new building. These could change since the project is currently in conceptual design phase. Structural loads are assumed to be less than 500 kips for column loads. The final layout of the new building and proposed grading have not been determined at this time.

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

1.1 GENERAL SITE DESCRIPTIONS

The western part of the campus is located mostly on a level alluvial plain west of Rheem Creek. The eastern portion of the campus slopes upward to the northeast. The active Hayward fault, which crosses the campus, approximately separates the flat lying portion of the campus with the elevated/hillside portion of the campus. Rheem Creek flows through the campus in a northwesterly direction generally parallel to the base of the hillside. Most of the academic buildings on the campus are located on the hillside portion of the campus, while the flat lying portion of the campus contains mostly the athletic buildings and facilities. The ground surface elevation at the campus ranges from about 50 feet above mean sea level along the southwestern margin of the campus to about 130 feet in the northeast corner along Campus Drive.

According to the U.S. Geological Survey (USGS, 1993) 7½-Minute Richmond Topographic Quadrangle map, the existing ground elevation at the subject site ranges between about 70 and 100 feet above mean sea level. The coordinates at the center of the planned new science center location are approximately:

Latitude: 37.9697° N
Longitude: 122.3369° W

1.2 PREVIOUS INVESTIGATIONS

Kleinfelder previously performed several fault trench and geotechnical investigations at the campus. The results of these previous investigations were presented in the following reports:

- Kleinfelder's report titled *Subsurface Fault Investigation, Proposed Addition to the Student Activities Building, Contra Costa College, San Pablo, California*, dated December 2, 2003 (File No. 33133/SSA);
- Kleinfelder's report titled *Geotechnical Investigation Report, Student Activities Building Addition, Contra Costa College, San Pablo, California*, dated April 16, 2004 (File No. 40698/GEO);
- Kleinfelder's report titled *Subsurface Fault Investigation at the Existing Student Activities Building, Contra Costa College, San Pablo, California*, dated August 7, 2007 (File No. 82074/Report);
- Kleinfelder's report titled *Subsurface Fault Investigation in the Vicinity of the Existing Humanities Building, Contra Costa College, San Pablo, California*, dated February 20, 2008 (File No. 86352/Report);

- Kleinfelder's report titled *Master Plan Seismic Study, Contra Costa College Campus, San Pablo, California*, dated July 15, 2009 (Project No. 80412/Report);
- Kleinfelder report titled *Geotechnical Investigation Report, Campus Center, Contra Costa College, San Pablo, California*, dated February 17, 2011;
- Kleinfelder report titled *Re-Assessment of Fault-Related Exclusionary Boundaries Pertaining to Habitable Structures for the Campus Center Project/New Student Activities Building Proposed within the Contra Costa College Campus, San Pablo, California*, dated March 24, 2011 (Project No. 112252/PWPortables/PLE11L027); and
- Kleinfelder report titled *Amendment to Master Plan Seismic Study, Contra Costa College Campus, San Pablo, California*, dated April 16, 2012 (Project No. 124348/SRO12R0273).
- Kleinfelder report titled *Subsurface Fault Investigation, Lower Parking Area, Contra Costa Community College, San Pablo, California,* dated November 16, 2016.
- Kleinfelder report titled *Subsurface Fault Investigation, Proposed C-4001 Campus Safety Center, Contra Costa Community College, San Pablo, California,* dated June 29, 2016.
- Kleinfelder report titled *Geotechnical Investigation Report, Campus Safety Center, Contra Costa Community College, 2600 Mission Bell Drive, San Pablo, California*, dated March 17, 2017 (20164720.001A).
- Kleinfelder report titled *Geologic and Seismic Hazards Assessment Report, Planned Campus Safety Center, Contra Costa College, 2600 Mission Bell Drive, San Pablo, California*, dated March 30, 2017 (20164720.001A).
- Kleinfelder report titled *Geologic and Seismic Hazards Assessment and Geotechnical Investigation Report, C-608 PE/Kinesiology Renovation Project, Contra Costa Community College, 2600 Mission Bell Drive, San Pablo, California, dated August 28, 2017 (Project No. 20181293.001A)*

1.3 PURPOSE AND SCOPE OF SERVICES

The purpose of our geotechnical investigation was to explore and evaluate the subsurface conditions at the site in order to develop recommendations related to the geotechnical aspects of project design and construction. The proposed scope of our services was outlined in our proposal

(MF180192.001P/PLE17P62057) dated June 30, 2017, revised July 18, 2017. Our services as presented in this report include the following:

- A site reconnaissance to observe the surface conditions
- A field investigation that consisted of drilling four borings to explore the subsurface conditions
- Laboratory testing of selected soil samples obtained during the field investigation to evaluate relevant physical and engineering parameters of the subsurface soils
- Evaluation of the field and laboratory data obtained and performing engineering analyses to develop our geotechnical conclusions and recommendations
- Preparation of this report which includes:
    - Site Vicinity Map, and Site Plan showing the approximate test boring locations;
    - Description of the project;
    - Discussion of general site subsurface conditions, as encountered in our test borings;
    - Discussion of liquefaction analysis and settlement potential and magnitude;
    - Conclusions pertaining to feasibility of the proposed development, impacts of geotechnical and geologic features on the proposed development;
    - Recommendations for site preparation, subgrade preparation, earthwork, and fill compaction specifications;
    - Recommendations for design of footings including allowable soil pressures and embedment depths;
    - Anticipated total and differential settlements;
    - Recommendations for retaining walls including active and at-rest earth pressures, seismic surcharges, static surcharges, and passive resistance;
    - Slab-on-grade and flatwork support recommendations;
    - Recommendations for surface and subsurface drainage;
    - Soil corrosivity test results;
    - Construction considerations, and
    - An appendix including boring logs and laboratory test results;

Our current scope excluded an assessment of pipeline locations within 1,500 feet of the project site. Our evaluation also specifically excluded the assessment of environmental spills and hazardous substances at the site.

2 GEOLOGIC AND SEISMIC HAZARDS SUMMARY

A Geologic and Seismic Hazards Assessment was conducted for the subject project, and the results are presented in a separate report titled *Geologic and Seismic Hazards Assessment Report, C-4016 New Allied Science Building, Contra Costa College, 2600 Mission Bell Drive, San Pablo, California*, Project No. 20181569.001A, October 2017. We have also conducted an updated site-specific ground motion analysis for the subject project, and the results are presented in Appendix E of our Geologic and Seismic Hazards Assessment report dated.

More detailed discussion and our opinions regarding geologic and seismic hazards are presented in our Geologic and Seismic Hazards report. Brief summaries of our opinions regarding geologic and seismic hazards that are more related to geotechnical engineering, such as seismic shaking, fault-related ground surface rupture, liquefaction and lateral spreading, dynamic compaction, expansive soils/bedrock, and landslides, are provided below.

2.1 SEISMIC SHAKING

We expect the site to be subjected to substantial ground shaking due to a major seismic event on the surrounding faults, especially the active Hayward fault. Much of the campus, including the project site, is located within an Alquist-Priolo Earthquake Fault Zone, associated with the active Hayward fault.

2.2 FAULT-RELATED GROUND SURFACE RUPTURE

In 2009, Kleinfelder completed a Master Plan Seismic Study for the entire campus. The purpose of that study was to provide a campus-wide guidance document and map showing areas where the presence of active faulting has been cleared for future development at the campus (and no additional fault studies would be needed), as well as those areas that have been documented to be underlain by active faulting (building exclusion zones) and those areas that would require further studies to determine building potential. That study was reviewed and the conclusions were accepted by California Geological Survey (CGS). The current proposed project is located within the limits of the cleared or "Habitable Zone".

Much of the campus, including the subject 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 our project Certified Engineering Geologist (CEG). Therefore, it is our opinion that the potential for continued surface creep along the main fault trace located to the west/southwest of the project site is high. 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 campus cannot be ruled out. However, based on historic performance, the knowledge that the main trace is more than 50 feet away from the planned project site, and the setback from the nearest mapped secondary fault trace is about 50 feet, which is adequate, we conclude that the potential for fault-related ground surface rupture to impact the planned project is considered low because of the adequate setback distance noted.

2.3 LIQUEFACTION AND LATERAL SPREADING

Based on the subsurface data obtained from our field investigation, the project site subsurface consists mostly of interbedded layers of firm to hard fine-grained clayey soils underlain by bedrock. As a result, liquefaction potential at the site is considered minimal due to the soil types encountered. Also, we conclude that the potential for lateral spreading to occur at the site as a result of a future seismic event is low.

2.4 DYNAMIC (SEISMIC) COMPACTION

Based on the subsurface conditions observed during our investigation, we conclude that densification is not likely to occur at the site and would not result in significant settlement if it did occur.

2.5 EXPANSIVE SOILS/BEDROCK

Our laboratory test data indicate that the site soils and bedrock have low to high expansion potential. Recommended options for mitigation of expansive soil/rock behavior include deepening the footings (if a shallow foundation system is selected), blanketing the slab areas with “non-expansive” soil, and using special earthwork procedures, such as moisture-conditioning.

2.6 LANDSLIDES

No landslides are mapped in the project area and slope creep or cracks were not observed. Therefore, it is our opinion that the potential for seismically induced (or otherwise) landslides and slope failure to occur at the proposed site is considered low.

3 FIELD INVESTIGATION AND LABORATORY TESTING

3.1 FIELD INVESTIGATION

3.1.1 Pre-Field Activities

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

3.1.2 Exploratory Borings

We drilled four test borings at the planned new science building site on August 11, 2017 and August 18, 2017 to depths between approximately 31 and 41½ feet. The approximate locations of the borings are shown on Figure 2. The borings were drilled by Gregg Drilling & Testing, Inc., of Martinez, California, using a truck-mounted drill rig equipped with 6-inch outside-diameter hollow-stem augers. The boring locations were located in the field by measuring from existing landmarks. Horizontal coordinates and elevations of the borings were not surveyed.

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

Keys to the soil descriptions and symbols used on the boring logs are presented on Figures A-1 and A-2 in Appendix A. Rock description key is presented on Figure A-3. Logs of the borings are presented on Figures A-4 through A-7.

3.1.3 Sampling Procedures

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

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

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

3.2 LABORATORY TESTING

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

Limited corrosion analyses as listed below were performed on a composite sample by CERCO Analytical of Concord, California.

- Corrosion - Soluble Sulfate Content (ASTM D 4327)
- Corrosion - Soluble Chloride Content (ASTM D 4327)

- pH (ASTM D 4972)
- Minimum Resistivity (ASTM G57)

The soluble sulfate, soluble chloride, pH, and minimum resistivity test results are discussed in Section 6.9 of this report and the results are presented in Appendix C. Please note that our scope of services does not include corrosion engineering and, therefore, a detailed analysis of the corrosion test results is not included in this report. A qualified corrosion engineer should be retained to review the laboratory test results and design protective systems that may be required. Kleinfelder may be able to provide those services, if requested.

4 SURFACE AND SUBSURFACE CONDITIONS

4.1 SURFACE CONDITIONS

The existing buildings of the subject site are currently situated northeast of Rheem Creek along the elevated portion of the campus. As shown on Figure 2, the buildings are situated in between the Physical Sciences building (located to the northeast), Administrative and Applied Arts building (located to the southeast), and Library and Learning Resource Center (located to the west). In between the Library and Learning Resource Center and Liberal Arts and Health Sciences buildings is an open, grass covered courtyard area gently sloping to the southwest. A fire access road runs parallel with the Liberal Arts and Health Sciences buildings along the northeastern end of the buildings, situated at a higher topographic level than the grass covered open area. The project site generally slopes to the southwest. Sloped walkways and stairways are located around the buildings.

4.2 SUBSURFACE CONDITIONS

The subsurface conditions described herein are based on the soil/bedrock and groundwater conditions encountered during the current and previous geologic and geotechnical investigations in the vicinity of the site area. The project site subsurface consists mostly of fill and native soils underlain by claystone. The fill was encountered in Borings B-3 and B-4 measuring between depths of about 8 to 13 feet and generally consisting of very stiff to hard sandy clays. The native soil consisted stiff sandy clays interbedded with clayey sands and gravels, which in turn were underlain by weathered claystone. The claystone was generally weak to strong, moderately to highly weathered, and highly fractured.

Groundwater was not observed or encountered in our current borings. However, groundwater was observed in our previous borings and fault trenches at depths of about 9 to 23 feet below the ground surface. It should be noted that groundwater levels can fluctuate depending on factors such as seasonal rainfall, landscape irrigation, and construction activities on this or adjacent properties, and may rise several feet during a normal rainy season. It is also common to find perched layers of groundwater at the soil/rock interface.

The above is a general description of soil/bedrock and groundwater conditions encountered in the borings from this investigation and our experience at the campus. More detailed descriptions of the subsurface conditions encountered are presented on the Boring Logs on Figures A-4 through A-7 in Appendix A.

Soil/bedrock 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.

5 DISCUSSION AND CONCLUSIONS

5.1 GENERAL

Based on our findings, it is our professional opinion that the proposed project is feasible from a geotechnical engineering standpoint provided the recommendations contained herein are incorporated into the final plans and specifications. The proposed new science building may be supported on a shallow foundation system bearing on firm engineered fill or native soils. Specific conclusions and recommendations regarding the geotechnical aspects of design and construction are presented in the following sections.

The primary geotechnical concern for the project is the presence of the nearby Hayward fault and the high likelihood that the site will be exposed to a significant seismic event within the project's design life. The proposed structure should be designed to accommodate the anticipated seismic shaking. We understand the layout of the new science building has not been finalized. The final layout of the new building should be located within areas of the site previously designated as habitable zones. Also, according to the California Administrative Code (CAC) Section 4-317(e), the new structure cannot be located within 50 feet of the trace of an active fault.

The second primary geotechnical concern is that varying subsurface conditions are expected at the lower floor level, which could potentially create differential settlement and heaving of the footings as well as the lower level floor slabs. Based on data obtained from our borings and the current layout of the new building with a Lower Plaza elevation of 72 feet, either claystone bedrock, highly expansive undocumented fill with unsuitable materials, or loose native clayey sand, is expected at the foundation and floor slab bearing levels (see Cross Sections A-A' and B-B' on Figures 3 and 4). Instead of supporting the new science building on a deep foundation system, which is expensive, we recommend conducting over-excavation during site grading and supporting the new science building on a shallow foundation system. Also, we recommend that a layer of non-expansive import material be provided below the lower level floor slab.

Additional discussions of the conclusions drawn from our investigation, including general recommendations, are presented below. Specific recommendations regarding geotechnical design and construction aspects for the project are presented in Section 6 of this report.

5.2 FOUNDATIONS AND SLAB SUPPORT

As stated above, over-excavation is recommended. After over-excavation, the new building can be supported on shallow footings or mat slabs, while retaining walls can be supported on shallow footings. Because the site surface soils have high expansion potential, the foundations for the new structures will need to extend deeper than usual if the new buildings are supported on shallow footings. Also, footings (if used) should be continuous around the perimeter of the buildings to reduce the potential for moisture content fluctuations within the expansive soils and bedrock underlying the building footprint. This measure should reduce the development of swell and shrinkage cycles of soils underneath the buildings.

Although mat slabs can be used at the site, our experience shows that it is more difficult to adapt future tenant improvements to this type of foundation because such improvements usually require re-routing of underground utility lines and cutting of the floor slab. Therefore, we suggest using shallow footings to support the new buildings instead of mat slabs. Cast-in-drilled-hole (CIDH) piers may be used to resist uplift loads for the new buildings. Therefore, Section 6 of this report includes design recommendations for shallow footings, mat slabs, and CIDH piers.

Total and differential foundation settlements due to static loads are estimated to be less than 1 inch over a horizontal distance of 70 feet. Our estimated static settlements are based on the anticipated building loads and the assumption that the geotechnical recommendations contained in Section 6 of this report will be incorporated into the design and construction of the project. Static foundation settlements should be primarily elastic in nature, with a majority of the estimated settlement occurring upon application of the load during construction.

The building slabs can be supported on grade. However, due to the presence of expansive soils at the site, the 6-inch layer of ¾-inch crushed rock or slab capillary break material should be underlain by 12 inches of “non-expansive” fill material. The slab subgrade soils will also need to be properly moisture-conditioned prior to the placement of the “non-expansive” material. In a similar fashion, exterior concrete flatwork should be underlain by 6 inches of “non-expansive” material along with proper moisture conditioning of the subgrade soil.

5.3 EXISTING FOUNDATIONS

No information on the foundation type and size of the existing Liberal Arts building is available to us at this time. If the existing building is supported on a shallow foundation system, the existing building and all shallow foundations should be removed and the resulting excavations properly backfilled with compacted engineered fill. On the other hand, if the existing building is supported on a deep foundation system such as drilled piers connected by grade beams, the upper portion of the existing deep foundations should be cutoff to provide a minimum vertical clearance of 3 feet below the bottom of new footings, slabs, and underground utility lines to reduce the risk they will adversely impact their performance and/or constructability.

5.4 EXCAVATION CONDITIONS

We anticipate that excavations at the site can be made with standard earthwork equipment, such as excavators, dozers, backhoes, and trenchers. Claystone bedrock material was encountered in our borings. However, the degree of weathering of the bedrock material varies from moderately to highly weathered. For this reason, we expect the degree of excavation difficulty in the bedrock material would be similar to that of hard/dense soils.

5.5 SOIL/BEDROCK TRANSITION LINES

Based on the subsurface conditions encountered at the site, the southwestern portion of the new building will be founded on soil, while the northeastern portion will be founded on bedrock. To help mitigate possible floor slab distress along bedrock/soil transition lines, over-excavation is recommended.

5.6 UNDOCUMENTED FILL

The undocumented fill encountered during our current investigation is likely the result of past grading at the site during construction of the existing campus buildings and related improvements. The fill appears to be relatively free of organic and deleterious matter and to have been mechanically compacted during grading based on its consistency. Because the site was developed in the 1950's and 1960's, we believe the fill has been in place for several decades. If soft/loose areas are encountered within the fill during excavation of foundations and mass grading for the subject project, additional over-excavation may be required. Deleterious matter encountered in the fill, such as organic laden soil, should be either removed and disposed offsite or possibly be used as general fill in landscaping areas of the site if it is not considered environmentally hazardous. The final vertical and lateral extents of additional over-excavation should be determined by the project Geotechnical Engineer during construction based on exposed subsurface conditions.

6 RECOMMENDATIONS

6.1 GENERAL EARTHWORK

We recommend that Kleinfelder be retained to provide observation and testing services during earthwork and foundation construction. This will allow us the opportunity to compare conditions exposed during construction with those inferred from our investigation and, if necessary, to expedite supplemental recommendations if warranted by the exposed subsurface conditions. We also recommend that, prior to construction, Kleinfelder be retained to review foundation plans and specifications to verify conformance with our recommendations. It has been our experience that this review provides an opportunity to detect misinterpretation or misunderstandings prior to the completion of design and start of construction.

No major filling to raise site grade is expected. Based on the current conceptual design, cutting of about 9 feet may be required to create the building pad. As stated in previous sections of this report, over-excavation is recommended.

We recommend that all permanent cut and fill slopes, if any, be designed to be no steeper than 2 (horizontal) to 1 (vertical).

6.1.1 Site Preparation

Prior to the start of construction, all obstructions, debris and deleterious materials, including any existing structures such as foundations, pavements, concrete slabs, underground utility lines, and wells, if any, should be removed from the construction areas. Stumps and primary roots of any trees and brush should be grubbed. Removal of existing underground utilities should include removal of associated granular bedding material.

After site clearing, we recommend that over-excavation be conducted by excavating the soil/bedrock to a level at least 4 feet below the lower floor slab over the entire building footprint, scarifying and recompacting the over-excavation bottom, and backfilling the over-excavation with moisture-conditioned and compacted onsite soils. The over-excavation should extend laterally to about 5 feet beyond the footprint of the new building, where physically possible. With this over-excavation requirement and a recommended footing embedment depth of 2½ feet (see Section

6.3.1 of this report), there should be at least 1½ feet of engineered fill below the bottoms of footings, and at least 2 feet of engineered fill (not including the non-expansive import materials as described in the next paragraph) below the lower floor slab. Final over-excavation depths should be determined by Kleinfelder during construction based on the exposed subsurface conditions. Additional over-excavations may be required. Geotechnical recommendations related to scarifying, fill material specifications, backfilling, and compacting are presented in Section 6.1.5 of this report.

As stated in Section 5.2 of this report, we recommend that at least 12 inches of non-expansive import materials meeting the import fill requirements be provided beneath the lower floor slab. Imported material may also be used to backfill the over-excavation. However, they should be placed in the upper portion of the over-excavation so that the new exterior continuous footings are keyed at least one foot into the onsite recompacted clayey soils. This requirement reduces the risk of excessive moisture accumulating in the granular fill below the new floor slabs. If restricting the thickness of the granular fill layer is not possible, deepening the exterior continuous footings may be required.

Depressions, voids, and holes (including excavations from removal of underground improvements) that extend below the proposed finished grades should be cleaned and backfilled with engineered fill compacted to the requirements given in Section 6.1.5 of this report. All clearing and backfill work should be performed under the observation of the project Geotechnical Engineer.

6.1.2 Subgrade Preparation

The bottom of the over-excavation and all subgrade areas that will receive engineered fill for support of structures should be scarified to a depth of 12 inches, uniformly moisture-conditioned to a moisture content of at least 2 percent above the optimum moisture content, and compacted as engineered fill to at least 90 percent relative compaction (ASTM D 1557). Over-excavation of disturbed soil, scarification and compaction of the exposed subgrade, and replacement with engineered fill may be required to sufficiently densify all disturbed soil. If the over-excavation bottom or subgrade surface consists of undisturbed bedrock, this scarifying and re-compacting processes are not required.

Following rough grading, construction and trenching activities often loosen or otherwise disturb the subgrade soils. On occasion, this disturbance can lead to isolated movement of the subgrade soils following construction and cracking of overlying slabs and pavement. Accordingly, loose/disturbed areas should be repaired and trench backfill should be properly compacted prior to placement of concrete.

6.1.3 Temporary Excavations

Construction site safety is the sole responsibility of the contractor, who shall also be solely responsible for the means, methods, and sequencing of construction operations. The contractor should be aware that slope heights, slope inclinations, or excavation depths (including utility trench excavations) should in no case exceed those specified in local, state, and/or federal safety regulations (e.g., OSHA Health and Safety Standards for Excavations, 29 CFR Part 1926, or successor regulations). Flatter slopes and/or trench shields may be required if loose, cohesionless soils and/or water are encountered along the slope face. Heavy construction equipment, building materials, excavated soil, and vehicular traffic should not be allowed within a lateral distance equal to one-third the slope height from the top of any excavation. During wet weather, earthen berms or other methods should be used to prevent runoff water from entering all excavations. All runoff water, seepage, and/or groundwater encountered within excavations should be collected and disposed of outside the construction limits.

6.1.4 Fill Materials

The native soils and existing fill materials encountered in our borings and broken-down bedrock materials, minus debris, rock particles larger than 3 inches in maximum dimension, and deleterious materials, may be suitable for use as engineered fill in the proposed building area. This material, however, should not be used as retaining wall backfill due to additional pressure it might impose on the retaining wall. Import non-expansive material should be used as retaining wall backfill. The native soils and broken-down bedrock materials should be well-mixed and moisture-conditioned. It should be reviewed and tested by Kleinfelder prior to being used as engineered fill.

Import fill soils should be nearly free of organic or other deleterious debris, essentially non-plastic, and contain rock particles less than 3 inches in maximum dimension. In general, well-graded mixtures of gravel, sand, non-plastic silt, and small quantities of cobbles, rock fragments, and/or clay are acceptable for use as import fill. All import fill materials to be used for engineered fill should be sampled and tested by the project Geotechnical Engineer prior to being transported to the site. Import fill guidelines are provided below.

Table 6-1
Import Fill Guidelines

Fill Requirement		Test Procedures	
		ASTM[1]	Caltrans[2]
Gradation			
Sieve Size	**Percent Passing**		
3 inch	100	D422	202
¾ inch	70-100	D422	202
No. 200	20-50	D422	202
Plasticity			
Liquid Limit	**Plasticity Index**		
<30	<12	D4318	204
Organic Content			
No visible organics		---	---
Expansion Potential			---
20 or less		D4829	---
Soluble Sulfates			
Less than 1,000 ppm		---	417
Soluble Chloride			
Less than 300 ppm		---	422
Resistivity			
Greater than 2,000 ohm-cm		---	643

[1]*American Society for Testing and Materials Standards (latest edition)*
[2]*State of California, Department of Transportation, Standard Test Methods (latest edition)*

Trench backfill and bedding placed within existing or future City right-of-ways should meet or exceed the requirements outlined in the current City specifications. Trench backfill or bedding placed outside existing or future right-of-ways could consist of native or imported soil that meets the requirements for fill material provided above. However, coarse-grained sand and/or gravel should be avoided for pipe bedding or trench zone backfill unless the material is fully enclosed in a geotextile filter fabric such as Mirafi 140N or an equivalent substitute. In a very moist or saturated condition, fine-grained soil can migrate into the coarse sand or gravel voids and cause "loss of ground" or differential settlement along and/or adjacent to the trenches, thereby leading to pipe joint displacement and pavement distress.

Trench backfill recommendations provided above should be considered minimum requirements only. More-stringent material specifications may be required to fulfill bedding requirements for specific types of pipe. The project Civil Engineer should develop these material specifications based on planned pipe types, bedding conditions, and other factors beyond the scope of this study.

6.1.5 Engineered Fill

All fill soils, either native or imported, required to bring the site to final grade should be compacted as engineered fill. Onsite clayey fill should be uniformly moisture-conditioned to a moisture content of at least 2 percent above the optimum moisture content, placed in horizontal lifts less than 8 inches in loose thickness, and compacted to between 90 and 93 percent of the maximum dry density as determined by ASTM Test Method D 1557. Imported granular fill should be uniformly moisture-conditioned to a moisture content to near the optimum moisture content, placed in horizontal lifts less than 8 inches in loose thickness, and compacted to at least 90 percent of the maximum dry density. Additional fill lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable. Discing and/or blending may be required to uniformly moisture-condition soils used for engineered fill. The uppermost 6 inches of exterior slabs or pavements where vehicular traffic is expected should be compacted to at least 95 percent of the maximum dry density. The subgrade should be stable, or non-pumping, prior to the construction of slabs or pavements.

All trench backfill in building or other structural areas should be placed and compacted in accordance with the recommendations provided above for engineered fill. During backfill, mechanical compaction of engineered fill is recommended.

New fill slopes, if any, should be constructed in level lifts, and proper keying and benching techniques should be used. Fill slopes should be constructed “fat” and trimmed back to expose the firm compacted surface.

6.1.6 Wet/Unstable Subgrade Mitigation

If construction is to proceed during the winter and spring months, the moisture content of the near-surface soils may be significantly above optimum. This condition, if encountered, could seriously delay grading by causing an unstable subgrade condition. Typical remedial measures include discing and aerating the soils, mixing the soils with dryer materials, removing and replacing the soils with an approved fill material, stabilization with a geotextile fabric or grid, or mixing the soils with an approved hydrating agent such as a lime or cement product. Our firm should be consulted prior to implementing any remedial measure to observe the unstable subgrade condition and provide site-specific recommendations.

6.2 SEISMIC DESIGN CRITERIA

We have conducted an updated site-specific ground motion analysis for the subject project, and the results are presented in Appendix E of our Geologic and Seismic Hazards Assessment report dated October 2017.

6.3 FOUNDATIONS

6.3.1 Footings

The building may be supported on shallow isolated spread footings and/or continuous wall footings founded on engineered fill. We recommend that a continuous exterior wall footing be used. A net allowable bearing pressure of 3,000 pounds per square foot (psf) for dead plus sustained live loading may be used to size column and continuous footings. A one-third increase in the allowable bearing pressures may be applied when considering short-term loading due to wind or seismic forces.

Footings should have a minimum width of 18 inches for continuous footings and 36 inches for isolated square footings. Spread or strip footings should be founded at least 30 inches below the lowest adjacent finished grade. Footings on slope, or near the top of slope, may have to be either deepen or have setback in accordance with the requirements as shown in Figure 1808A.7.1 of the 2016 California Building Code.

Total settlement of an individual foundation will vary depending on the plan dimensions of the foundation and the actual load supported. Based on the anticipated/assumed foundation dimensions and loads, we estimate the total and differential settlement to be on the order of 1 inch, provided the recommendations presented in this report are followed.

Prior to placing steel or concrete, footing excavations should be cleaned of all debris, loose or soft soil, and water. All footing excavations should be observed by the project Geotechnical Engineer just prior to placing steel or concrete to verify the recommendations contained herein are implemented during construction. The project Structural Engineer should evaluate footing configurations and reinforcement requirements to account for loading and settlement.

6.3.2 Mat Slabs

Mat slabs may be used as an alternative to shallow footings. The mats may be designed for an allowable pressure of 1,500 psf and should have a minimum depth at the edges of 18 inches. The allowable pressure may be increased by one-third for supporting total loads, including wind and seismic loads. The dead plus live load bearing pressure includes a safety factor of at least 2 and the total design bearing pressure of 2,000 psf (including wind and seismic) includes a safety factor of at least 1.5.

6.3.3 CIDH Piers

If piers are required to resist uplift loads for the new science building, Cast-in-drilled-hole (CIDH) piers can be used. The piers should derive their load capacities through skin friction on the side of the piers. For resistance to uplift loads, the effective weight of the piers and the skin friction between the piers and native soils may be used. An allowable skin friction value of 800 psf may be used to resist downward loads. A one-third increase is permitted for downward wind and/or seismic loading. The dead plus live load friction resistance includes a safety factor of at least 2 and the total design downward frictional resistance of about 1,100 psf (including wind and seismic) includes a safety factor of at least 1.5. Uplift loads for short-term conditions should not exceed 2/3 of the allowable downward skin friction (about 500 psf). These values may be doubled for the portion of piers that are in the claystone. Kleinfelder should review the design of any piers that use this increase. The piers should have a minimum depth of 10 feet for structures that are sensitive to seasonal shrinkage and swell movements, and 5 feet for stand-alone structures, such as light poles. The piers should have a minimum diameter of 18 inches and should be spaced at least 3 diameters apart (center to center) or skin friction capacity reductions may be necessary.

We recommend that steel reinforcement and concrete be placed within about 4 to 6 hours upon completion of each pier hole. As a minimum, the holes should be poured the same day they are drilled. The steel reinforcement should be centered in the pier hole. Concrete used for pier construction should be discharged vertically into the pier holes to reduce aggregate segregation. Under no circumstances should concrete be allowed to free-fall against either the steel reinforcement or the sides of the excavation during construction.

If water more than 6 inches deep is present during concrete placement, either the water needs to be pumped out or the concrete needs to be placed into the hole using tremie methods. Tremie methods may also be needed if after pumping the water quickly returns to the hole. If tremie methods are used, 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 design slump of from 4 to 6 inches if placed in a dry shaft without temporary casing, and from 6 to 8 inches if temporary casing is used. Casing is not anticipated for most of the piers due to the clayey nature of the soils within their probable depth. However, localized sandy layers found below the site may experience caving below the ground water level, which may require casing of some piers during construction. We expect conventional drilling equipment can be used for the installation of CIDH piers. However, hard drilling, especially in sandstone bedrock, could be encountered during construction. Also, old caissons or piers, if exist, could interfere with the installation of new CIDH piers. Unit prices for casing, de-watering, placement of concrete using tremie methods, and contingencies for removal of existing deep foundations and for slower than anticipated drilling should be obtained during bidding.

The bottom of the pier holes should be cleaned such that no more than two inches of loose soil remains in the hole prior to the placement of concrete. A concrete mix with a low water/cement ratio should be used in the construction of the piers to reduce shrinkage of the concrete. To increase the fluidity of the mix for improved consolidation and bond with the reinforcing steel, increased slump may be desirable. If this is the case, the slump should be increased via use of a plasticizer, rather than by adding water to the mix, because a low water to cement ratio is desired for shrinkage control.

A representative from Kleinfelder should be present to observe pier holes on a full-time basis to confirm bottom conditions prior to placing steel reinforcement. The soils exposed in the holes should not be allowed to dry prior to the placement of concrete, since such drying could have an adverse impact on the performance of the piers.

6.3.4 Resistance to Lateral Loads

Lateral loads applied against footings and mats may be resisted by a combination of friction between the foundation bottoms and the supporting subgrade, and by passive resistance acting against the vertical faces of the foundation. The frictional and passive resistance may be assumed in design to act concurrently. An allowable friction coefficient of 0.30 between the foundations and supporting subgrade soils may be used. For passive resistance at this site, an allowable equivalent fluid pressure (unit weight) of 350 pounds per cubic foot (pcf) may be used against the sides of foundations. For footings located near the top of a slope, or on a slope, an allowable passive equivalent fluid weight of 175 pcf is recommended. The friction coefficient and passive pressure values include factors of safety of about 1.5. We based these lateral load resistance values on the assumption that the concrete for footings are either placed directly against undisturbed soils or that the voids created from the use of forms are backfilled with soil (compacted to a minimum of 90 percent compaction, ASTM D 1557), or other approved material such as lean concrete.

Resistance to lateral loads for CIDH piers can be provided by passive resistance against the piers using an allowable equivalent fluid pressure of 350 pcf up to a maximum of 2,000 psf acting against the piers. The passive resistance may be applied to a width of twice the diameter of the piers. Piers should be spaced at least 6 diameters apart (center to center) or lateral resistance capacity reductions may be necessary. The passive pressure value includes a factor of safety of about 1.5.

Passive resistance in the upper foot of soil cover below finished grades should be neglected unless the ground surface is protected from erosion (or other disturbance that could remove this upper foot) by concrete slabs, pavements, or other such positive protection. If load-deflection (p-y) curves are needed for the design of the CIDH piers, we should be consulted.

6.4 MODULUS OF SUBGRADE REACTION

A modulus of subgrade reaction (Kv1) of 150 pounds per cubic inch (pci) may be used for the design of slabs-on-grade and mat slabs bearing on undisturbed site soils or properly compacted engineered fill. This value is based on the correlations to soil strength using one foot by one foot plate-load tests and should therefore be scaled (adjusted) to the mat/slab width. If the slab-on-grade floor is also underlain by sand, a vapor retarder, and gravel, the impact of those materials on the modulus of subgrade reaction must be taken into account in the structural design of the slab. The actual floor slab thickness and reinforcing should be designed by the structural engineer for the actual use and loads to be carried by the floor slab.

6.5 RETAINING WALLS

Retaining walls should be designed to resist lateral pressures caused by wall backfill/soil/bedrock, seismic pressures, and external surface loads. As stated in Section 6.1.4 of this report, the onsite materials could impose additional lateral pressure on the wall due to their potential expansive characteristic; therefore, should not be used as retaining wall backfill. Wall backfill should consist of a 1:1 wedge of import non-expansive fill. The magnitude of the lateral pressures will depend on wall flexibility, wall backslope configuration, backfill properties, the magnitude of seismic load, the magnitude of surcharge loads, and the back-drainage provisions. Basement walls or building walls are expected to be braced and restrained from deflection. Therefore, pressures against the basement walls or building walls should be based on at-rest earth pressures. The recommended lateral pressures presented as equivalent fluid weights are show in Table 6-2 below. The resultant force should be applied at a distance of H/3 above the bottom of the wall, where H = wall height. These recommended pressures contain a safety factor of 1.

Table 6-2

Recommended Lateral Pressures for Wall Design

	Equivalent Fluid Weight (pcf)*		
Backslope Condition	**Active**	**At-Rest**	**Seismic (active + seismic increment)**
Level	45	65	108
2H to 1V	65	94	167

Note: *Does not include lateral pressures due to groundwater and surcharges

The additional pressure due to a surcharge at the ground surface behind the wall acting against unrestrained walls may be taken as a uniform pressure estimated by multiplying the surface load by a factor of 0.3. The additional pressure due to a surcharge at the ground surface behind the wall acting against restrained walls may be taken as a uniform pressure estimated by multiplying the surface load by a factor of 0.5. These resultant forces should be applied at a distance of H/2 above the bottom of the wall, where H = wall height.

The recommended lateral pressures presented above were developed assuming that the walls are fully drained. Wall drainage should consist of a drain rock layer at least 12 inches thick and extend to within 1 foot of the ground surface. A 4-inch diameter perforated rigid-wall PVC, or similar material, pipe should be installed along the base of the walls in the drain rock with the perforations facing down. The bottom of pipe should rest on an about 2-inch thick bed of drain rock, and designed to slope to drain by gravity to a sump or other drainage facility. Drain rock should conform to Caltrans specifications for Class 2 Permeable Material. A 1-foot thick cap of clayey soil should be placed over the drain rock to inhibit surface water infiltration.

Kleinfelder should review and approve the proposed wall backfill materials before they are used in construction. Over-compaction of wall backfill should be avoided because increased compaction effort can result in lateral pressures significantly greater than those used in design.

We recommend that all backfill placed with 3 feet of the walls be compacted with hand-operated equipment. Placement of wall backfill should not begin until the wall concrete strength has reach a specific level as determined by the project Structural Engineer.

6.6 BUILDING SLABS-ON-GRADE

6.6.1 Subgrade Preparation

Prior to constructing interior concrete slabs supported-on-grade, surficial soils should be processed as recommended in Section 6.1.1 of this report.

6.6.2 Capillary Break

For floor slabs with moisture-sensitive floor coverings, or where moisture-sensitive storage is anticipated, we recommend the compacted subgrade be overlain with a minimum 4-inch thick of compacted crushed rock to serve as a capillary break. The material should have less than 5 percent by weight passing the No. 4 sieve size. A capillary break may reduce the potential for soil moisture migrating upwards toward the slab. In general, Caltrans Class 2 aggregate base or similar materials do not meet the above recommendations and should not be used to underlay interior concrete slabs supported-on-grade where moisture sensitive floor coverings or storage is anticipated.

6.6.3 Vapor Barrier

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. To reduce the impact of this subsurface moisture and the potential impact of introduced moisture (such as landscape irrigation or plumbing leaks) the current industry standard is to place a vapor retarder membrane (meeting ASTM E 1745 specifications) over the capillary break crushed rock layer. This membrane typically consists of polyvinyl or similar plastic sheeting at least 10 mils in thickness. Thicker polyolefin vapor barrier membranes (meeting ASTM E 1745 Class A) are currently available that are less prone to punctures and have much lower water vapor transmission rates. They should be installed according to American Concrete Institute (ACI) publication 302. The vapor retarder should be properly lapped and sealed. The joints between the sheets and the openings for utility piping should be lapped and taped. The sheeting should also be lapped into the sides of the footing trenches a minimum of 6 inches. Any puncture of the vapor retarder should be repaired prior to casting concrete.

Normally, a thin layer of moist clean sand (about two inches thick) is placed on the sheeting to facilitate concrete curing and to decrease the likelihood of slab curling. The final decision for the need and thickness of sand above the vapor barrier is the purview of the slab designer/structural engineer. The moisture vapor retarder is intended only to reduce moisture vapor transmission from the soil beneath the concrete and will not provide a waterproof or vapor proof barrier or reduce vapor transmission from sources above the retarder.

It should be noted that this system, although currently the industry standard, may not be completely effective in preventing moisture transmission through the floor slab and related floor covering 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 will 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 may 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 (water-cement ratio) and the permeability of the on-site soils affect slab moisture and can influence performance. In many cases, floor moisture problems are the result of water-cement ratio, improper curing of floor slabs, improper application of flooring adhesives, or a combination of these factors. Studies have shown that concrete water-cement ratios lower than 0.5 and proper slab curing can significantly reduce the potential for vapor transmission through floor slabs. 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 ACI Manual.

It is emphasized that we are not concrete slab-on-grade floor moisture-proofing experts. We make no guarantee nor provide any assurance that use of the 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 slab moisture protection.

All exterior utility trenches within 5 feet of perimeter foundations should be backfilled with compacted non-pervious fill material. Special care should be taken during installation of sub-floor water and sewer lines to reduce the possibility of leaks. Any utility penetrations through perimeter foundations should be completely sealed to prevent water intrusion beneath the floor slab.

6.7 EXTERIOR FLATWORK

Subgrade soils underlying exterior flatwork should be scarified 12 inches, moisture conditioned, and recompacted in accordance with the compaction requirements presented in this report. The subgrade preparation should extend beyond the proposed improvements a horizontal distance of at least 2 feet. The moisture content of the subgrade soils should be maintained at least 2 percent above optimum prior to the placement of any flatwork or engineered fill.

Where exterior flatwork is anticipated to be subjected to vehicular traffic, we recommend the flatwork be designed as pavement.

Moisture conditioning to the full 12-inch depth should be verified by the project Geotechnical Engineer's representative. Careful control of the water/cement ratio should be performed to avoid shrinkage cracking due to excess water or poor concrete finishing or curing. Unreinforced slabs should not be built in areas where further saturation may occur following construction. Proper moisture conditioning and compaction of subgrade soils is important. Even with proper site preparation, we anticipate that over time there will be some soil moisture change on the subgrade soil supporting the concrete flatwork. For example, exterior flatwork will be subjected to edge effects (shrink-swell) due to the drying out or wetting of subgrade soils where adjacent to landscaped or vacant areas. To help reduce edge effects, lateral cutoffs such as an inverted curb are suggested. Control joints should be also used to reduce the potential for flatwork panel cracks as a result of minor soil shrink-swell. Steel reinforcement will aid in keeping the control joints and other cracks closed.

6.8 SITE DRAINAGE

Proper site drainage is important for the long-term performance of the planned building, pavements, and concrete flatwork. The site should be graded to carry surface water away from the building foundations at a minimum gradient of 5 percent for a minimum lateral distance of 10 feet from the building limits (defined as the outside perimeter of building walls or footing outer limits, whichever results in the greatest building envelope), where feasible. Impervious surfaces, such as concrete flatwork and pavements, adjacent to the buildings should be sloped a minimum gradient of 2 percent. To reduce inducing surface water into the moisture sensitive clayey surface soil/rock, all roof gutters/leaders should be connected directly into a storm drainage system or drain on an impervious surface sloping away from the building, provided this does not create a safety hazard.

We recommend that landscape planters either not be located adjacent to buildings and pavement areas or be properly drained to area drains. Drought resistant plants and minimum watering are recommended for planters immediately adjacent to structures. No raised planters should be installed immediately adjacent to structures unless they are damp-proofed and have a drainpipe connected to an area drain outlet. Planters should be built such that water exiting from them will not seep into the foundation areas or beneath slabs and pavement. Where slabs or pavement areas abut landscaped areas, the aggregate base and subgrade soil should be protected against saturation.

Vertical cut-off structures are recommended to reduce lateral seepage under slabs from adjacent landscaped areas. Vertical cut-off structures may consist of deepened concrete perimeters, or equivalent, extending at least four (4) inches below the base/subgrade interface. Vertical cut-off structures should be poured neat against undisturbed native soil or compacted clayey fill. The cut-off structures should be continuous.

Roof water should be directed to fall on hardscape areas sloping to an area drain, or roof gutters and downspouts should be installed and routed to area drains.

In any event, maintenance personnel should be instructed to limit irrigation to the minimum actually necessary to properly sustain landscaping plants. Should excessive irrigation, waterline breaks or unusually high rainfall occur, saturated zones and "perched" groundwater may develop. Consequently, the site should be graded so that water drains away readily without saturating the foundation or landscaped areas. Potential sources of water such as water pipes, drains, and the like should be frequently examined for signs of leakage or damage. Any such leakage or damage should be promptly repaired. Wet utilities should also be designed to be watertight.

Surface water collected on top of slope should not be designed to flow over the top of slope and onto the slope surface. The top of slope should either be sloped back, or ditches be installed to intercept the water from flowing onto the slope. Erosion control measures should be provided on permanent cut or fill slope to reduce the potential of slope erosion.

6.9 SOIL CORROSIVITY

A composite sample of the near-surface soils of the near-surface soils encountered at the site was subjected to chemical analysis for the purpose of corrosion assessment. The sample was tested for chloride concentration, sulfate concentration, pH, oxidation reduction potential, and electrical resistivity by CERCO of Concord, California. The results of the tests are presented in Appendix C and are summarized in Table 6-3. If fill materials will be imported to the project site, similar corrosion potential laboratory testing should be completed on the imported material. Our scope of services does not include corrosion engineering and, therefore, a detailed analysis of the corrosion test results is not included in this report. A qualified corrosion engineer should be retained to review the test results and design protective systems that may be required. Kleinfelder may be able to provide those services.

Table 6-3
Corrosivity Laboratory Test Results

Boring and Depth	Resistivity, ohm-cm		pH	Oxidation Reduction Potential, mV	Water-Soluble Ion Concentration, ppm		
	Saturated	In-Situ Moisture			Chloride	Sulfide	Sulfate
B-3, sample 2C at 6'	1,100	720	7.86	+440	N.D.	N.D.	N.D.

Note: N.D. - None Detected

Ferrous metal and concrete elements in contact with soil, whether part of a foundation or part of the supported structure, are subject to degradation due to corrosion or chemical attack. Therefore, buried ferrous metal and concrete elements should be designed to resist corrosion and degradation based on accepted practices.

Based on the “10-point” method developed by the American Water Works Association (AWWA) in standard AWWA C105/A21.5, the soils at the site are extremely to highly corrosive to buried ferrous metal piping, cast iron pipes, or other objects made of these materials. We recommend that a corrosion engineer be consulted to recommend appropriate protective measures.

The degradation of concrete or cement grout can be caused by chemical agents in the soil or groundwater that react with concrete to either dissolve the cement paste or precipitate larger compounds within the concrete, causing cracking and flaking. The concentration of water-soluble sulfates in the soils is a good indicator of the potential for chemical attack of concrete or cement grout. The American Concrete Institute (ACI) in their publication “Guide to Durable Concrete*”* (ACI 201.2R-08) provides guidelines for this assessment. The sulfate test indicated the sample had a concentration below the detectable limit. The results of sulfate test indicate the potential for deterioration of concrete is mild, no special requirements should be necessary for the concrete mix.

Concrete and the reinforcing steel within it are at risk of corrosion when exposed to water-soluble chloride in the soil or groundwater. Chloride tests indicated the sample had concentrations below the detection limit. The project structural engineer should review this data to determine if remedial measures are necessary for the concrete reinforcing steel.

7 ADDITIONAL SERVICES

The review of final plans and specifications, and field observations and testing during construction by Kleinfelder is an integral part of the conclusions and recommendations made in this report. If Kleinfelder is not retained for these services, the client agrees to assume Kleinfelder's responsibility for any potential claims that may arise during construction. The actual tests and observations by Kleinfelder during construction will vary depending on type of project and soil/bedrock conditions. The tests and observations would be additional services provided by our firm. The costs for these services are not included in our current fee arrangements.

As a minimum, our construction services should include observation and testing during site preparation, grading, and placement of engineered fill, observation of foundation excavations prior to placement of reinforcing steel, and observation of CIDH construction. Many of our clients find it helpful to have concrete compressive tests performed for each building even though this information may not be required by any agency. It may also be helpful to perform a floor level and crack survey of all slab-on-grade floors prior to the application of any floor covering. The floor level survey can be readily performed by the client or as an additional service provided by Kleinfelder using a manometer device.

8 LIMITATIONS

The conclusions and recommendations of this report are provided for the design and construction of the proposed new science building at the Contra Costa College campus in San Pablo, California, as described in the text of this report. The conclusions and recommendations in this report are invalid if:

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

The scope of services was limited to the drilling of four test borings in area accessible to our drill rig. It should be recognized that definition and evaluation of subsurface conditions are difficult. Judgments leading to conclusions and recommendations are generally made with incomplete knowledge of the subsurface conditions present due to the limitations of data from field studies. The conclusions of this assessment are based on our subsurface exploration including four test boring drilled to a maximum depth of about 41½ feet; groundwater level measurements in the test borings during our field exploration; and geotechnical engineering analyses.

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

Recommendations contained in this report are based on our field observations and subsurface explorations, and our present knowledge of the proposed construction. It is possible that soil/bedrock or groundwater conditions could vary between or beyond the points explored. If soil/bedrock or groundwater conditions are encountered during 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.

As the geotechnical engineering firm that performed the geotechnical evaluation for this project, Kleinfelder should be retained to evaluate whether the recommendations of this report are properly incorporated in the design of this project and properly implemented during 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/bedrock conditions are encountered. As a minimum, Kleinfelder should be retained to provide the following continuing services for the project:

- Review the project plans and specifications, including any revisions or modifications
- Observe the site earthwork operations to assess whether the subgrade soils/bedrock are suitable for construction of foundations, slabs-on-grade, pavements and placement of engineered fill
- Evaluate whether engineered fill for the structure and other improvements is placed and compacted per the project specifications
- Observe foundation bearing soils to evaluate whether conditions are as anticipated
- Observe the construction of CIDH piers, if any

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

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

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

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

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

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

9 REFERENCES

Kleinfelder, 2003, Subsurface Fault Investigation, Proposed Addition to the Student Activities Building, Contra Costa College, San Pablo, California, dated December 2, 2003 (File No. 33133/SSA).

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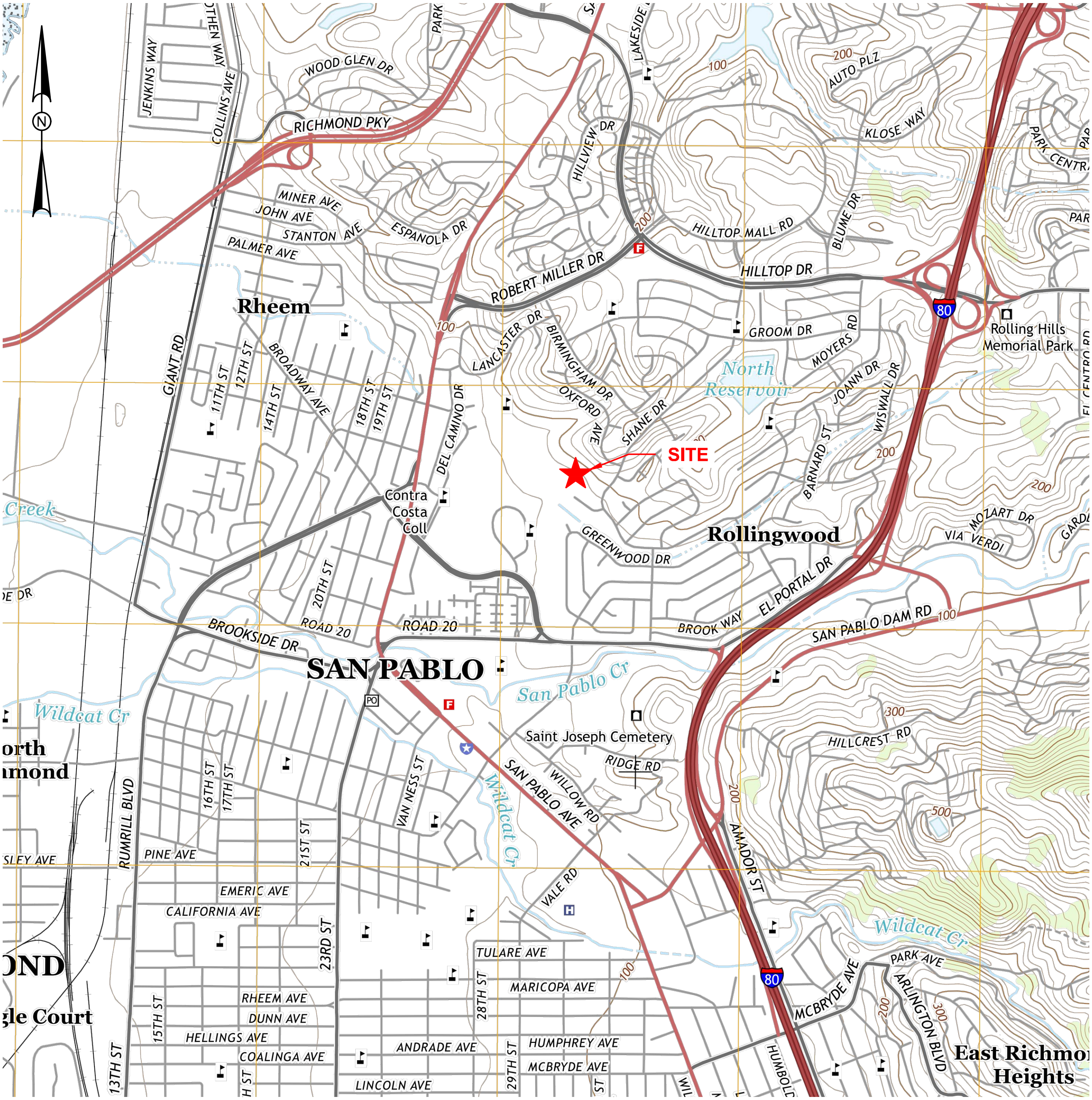
SMITHGROUP JJR, 2017, Meeting notes for Contra Costa College New Science Building, Project Number CCCCD C-4016, dated May 30, 2017.

United States Geological Survey, 1993, Topographic Map of the Richmond 7½-Minute Quadrangle, Scale 1:24,000.

FIGURES



REFERENCE:

VICINITY MAP CREATED FROM DATA COMPILED BY USGS US TOPO RICHMOND, CA. QUADRANGLE 7.5-MINUTE, 2015

0 2000 4000

SCALE: 1" = 2000' SCALE IN FEET

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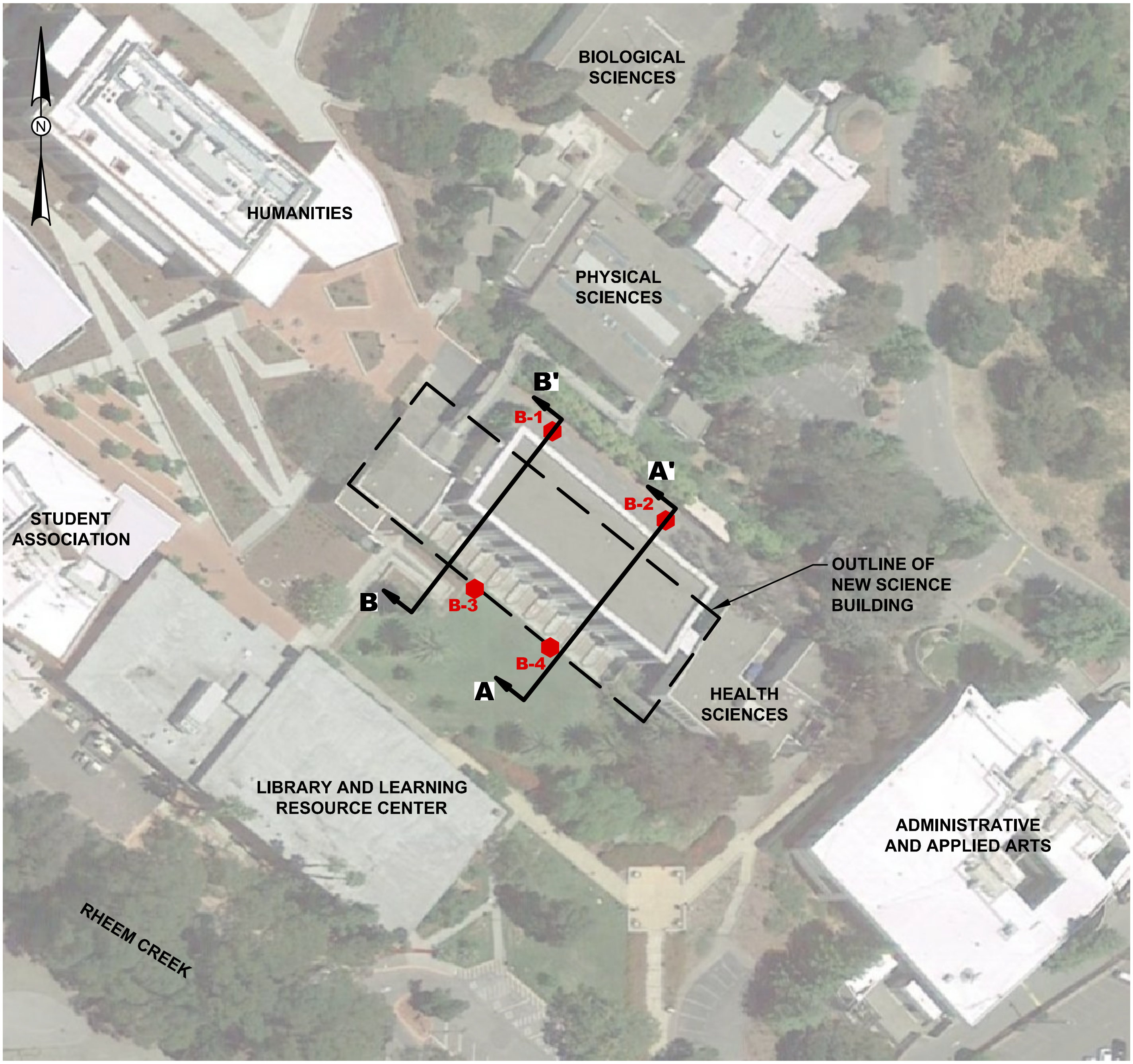
PROJECT NO.	20181569
DRAWN BY:	JDS
CHECKED BY:	JA
DATE:	08/30/2017
REVISED:	

SITE VICINITY MAP

CONTRA COSTA COMMUNITY COLLEGE
NEW SCIENCE BUILDING
2600 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA

FIGURE
1



0 100 200

SCALE: 1" = 100' SCALE IN FEET

REFERENCE:

IMAGE CREATED FROM DATA COMPILED BY GOOGLE EARTH PRO., DATED 5-20-2017

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LEGEND

SOIL BORING (By Kleinfelder, 2017)

B' B CROSS SECTION LINE (See Figures 3 & 4 for cross section view)

NOTE: All locations are approximate.

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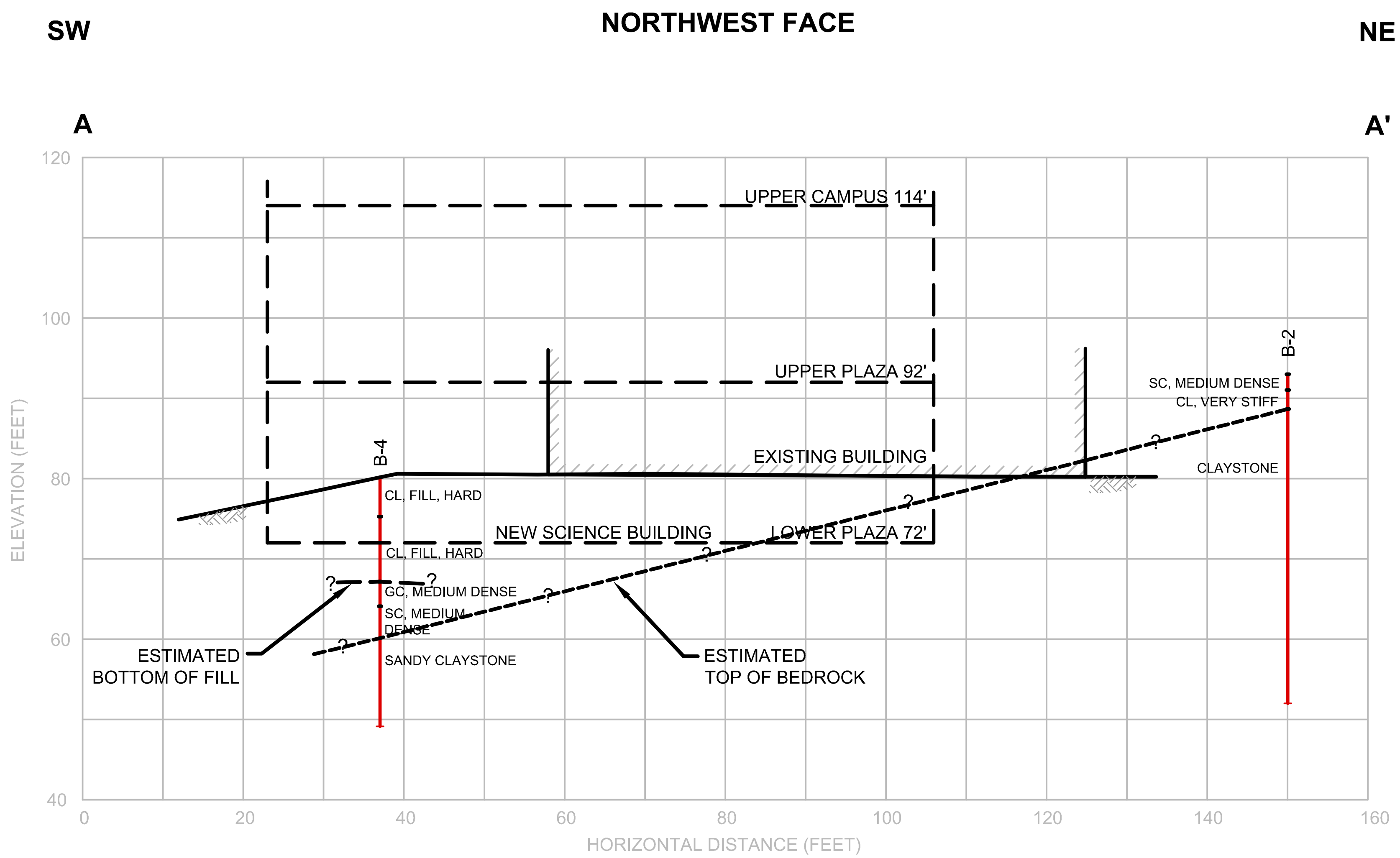
PROJECT NO. 20181569	SITE PLAN		FIGURE
DRAWN BY: JDS			
CHECKED BY: EM	CONTRA COSTA COMMUNITY COLLEGE NEW SCIENCE BUILDING 2600 MISSION BELL DRIVE SAN PABLO, CALIFORNIA		2
DATE: 10/01/2017			
REVISED:			

PLOTTED: 10/12/2017 9:53 AM BY: jeff sala

CAD FILE: W:\2018\20181569.001A_CCCCD_C-4016 GeoSeis\GRAPHICs\20181569.dwg



0 20 40

SCALE: 1" = 20' SCALE IN FEET
VERTICAL AND HORIZONTAL

NOTES:

1. SEE FIGURE 2 FOR LOCATION OF CROSS SECTION.

2. " BOTTOM OF FILL LINE" AND "TOP OF BEDROCK LINE" ARE ROUGH ESTIMATED INTERFACE LINES BASED ON LIMITED SUBSURFACE INFORMATION. THE ACTUAL SUBSURFACE CONDITIONS COULD BE SIGNIFICANTLY DIFFERENT FROM THOSE SHOWN ABOVE DUE TO PAST GRADING AND CONSTRUCTION AT THE SITE.

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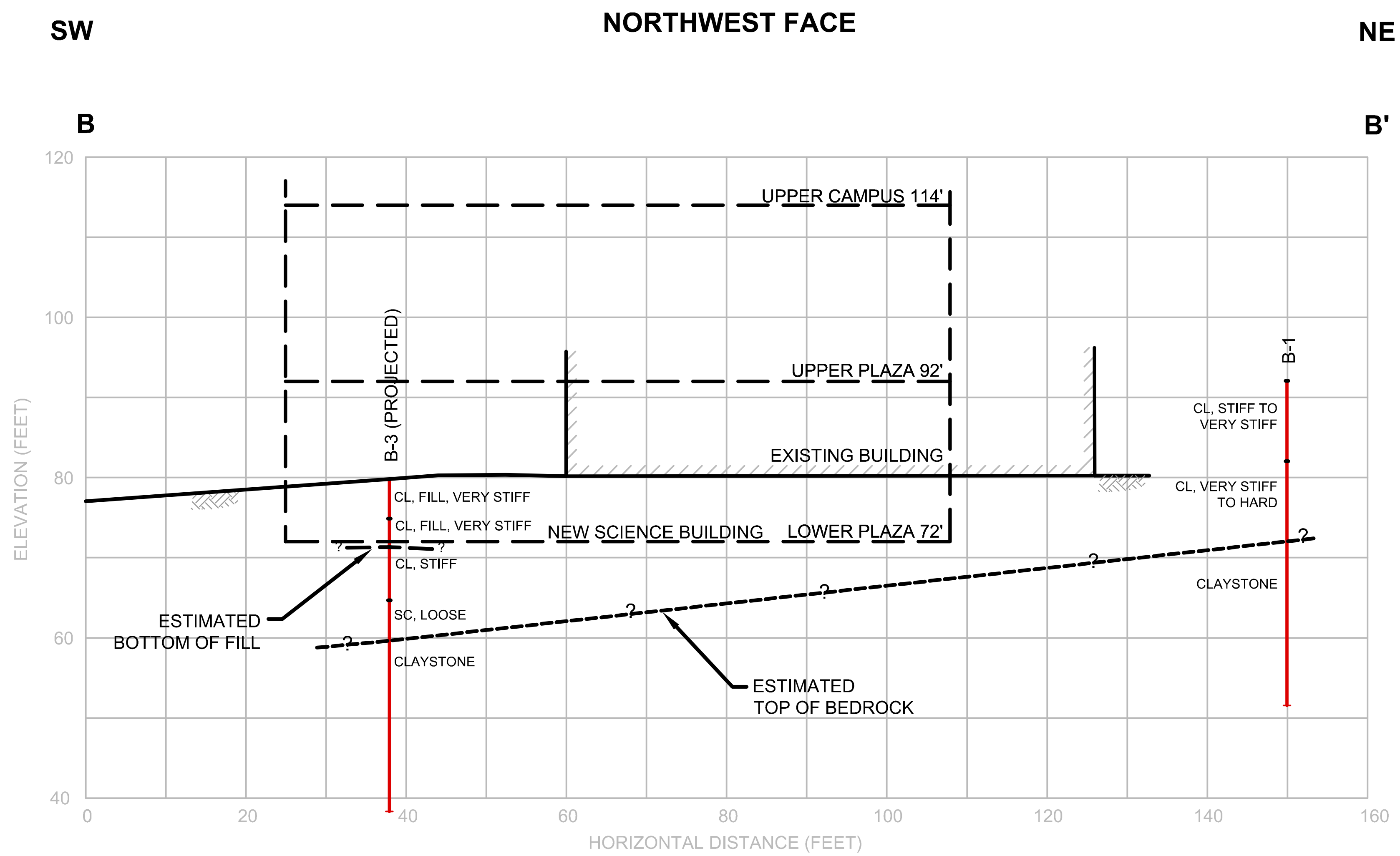
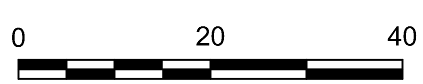
PROJECT NO. 20181569	CROSS SECTION A-A'		PLATE
DRAWN BY JDS			
CHECKED BY EM	CONTRA COSTA COMMUNITY COLLEGE NEW SCIENCE BUILDING 2600 MISSION BELL DRIVE SAN PABLO, CALIFORNIA		3
DATE: 10/04/2017			
REVISED:			

PLOTTED: 10/5/2017 5:21 PM BY: jeff sala
CAD FILE: W:\2018\20181569.001A_CCCCD_C-4016 GeoSeis\GRAPHICs\20181569.dwg LAYOUT: SECT A-A





SCALE: 1" = 20' SCALE IN FEET
VERTICAL AND HORIZONTAL

NOTES:

1. SEE FIGURE 2 FOR LOCATION OF CROSS SECTION.
2. " BOTTOM OF FILL LINE" AND "TOP OF BEDROCK LINE" ARE ROUGH ESTIMATED INTERFACE LINES BASED ON LIMITED SUBSURFACE INFORMATION. THE ACTUAL SUBSURFACE CONDITIONS COULD BE SIGNIFICANTLY DIFFERENT FROM THOSE SHOWN ABOVE DUE TO PAST GRADING AND CONSTRUCTION AT THE SITE.

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PROJECT NO. 20181569	CROSS SECTION B-B'		PLATE
DRAWN BY JDS			
CHECKED BY EM	CONTRA COSTA COMMUNITY COLLEGE NEW SCIENCE BUILDING 2600 MISSION BELL DRIVE SAN PABLO, CALIFORNIA		4
DATE: 10/04/2017			
REVISED:			

APPENDIX A
LOGS OF TEST BORINGS

SAMPLER AND DRILLING METHOD GRAPHICS

- BULK / GRAB / BAG SAMPLE
- MODIFIED CALIFORNIA SAMPLER (2 or 2-1/2 in. (50.8 or 63.5 mm.) outer diameter)
- CALIFORNIA SAMPLER (3 in. (76.2 mm.) outer diameter)
- STANDARD PENETRATION SPLIT SPOON SAMPLER (2 in. (50.8 mm.) outer diameter and 1-3/8 in. (34.9 mm.) inner diameter)
- SHELBY TUBE SAMPLER
- HOLLOW STEM AUGER
- SOLID STEM AUGER
- WASH BORING

GROUND WATER GRAPHICS

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

NOTES

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

ABBREVIATIONS
WOH - Weight of Hammer
WOR - Weight of Rod

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487)

COARSE GRAINED SOILS (More than half of material is larger than the #200 sieve)	**GRAVELS** (More than half of coarse fraction is larger than the #4 sieve)	CLEAN GRAVEL WITH <5% FINES	Cu ≥4 and 1≤ Cc≤3	**GW**	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
			Cu <4 and/ or 1>Cc >3	**GP**	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
		GRAVELS WITH 5% TO 12% FINES	Cu ≥4 and 1≤ Cc≤3	**GW-GM**	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GW-GC	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
			Cu <4 and/ or 1>Cc >3	**GP-GM**	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GP-GC	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
		GRAVELS WITH > 12% FINES		**GM**	SILTY GRAVELS, GRAVEL-SILT-SAND MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
				GC-GM	CLAYEY GRAVELS, GRAVEL-SAND-CLAY-SILT MIXTURES
	SANDS (More than half of coarse fraction is smaller than the #4 sieve)	CLEAN SANDS WITH <5% FINES	Cu ≥6 and 1≤ Cc≤3	**SW**	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
			Cu <6 and/ or 1>Cc >3	**SP**	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
		SANDS WITH 5% TO 12% FINES	Cu ≥6 and 1≤ Cc≤3	**SW-SM**	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SW-SC	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
			Cu <6 and/ or 1>Cc >3	**SP-SM**	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SP-SC	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
		SANDS WITH > 12% FINES		**SM**	SILTY SANDS, SAND-GRAVEL-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-GRAVEL-CLAY MIXTURES
				SC-SM	CLAYEY SANDS, SAND-SILT-CLAY MIXTURES

FINE GRAINED SOILS (More than half of material is smaller than the #200 sieve)	SILTS AND CLAYS (Liquid Limit less than 50)	**ML**	INORGANIC SILTS AND VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS, SILTS WITH SLIGHT PLASTICITY
		CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		CL-ML	INORGANIC CLAYS-SILTS OF LOW PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS (Liquid Limit greater than 50)	**MH**	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILT
		CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
		OH	ORGANIC CLAYS & ORGANIC SILTS OF MEDIUM-TO-HIGH PLASTICITY

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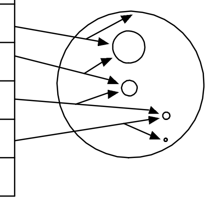
PROJECT NO.: 20181569	GRAPHICS KEY	FIGURE
DRAWN BY: MAP/JDS		
CHECKED BY: OK	CONTRA COSTA COMMUNITY COLLEGE NEW SCIENCE BUILDING 2600 MISSION BELL DRIVE SAN PABLO, CALIFORNIA	A-1
DATE: 9/19/2017		
REVISED: -		

GRAIN SIZE

DESCRIPTION		SIEVE SIZE	GRAIN SIZE	APPROXIMATE SIZE
Boulders		>12 in. (304.8 mm.)	>12 in. (304.8 mm.)	Larger than basketball-sized
Cobbles		3 - 12 in. (76.2 - 304.8 mm.)	3 - 12 in. (76.2 - 304.8 mm.)	Fist-sized to basketball-sized
Gravel	coarse	3/4 -3 in. (19 - 76.2 mm.)	3/4 -3 in. (19 - 76.2 mm.)	Thumb-sized to fist-sized
	fine	#4 - 3/4 in. (#4 - 19 mm.)	0.19 - 0.75 in. (4.8 - 19 mm.)	Pea-sized to thumb-sized
Sand	coarse	#10 - #4	0.079 - 0.19 in. (2 - 4.9 mm.)	Rock salt-sized to pea-sized
	medium	#40 - #10	0.017 - 0.079 in. (0.43 - 2 mm.)	Sugar-sized to rock salt-sized
	fine	#200 - #40	0.0029 - 0.017 in. (0.07 - 0.43 mm.)	Flour-sized to sugar-sized
Fines		Passing #200	<0.0029 in. (<0.07 mm.)	Flour-sized and smaller



SECONDARY CONSTITUENT

Term of Use	AMOUNT: Secondary Constituent is Fine Grained	AMOUNT: Secondary Constituent is Coarse Grained
Trace	<5%	<15%
With	≥5 to <15%	≥15 to <30%
Modifier	≥15%	≥30%

MOISTURE CONTENT

DESCRIPTION	FIELD TEST
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

CEMENTATION

DESCRIPTION	FIELD TEST
Weakly	Crumbles or breaks with handling or slight finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

CONSISTENCY - FINE-GRAINED SOIL

CONSISTENCY	SPT - N_{60} (# blows / ft)	Pocket Pen (tsf)	UNCONFINED COMPRESSIVE STRENGTH (Q_u)(psf)	VISUAL / MANUAL CRITERIA
Very Soft	<2	PP < 0.25	<500	Thumb will penetrate more than 1 inch (25 mm). Extrudes between fingers when squeezed.
Soft	2 - 4	0.25≤ PP <0.5	500 - 1000	Thumb will penetrate soil about 1 inch (25 mm). Remolded by light finger pressure.
Medium Stiff	4 - 8	0.5≤ PP <1	1000 - 2000	Thumb will penetrate soil about 1/4 inch (6 mm). Remolded by strong finger pressure.
Stiff	8 - 15	1≤ PP <2	2000 - 4000	Can be imprinted with considerable pressure from thumb.
Very Stiff	15 - 30	2≤ PP <4	4000 - 8000	Thumb will not indent soil but readily indented with thumbnail.
Hard	>30	4≤ PP	>8000	Thumbnail will not indent soil.

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

REACTION WITH HYDROCHLORIC ACID

DESCRIPTION	FIELD TEST
None	No visible reaction
Weak	Some reaction, with bubbles forming slowly
Strong	Violent reaction, with bubbles forming immediately

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	SPT-N_{60} (# blows/ft)	MODIFIED CA SAMPLER (# blows/ft)	CALIFORNIA SAMPLER (# blows/ft)	RELATIVE DENSITY (%)
Very Loose	<4	<4	<5	0 - 15
Loose	4 - 10	5 - 12	5 - 15	15 - 35
Medium Dense	10 - 30	12 - 35	15 - 40	35 - 65
Dense	30 - 50	35 - 60	40 - 70	65 - 85
Very Dense	>50	>60	>70	85 - 100

FROM TERZAGHI AND PECK, 1948

PLASTICITY

DESCRIPTION	LL	FIELD TEST
Non-plastic	NP	A 1/8-in. (3 mm.) thread cannot be rolled at any water content.
Low (L)	< 30	The thread can barely be rolled and the lump or thread cannot be formed when drier than the plastic limit.
Medium (M)	30 - 50	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.
High (H)	> 50	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.

STRUCTURE

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

ANGULARITY

DESCRIPTION	CRITERIA
Angular	Particles have sharp edges and relatively plane sides with unpolished surfaces.
Subangular	Particles are similar to angular description but have rounded edges.
Subrounded	Particles have nearly plane sides but have well-rounded corners and edges.
Rounded	Particles have smoothly curved sides and no edges.

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PROJECT NO.: 20181569	SOIL DESCRIPTION KEY	FIGURE
DRAWN BY: MAP/JDS		
CHECKED BY: OK	CONTRA COSTA COMMUNITY COLLEGE NEW SCIENCE BUILDING 2600 MISSION BELL DRIVE SAN PABLO, CALIFORNIA	A-2
DATE: 9/19/2017		
REVISED: -		

PLOTTED: 09/19/2017 09:36 AM BY: JSala

OFFICE FILTER: PLEASANTON

PROJECT NUMBER: 20181569.001A

gINT FILE: Klf_gint_master_2017

gINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2017.GLB [LEGEND 2 (SOIL DESC KEY)]

INFILLING TYPE

NAME	ABBR	NAME	ABBR
Albite	Al	Muscovite	Mus
Apatite	Ap	None	No
Biotite	Bi	Pyrite	Py
Clay	Cl	Quartz	Qz
Calcite	Ca	Sand	Sd
Chlorite	Ch	Sericite	Ser
Epidote	Ep	Silt	Si
Iron Oxide	Fe	Talc	Ta
Manganese	Mn	Unknown	Uk

DENSITY/SPACING OF DISCONTINUITIES

DESCRIPTION	SPACING CRITERIA
Unfractured	>6 ft. (>1.83 meters)
Slightly Fractured	2 - 6 ft. (0.061 - 1.83 meters)
Moderately Fractured	8 in - 2 ft. (203.20 - 609.60 mm)
Highly Fractured	2 - 8 in (50.80 - 203.30 mm)
Intensely Fractured	<2 in (<50.80 mm)

ADDITIONAL TEXTURAL ADJECTIVES

DESCRIPTION	RECOGNITION
Pit (Pitted)	Pinhole to 0.03 ft. (3/8 in.) (>1 to 10 mm.) openings
Vug (Vuggy)	Small openings (usually lined with crystals) ranging in diameter from 0.03 ft. (3/8 in.) to 0.33 ft. (4 in.) (10 to 100 mm.)
Cavity	An opening larger than 0.33 ft. (4 in.) (100 mm.), size descriptions are required, and adjectives such as small, large, etc., may be used
Honeycombed	If numerous enough that only thin walls separate individual pits or vugs, this term further describes the preceding nomenclature to indicate cell-like form.
Vesicle (Vesicular)	Small openings in volcanic rocks of variable shape and size formed by entrapped gas bubbles during solidification.

ADDITIONAL TEXTURAL ADJECTIVES

DESCRIPTION	CRITERIA
Unweathered	No evidence of chemical / mechanical alternation; rings with hammer blow.
Slightly Weathered	Slight discoloration on surface; slight alteration along discontinuities; <10% rock volume altered.
Moderately Weathered	Discoloring evident; surface pitted and alteration penetration well below surface; Weathering "halos" evident; 10-50% rock altered.
Highly Weathered	Entire mass discolored; Alteration pervading most rock, some slight weathering pockets; some minerals may be leached out.
Decomposed	Rock reduced to soil with relic rock texture/structure; Generally molded and crumbled by hand.

RELATIVE HARDNESS / STRENGTH DESCRIPTIONS

GRADE		UCS (Mpa)	FIELD TEST
R0	Extremely Weak	0.25 - 1.0	Indented by thumbnail
R1	Very Weak	1.0 - 5.0	Crumbles under firm blows of geological hammer, can be peeled by a pocket knife.
R2	Weak	5.0 - 25	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer.
R3	Medium Strong	25 - 50	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single firm blow of a geological hammer.
R4	Strong	50 - 100	Specimen requires more than one blow of geological hammer to fracture it.
R5	Very Strong	100 - 250	Specimen requires many blows of geological hammer to fracture it.
R6	Extremely Strong	> 250	Specimen can only be chipped with a geological hammer.

ROCK QUALITY DESIGNATION (RQD)

DESCRIPTION	RQD (%)
Very Poor	0 - 25
Poor	25 - 50
Fair	50 - 75
Good	75 - 90
Excellent	90 - 100

APERTURE

DESCRIPTION	CRITERIA [in (mm)]
Tight	<0.04 (<1)
Open	0.04 - 0.20 (1 - 5)
Wide	>0.20 (>5)

BEDDING CHARACTERISTICS

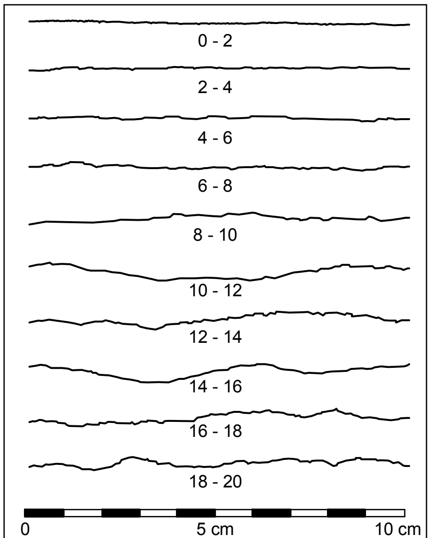
DESCRIPTION	Thickness [in (mm)]
Very Thick Bedded	>36 (>915)
Thick Bedded	12 - 36 (305 - 915)
Moderately Bedded	4 - 12 (102 - 305)
Thin Bedded	1 - 4 (25 - 102)
Very Thin Bedded	0.4 - 1 (10 - 25)
Laminated	0.1 - 0.4 (2.5 - 10)
Thinly Laminated	<0.1 (<2.5)

Bedding Planes — Planes dividing the individual layers, beds, or stratigraphy of rocks.

Joint — Fracture in rock, generally more or less vertical or traverse to bedding.

Seam — Applies to bedding plane with unspecified degree of weather.

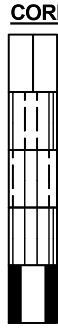
JOINT ROUGHNESS COEFFICIENT (JRC)



From Barton and Choubey, 1977

RQD — Rock-quality designation (RQD) Rough measure of the degree of jointing or fracture in a rock mass, measured as a percentage of the drill core in lengths of 10 cm. or more.

CORE SAMPLER TYPE GRAPHICS



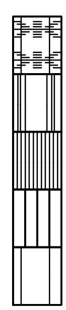
CORE SAMPLER

AQ CORE BARREL (1.067 in. (27.1 mm.) core diameter)

AX CORE BARREL (1.185 in. (30.1 mm.) core diameter)

BQ CORE BARREL (1.433 in. (36.4 mm.) core diameter)

CONTINUOUS CORE SAMPLE (2.000 in. (50.8 mm.) core diameter)



EX CORE BARREL (0.846 in. (21.5 mm.) core diameter)

HQ CORE SAMPLE (2.500 in. (63.5 mm.) core diameter)

NQ CORE SAMPLE (1.874 in. (47.6 mm.) core diameter)

NO RECOVERY CORE SAMPLE

NX CORE SAMPLE (2.154 in. (54.7 mm.) core diameter)

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PROJECT NO.:	20181569
DRAWN BY:	MAP/JDS
CHECKED BY:	OK
DATE:	9/19/2017
REVISED:	-

ROCK DESCRIPTION KEY

CONTRA COSTA COMMUNITY COLLEGE
NEW SCIENCE BUILDING
2600 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA

FIGURE

A-3

PLOTTED: 09/19/2017 09:42 AM BY: JSala
OFFICE FILTER: PLEASANTON
PROJECT NUMBER: 20181569.001A
gINT FILE: Klf_gint_master_2017
gINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2017.GLB [LEGEND 3 (ROCK DESC KEY)]

BORING LOG B-1

Date Begin - End:	8/11/2017	**Drilling Co.-Lic.#:**	Gregg - #CA107979		
Logged By:	J. Anderson	**Drill Crew:**	Jeremy		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Truck Mounted M11	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Cloudy	**Exploration Diameter:**	Approx. 6 in.		

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 37.96986° N
Longitude: -122.33678° E
Approximate Ground Surface Elevation (ft.): 92.00
Surface Condition: Asphalt

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
92	0		approximately 2-inches of asphalt											
			Sandy Lean CLAY with Gravel (CL): low plasticity, yellowish brown, moist, stiff to very stiff, subrounded to subangular gravel											
90					BC=5 7 9	12"		18.9	109.7					
	5		olive brown, stiff to very stiff		BC=5 6 8	12"		19.1	108.8					TXUU: c = 2.12 ksf
85														
	10		**Sandy Lean CLAY (CL)**: fine-grained sand, some gravel, medium plasticity, reddish yellow mottled, moist, very stiff		BC=6 10 14	12"		14.0	115.8					TXUU: c = 2.55 ksf
80														
			some angular claystone fragments, yellowish brown, hard											
	15				BC=12 18 22	12"								
75														
	20		**CLAYSTONE**: fine-grained, medium plasticity, yellowish brown, moderately weathered, weak to medium strong		BC=22 36 50/5"	11"								
70														
	25				BC=11 29 50	12"								
65														
	30		moderately weathered, weak to medium strong, interbedded with siltstone		BC=29 50/3"	8"								
60														

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PROJECT NO.: 20181569	BORING LOG B-1		FIGURE
DRAWN BY: MAP/JDS			
CHECKED BY: OK	CONTRA COSTA COMMUNITY COLLEGE NEW SCIENCE BUILDING 2600 MISSION BELL DRIVE SAN PABLO, CALIFORNIA		A-4
DATE: 9/19/2017			
REVISED: -			

Date Begin - End:	8/11/2017	**Drilling Co.-Lic.#:**	Gregg - #CA107979		**BORING LOG B-1**
Logged By:	J. Anderson	**Drill Crew:**	Jeremy		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Truck Mounted M11	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Cloudy	**Exploration Diameter:**	Approx. 6 in.		

Latitude: 37.96986° N
Longitude: -122.33678° E
Approximate Ground Surface Elevation (ft.): 92.00
Surface Condition: Asphalt

FIELD EXPLORATION							LABORATORY RESULTS							
Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
			CLAYSTONE: fine-grained, yellowish brown, moderately weathered, medium strong		BC=26 50	2"								
55														
	40		- light brownish gray, slightly weathered, medium strong to strong		BC=44 50/2"	8"								
50														
	45													
45														
	50													
40														
	55													
35														
	60													
30														
	65													
25														

The boring was terminated at approximately 40.5 ft. below ground surface. The boring was backfilled with cement grout on August 11, 2017.

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

GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder.

Date Begin - End:	8/11/2017	Drilling Co.-Lic.#:	Gregg - #CA107979		BORING LOG B-2
Logged By:	J. Anderson	Drill Crew:	Jeremy		
Hor.-Vert. Datum:	Not Available	Drilling Equipment:	Truck Mounted M11	Hammer Type - Drop:	140 lb. Auto - 30 in.
Plunge:	-90 degrees	Drilling Method:	Hollow Stem Auger		
Weather:	Cloudy	Exploration Diameter:	Approx. 6 in.		

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 37.96973° N
Longitude: -122.33647° E
Approximate Ground Surface Elevation (ft.): 93.00
Surface Condition: Asphalt

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
			approximately 2-inches of asphalt											
			Clayey SAND (SC): fine to medium-grained sand, low plasticity, mottled yellowish brown, dry, medium dense		BC=10 12 14	12"								
90			**Lean CLAY (CL)**: medium plasticity, yellowish brown, moist, very stiff					11.3	110.8					
	5		**CLAYSTONE**: fine-grained, yellowish brown, moderately weathered to highly weathered, weak to medium strong		BC=17 18 26	6"								
85														
			reddish yellow, fragmented moderately weathered, weak to medium strong											
	10				BC=16 14 50/4"	10"		9.5	118.9					
80														
	15		olive brown, weak to medium strong		BC=14 36 50/5"	2"								Very hard drilling
75														
	20		- yellowish brown with reddish brown stains, moderately weathered, intensely fractured medium strong		BC=23 50	4"								
70														
	25		weak		BC=13 14 20	2"								
65														
	30		medium-grained, yellow, moderately weathered, weak, highly fractured, interbedded with subrounded gravel		BC=11 18 34	10"								
60														

Date Begin - End: 8/11/2017	**Drilling Co.-Lic.#:** Gregg - #CA107979		**BORING LOG B-2**
Logged By: J. Anderson	**Drill Crew:** Jeremy		
Hor.-Vert. Datum: Not Available	**Drilling Equipment:** Truck Mounted M11	**Hammer Type - Drop:** 140 lb. Auto - 30 in.	
Plunge: -90 degrees	**Drilling Method:** Hollow Stem Auger		
Weather: Cloudy	**Exploration Diameter:** Approx. 6 in.		

			FIELD EXPLORATION				LABORATORY RESULTS							
Approximate Elevation (feet)	Depth (feet)	Graphical Log	Latitude: 37.96973° N Longitude: -122.33647° E Approximate Ground Surface Elevation (ft.): 93.00 Surface Condition: Asphalt Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
			CLAYSTONE: fine-grained, yellowish brown, moderately weathered to highly weathered, weak to medium strong fine-grained, light brownish gray, weak to medium strong, highly fractured		BC=9 29 50/5"	3"								
55														
	40				BC=21 50									
50														
	45													
45														
	50													
40														
	55													
35														
	60													
30														
	65													
25														

The boring was terminated at approximately 41 ft. below ground surface. The boring was backfilled with cement grout on August 11, 2017.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.
GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder.

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20181569	BORING LOG B-2	FIGURE
	DRAWN BY: MAP/JDS		A-5
	CHECKED BY: OK	CONTRA COSTA COMMUNITY COLLEGE NEW SCIENCE BUILDING 2600 MISSION BELL DRIVE SAN PABLO, CALIFORNIA	
	DATE: 9/19/2017		
	REVISED: -		PAGE: 2 of 2

BORING LOG B-3

Date Begin - End:	8/18/2017	**Drilling Co.-Lic.#:**	Gregg - #CA107979		
Logged By:	J. Anderson	**Drill Crew:**	Jeremy and Leo		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	D42	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Overcast	**Exploration Diameter:**	Approx. 6 in.		

Latitude: 37.96965° N
Longitude: -122.33695° E
Approximate Ground Surface Elevation (ft.): 80.00
Surface Condition: Grass

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
			Sandy Lean CLAY (CL): medium plasticity, olive brown, moist, very stiff, (FILL)		BC=3 8 13	12"						27	12	
75	5		**Lean CLAY with Sand (CL)**: medium plasticity, olive brown, moist, very stiff, (FILL)		BC=4 8 12	11"								
			Sandy Lean CLAY (CL): medium plasticity, yellowish brown, moist, stiff											
70	10				BC=2 4 7	12"		26.8	94.7					TXUU: c = 1.25 ksf
65	15		**Clayey SAND (SC)**: non-plastic to low plasticity, yellowish brown, moist, loose		BC=4 4 5	12"	SC				49	33	18	
60	20		**CLAYSTONE**: fine-grained, olive brown, weak to medium strong, interbedded with siltstone		BC=20 42 50/5"	11"								
55	25		light gray, medium strong to strong		BC=40 50/5"	11"								
50	30		moderately to slightly weathered, weak, highly fractured		BC=20 25 26	12"								

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PROJECT NO.:	20181569
DRAWN BY:	MAP/JDS
CHECKED BY:	OK
DATE:	9/19/2017
REVISED:	-

BORING LOG B-3

CONTRA COSTA COMMUNITY COLLEGE
NEW SCIENCE BUILDING
2600 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA

FIGURE
A-6

Date Begin - End:	8/18/2017	Drilling Co.-Lic.#:	Gregg - #CA107979		BORING LOG B-3
Logged By:	J. Anderson	Drill Crew:	Jeremy and Leo		
Hor.-Vert. Datum:	Not Available	Drilling Equipment:	D42	Hammer Type - Drop:	140 lb. Auto - 30 in.
Plunge:	-90 degrees	Drilling Method:	Hollow Stem Auger		
Weather:	Overcast	Exploration Diameter:	Approx. 6 in.		

Latitude: 37.96965° N
Longitude: -122.33695° E
Approximate Ground Surface Elevation (ft.): 80.00
Surface Condition: Grass

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
			CLAYSTONE: fine-grained, olive gray, weak		BC=18 27 30	12"								
40	40		olive, medium strong		BC=17 36 50/5"									
35	45													
30	50													
25	55													
20	60													
15	65													

The boring was terminated at approximately 41.5 ft. below ground surface. The boring was backfilled with cement grout on August 18, 2017.

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

GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder.

BORING LOG B-4

Date Begin - End: 8/18/2017	**Drilling Co.-Lic.#:** Gregg - #CA107979		
Logged By: J. Anderson	**Drill Crew:** Jeremy and Leo		
Hor.-Vert. Datum: Not Available	**Drilling Equipment:** D42	**Hammer Type - Drop:** 140 lb. Auto - 30 in.	
Plunge: -90 degrees	**Drilling Method:** Hollow Stem Auger		
Weather: Overcast	**Exploration Diameter:** Approx. 6 in.		

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 37.96953° N
Longitude: -122.33673° E
Approximate Ground Surface Elevation (ft.): 80.00
Surface Condition: Grass

Approximate Elevation (feet)	Depth (feet)	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		Lean Fat CLAY with Sand (CL): medium to high plasticity, olive brown, moist, hard, (FILL)											
				BC=11 13 16 PP=4-4.5+	11"								
75	5	**Lean CLAY with Sand (CL)**: medium plasticity, olive brown, moist, hard, (FILL)		BC=9 12 23 PP=4.5	12"						43	28	
70	10	increase in sand content, very stiff, organics, brick fragments with gravel and brick at 11.5 feet		BC=9 11 12 PP=1.5-1.75									
		Clayey GRAVEL with Sand (GC): dark brown, moist, medium dense, fine to coarse gravel											
65	15	**Clayey SAND with Gravel (SC)**: medium to coarse-grained, olive brown, moist, medium dense		BC=17 18 12						16			
60	20	**Sandy CLAYSTONE**: fine-grained, olive, weak to medium strong, moderately weathered, interbedded with siltstone		BC=20 27 25	12"								
55	25	medium strong		BC=18 33 48	12"								
50	30	medium strong to strong		BC=27 50/5"									

The boring was terminated at approximately 31 ft. below ground surface. The boring was backfilled with cement grout on August 18, 2017.

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

GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder.

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PROJECT NO.:	20181569
DRAWN BY:	MAP/JDS
CHECKED BY:	OK
DATE:	9/19/2017
REVISED:	-

BORING LOG B-4

CONTRA COSTA COMMUNITY COLLEGE
NEW SCIENCE BUILDING
2600 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA

FIGURE
A-7

APPENDIX B
LABORATORY DATA

Exploration ID	Depth (ft.)	Sample Description	Water Content (%)	Dry Unit Wt. (pcf)	Sieve Analysis (%)			Atterberg Limits			Additional Tests
					Passing 3/4"	Passing #4	Passing #200	Liquid Limit	Plastic Limit	Plasticity Index	
B-1	2.5	YELLOWISH BROWN SANDY LEAN CLAY WITH GRAVEL (CL)	18.9	109.7							
B-1	6.0	OLIVE BROWN SANDY LEAN CLAY (CL)	19.1	108.8							TXUU: c = 2.12 ksf
B-1	11.0	REDDISH YELLOW MOTTLED SANDY LEAN CLAY (CL)	14.0	115.8							TXUU: c = 2.55 ksf
B-2	2.5	YELLOWISH BROWN LEAN CLAY (CL)	11.3	110.8							
B-2	11.0	REDDISH YELLOW CLAYSTONE	9.5	118.9							
B-3	2.5	OLIVE BROWN CLAYEY SAND (SC)						27	15	12	
B-3	11.0	YELLOWISH BROWN SANDY LEAN CLAY (CL)	26.8	94.7							TXUU: c = 1.25 ksf
B-3	16.0	OLIVE BROWN CLAYEY SAND (SC)					49	33	15	18	
B-4	6.0	OLIVE BROWN LEAN CLAY WITH SAND (CL)						43	15	28	
B-4	16.0	OLIVE BROWN CLAYEY SAND WITH GRAVEL (SC)					16				

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

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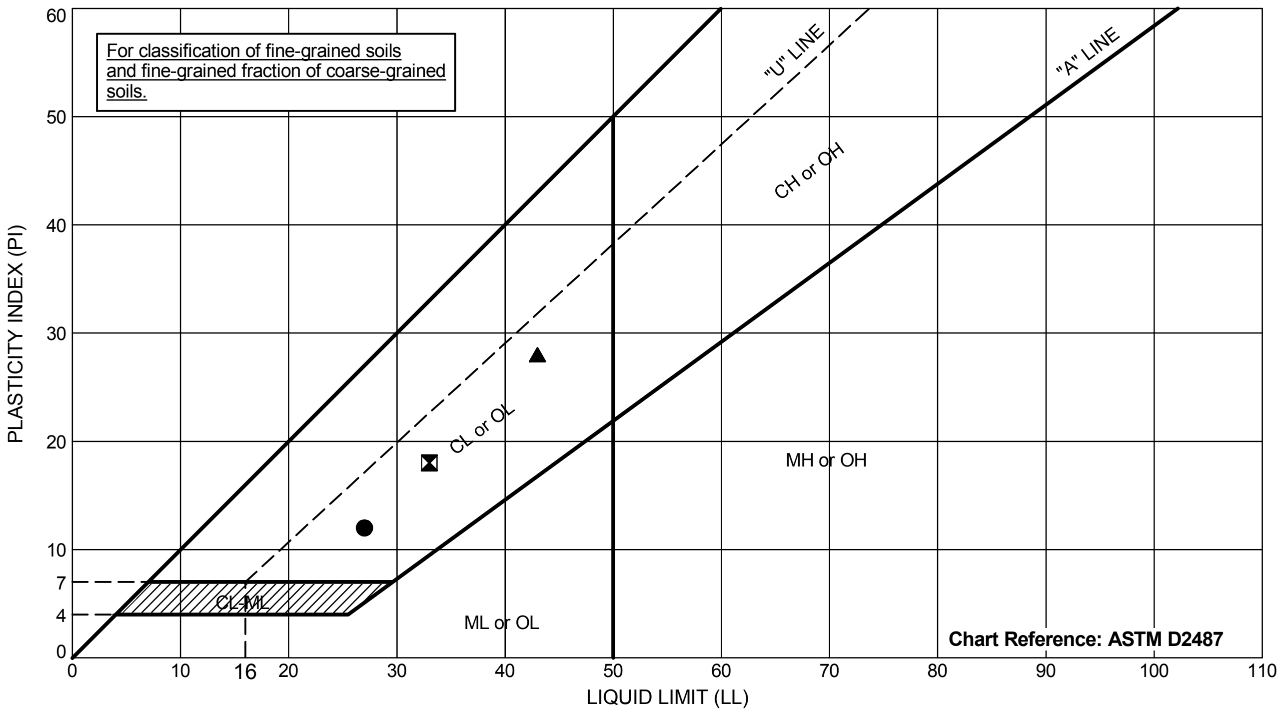
PROJECT NO.: 20181569
DRAWN BY: MAP/JDS
CHECKED BY: OK
DATE: 9/19/2017
REVISED: -

LABORATORY TEST
RESULT SUMMARY

CONTRA COSTA COMMUNITY COLLEGE
NEW SCIENCE BUILDING
2600 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA

FIGURE

B-1



	Exploration ID	Depth (ft.)	Sample Description	Passing #200	LL	PL	PI
●	B-3	2.5	OLIVE BROWN CLAYEY SAND (SC)	NM	27	15	12
⊠	B-3	16	OLIVE BROWN CLAYEY SAND (SC)	49	33	15	18
▲	B-4	6	OLIVE BROWN LEAN CLAY WITH SAND (CL)	NM	43	15	28

Testing perfomed in general accordance with ASTM D4318.
NP = Nonplastic
NM = Not Measured

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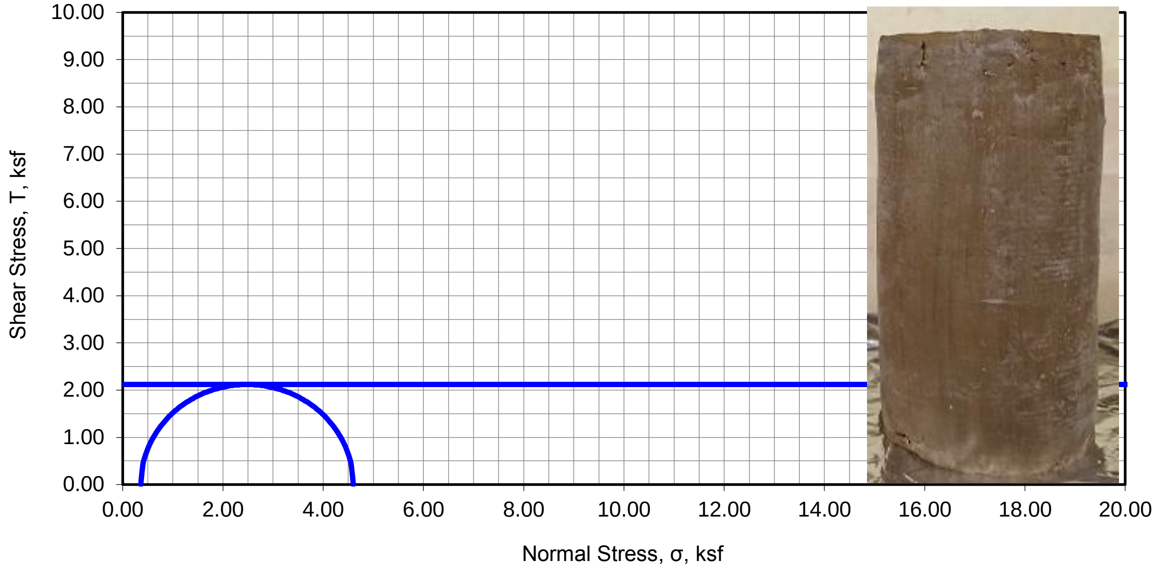
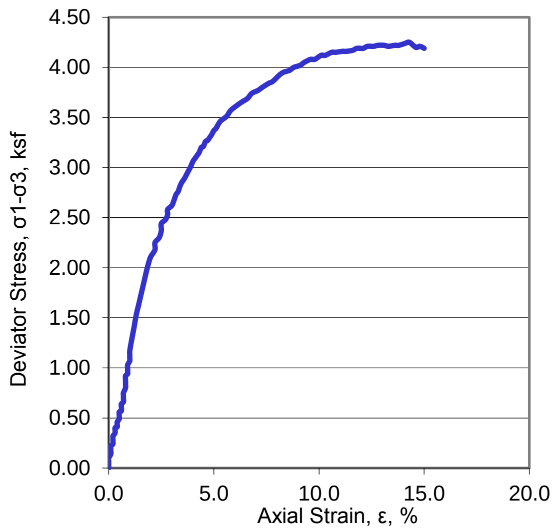
PROJECT NO.: 20181569	ATTERBERG LIMITS	FIGURE
DRAWN BY: MAP/JDS		
CHECKED BY: OK	CONTRA COSTA COMMUNITY COLLEGE NEW SCIENCE BUILDING 2600 MISSION BELL DRIVE SAN PABLO, CALIFORNIA	B-2
DATE: 9/19/2017		
REVISED: -		

Total		
c =	2.12	ksf

Specimen Shear Picture





Specimen No.			1
Initial	Diameter, in	D_O	2.39
	Height, in	H_O	5.69
	Water Content, %	ω_O	19.1
	Dry Density, lbs/ft^3	γd_o	108.8
	Saturation, %	S_O	97
	Void Ratio	e_O	0.519
Minor Principal Stress, ksf		σ_3	0.36
Maximum Deviator Stress, ksf		$(\sigma1-\sigma3)_{max}$	4.25
Time to $(\sigma_1\text{-}\sigma_3)_{max}$, min		t_f	14.33
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	4.19
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		$'\varepsilon$	1.00
Axial Strain at Failure, %		ε_f	14.33

Description of Specimen: Olive Brown Sandy Lean Clay (CL)

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

Boring:	B-1
Sample:	2C
Depth, ft:	6.0
Test Date:	9/8/17

Remarks: nm= not measured, na = not applicable

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2601 Barrington Ct, Hayward, CA 94545

Project No.:	20181569
Date:	9/11/17
Entry By:	CP
Checked By:	CP
File Name:	HL10558

TRIAXIAL COMPRESSION TEST (UU)

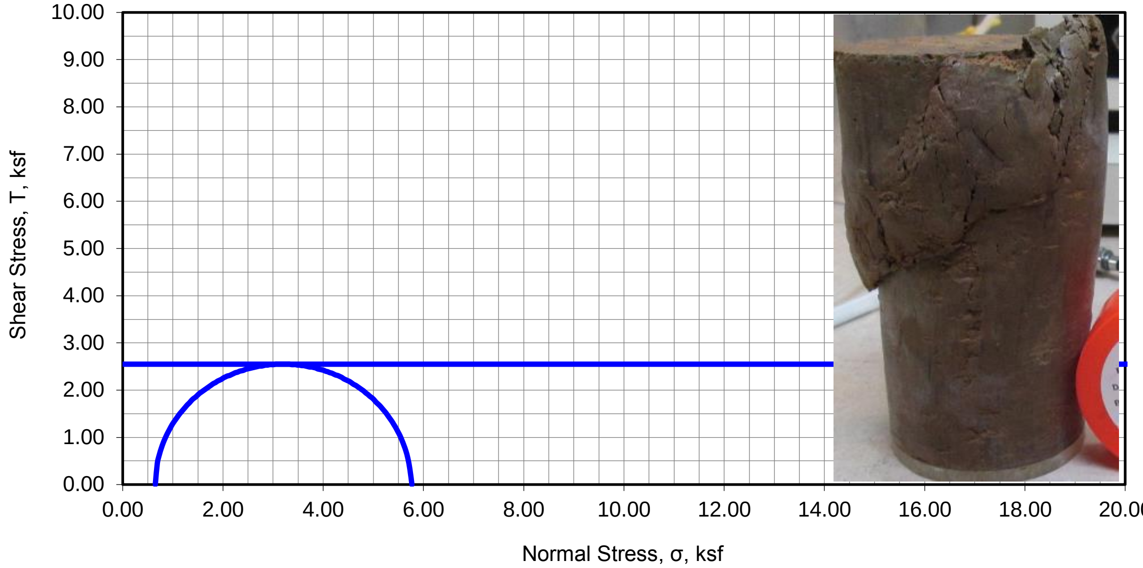
CONTRA COSTA COMMUNITY COLLEGE
NEW SCIENCE BUILDING
2600 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA

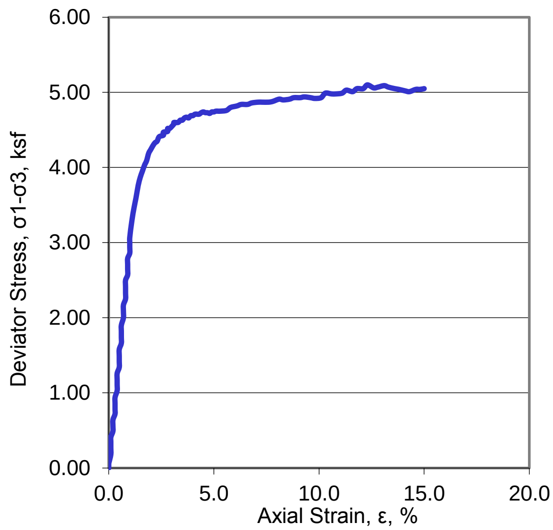
FIGURE

B-3

Total	
c = 2.55	ksf

Specimen Shear Picture





Specimen No.			1
Initial	Diameter, in	D_O	2.43
	Height, in	H_O	5.67
	Water Content, %	ω_O	14.0
	Dry Density, lbs/ft^3	γ_{d_O}	115.8
	Saturation, %	S_O	87
	Void Ratio	e_O	0.428
Minor Principal Stress, ksf		σ_3	0.65
Maximum Deviator Stress, ksf		$(\sigma1-\sigma3)_{max}$	5.10
Time to $(\sigma_1-\sigma_3)_{max}$, min		t_f	12.33
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	5.05
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		'ε	1.00
Axial Strain at Failure, %		ε_f	12.33

Description of Specimen: Brown Lean Clay with Sand (CL)

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

Boring:	B-1
Sample:	3C
Depth, ft:	11.0
Test Date:	9/8/17

Remarks: nm= not measured, na = not applicable

KLEINFELDER
Bright People. Right Solutions.

2601 Barrington Ct, Hayward, CA 94545

Project No.:	20181569
Date:	9/11/17
Entry By:	CP
Checked By:	CP
File Name:	HL10558

TRIAXIAL COMPRESSION TEST (UU)

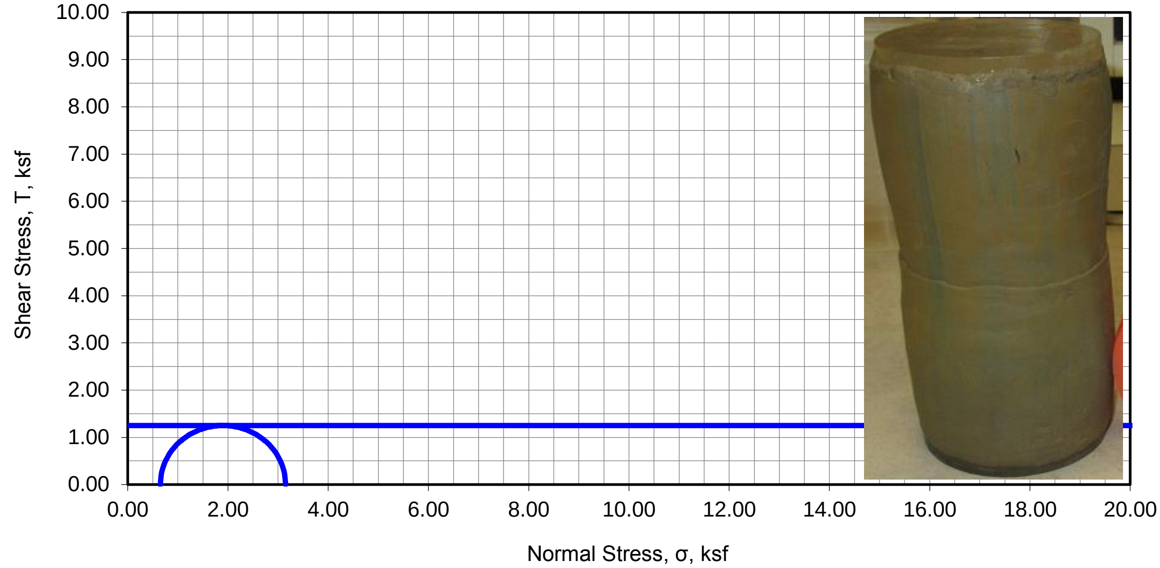
CONTRA COSTA COMMUNITY COLLEGE
NEW SCIENCE BUILDING
2600 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA

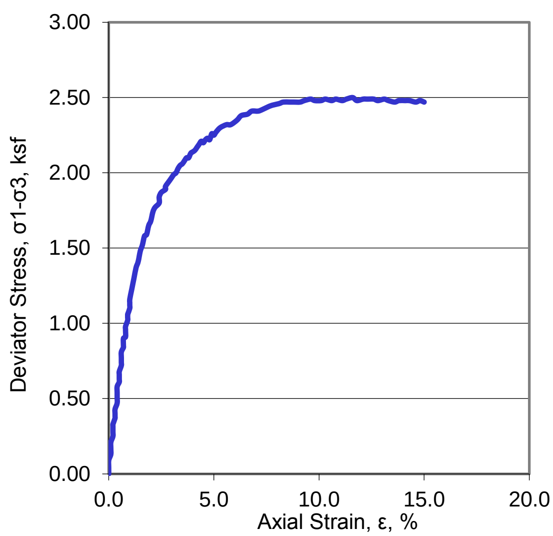
FIGURE

B-4

Total	
c = 1.25	ksf

Specimen Shear Picture





Specimen No.			1
Initial	Diameter, in	D_O	2.43
	Height, in	H_O	5.70
	Water Content, %	ω_O	26.8
	Dry Density, lbs/ft^3	γd_o	94.7
	Saturation, %	S_O	95
	Void Ratio	e_O	0.747
Minor Principal Stress, ksf		σ_3	0.65
Maximum Deviator Stress, ksf		$(\sigma1-\sigma3)_{max}$	2.50
Time to $(\sigma_1-\sigma_3)_{max}$, min		t_f	11.58
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	2.47
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		'ε	1.00
Axial Strain at Failure, %		ε_f	11.58

Description of Specimen: Brown Lean Clay (CL)

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

Boring:	B-3
Sample:	3C
Depth, ft:	11.0
Test Date:	9/8/17

Remarks: nm= not measured, na = not applicable

KLEINFELDER
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2601 Barrington Ct, Hayward, CA 94545

Project No.:	20181569
Date:	9/11/17
Entry By:	CP
Checked By:	CP
File Name:	HL10558

TRIAXIAL COMPRESSION TEST (UU)

CONTRA COSTA COMMUNITY COLLEGE
NEW SCIENCE BUILDING
2600 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA

FIGURE

B-5

APPENDIX C
CORROSIVITY TEST RESULTS

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client: Kleinfelder
Client's Project No.: 20181569
Client's Project Name: Contra Costa College-New Allied Science Bldg. (C-4016)
Date Sampled: 08/11 & 18/17
Date Received: 8-Sep-2017
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 21-Sep-2017

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Resistivity (As Received) (ohms-cm)	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
1709047-001	B-3 2C @ 6'	+440	7.86	720	1,100	N.D.	N.D.	N.D.

Method:	ASTM D1498	ASTM D4972	ASTM G57	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	-	-	50	15	15
Date Analyzed:	14-Sep-2017	14-Sep-2017	13-Sep-2017	13-Sep-2017	20-Sep-2017	14-Sep-2017	14-Sep-2017

Cheryl McMillen
Laboratory Director

* Results Reported on "As Received" Basis
N.D. - None Detected

Quality Control Summary - All laboratory quality control parameters were found to be within established limits

APPENDIX D
GBA INFORMATION SHEET

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

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.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation*.

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

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.***

GBA GEOPROFESSIONAL BUSINESS ASSOCIATION

Telephone: 301/565-2733
e-mail: info@geoprofessional.org www.geoprofessional.org

GEOLOGIC AND SEISMIC HAZARDS ASSESSMENT REPORT
C-4016 NEW SCIENCE BUILDING
CONTRA COSTA COLLEGE
2600 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA

PROJECT No.: 20181569.001A

OCTOBER 20, 2017

October 20, 2017
Project No.: 20181569.001A

Contra Costa Community College District (District)
2600 Mission Bell Drive
San Pablo, California 94806
C/O Mr. Ron Johnson
ronj@csipm.com

SUBJECT: Geologic and Seismic Hazards Assessment Report
C-4016 New Science Building
Contra Costa College
2600 Mission Bell Drive, San Pablo, California

Dear Mr. Johnson:

Kleinfelder is pleased to present this geologic and seismic hazards assessment report for the planned New Science building at Contra Costa College in San Pablo, California. Figure 1 – Site Vicinity Map and Figure 2 – Site Plan and Geology Map show the approximate location of the planned project within the college campus. The project site is currently occupied by the Liberal Arts and Health Sciences buildings, which are abandoned and earmarked for demolition.

This report is intended to identify and characterize potential geologic and seismic hazards at the project site and adjacent area of the campus in order to satisfy and comply with Note 48 guidelines and checklist items prepared by the California Geological Survey (CGS) for public school projects. The CGS reviews geologic and seismic hazard assessment reports for the Division of the State Architect (DSA). Conclusions pertaining to the potential impacts of these geologic hazards on the planned improvements are provided in the report.

The accompanying report summarizes the results of our field reconnaissance, data research and review, and engineering geologic interpretations, conclusions, and recommendations. In addition, this report describes the geologic setting, faulting, seismicity, and potential geologic and seismic hazards that could impact the planned project. The primary geologic/seismic hazard considerations performed as part of this assessment include fault-related ground surface rupture, seismically-induced ground failures (liquefaction, lateral spreading, and dynamic compaction), expansive soils, landslides, flooding including from heavy rainstorms, tsunamis and seiches hazards, naturally-occurring asbestos, soil corrosion, and radon gas. Conclusions pertaining to the potential impacts of these geologic and seismic hazards on the planned development are provided in the report.

A site-specific Seismic Hazards Analysis has been prepared for this site as part of our scope and is attached hereto in Appendix E. Kleinfelder (2017) has recently prepared a site-specific geotechnical engineering study for the subject project, which was issued under a separate cover and which we list in the References Section of this report.

Based on the results of our assessment, it is our opinion that, from an engineering geologic and geotechnical viewpoint, the subject site is considered suitable for the planned project and associated improvements provided that our conclusions and recommendations presented herein and in our concurrent geotechnical engineering report are adhered to and incorporated into the design and construction of the planned new science building. The primary geological and seismic issues of concern are:

1. The project site is situated within the limits of the Alquist-Priolo Earthquake Fault Zone (AP Zone) associated with the active Hayward fault;
2. The proximity of the planned project to the main creeping trace of the Hayward fault;
3. Anticipated strong to violent ground shaking as a result of future seismic events along the Hayward fault and one of the active earthquake faults within the region;
4. The presence of low to highly expansive soils;
5. The presence of undocumented fill; and
6. The potential for highly corrosive soils.

Utilizing subsurface trenching techniques, Kleinfelder and other consultants have evaluated the presence and activity of any secondary sympathetic fault splays associated with the active Hayward fault within the vicinity of the planned new science building. The geologic trenches were excavated and logged in the general vicinity of the project area between 1972 through 2008 as shown on Figure 2. Kleinfelder evaluated these geologic trenches and existing earthquake fault mapping in our report titled *Master Plan Seismic Study, Contra Costa College Campus, San Pablo, California,* dated July 15, 2009 (Project Number 80412/Report/PLE9R266). In this report, three colored zones were delineated across the campus as follows:

- Red – indicating the presence of active faulting and the limits of a setback zone excluding the construction of structures intended for human habitation and occupancy;
- Yellow – areas that have yet to be cleared of secondary fault traces and that would require additional exploration to assess faulting; and
- Green – habitable zones where it has been demonstrated that no active faults exist and no additional studies would be needed to clear the area for development including structures intended for human habitation and occupancy.

The report concluded, based on existing available data, that the Liberal Arts and Health Sciences buildings were free of active fault traces and the surrounding trenches provided enough coverage and "shadowing" for possible fault traces in a northwestwardly trend. Therefore, the buildings were placed within the green-zone. However, the report concluded that there should be a 50-foot setback zone established on the northeast side of the Liberal Arts building from the western most fault observed in the trenches excavated by Harding-Lawson and Associates in 1972/1973 for the then proposed Physical Sciences building addition. Based on the above information, the currently planned location of the New Science building will be situated within the habitable zone colored green on the campus-wide seismic and fault setback map, which we utilize as base for Figure 2 of this report.

The colored zones were further evaluated and adjusted by Kleinfelder in 2011 per the recommendations of CGS. Our conclusions and recommendations were provided in our letter report titled *Re-Assessment of Fault-Related Exclusionary Boundaries Pertaining to Habitable Structures for the Campus Center Project/New Student Activities Building Proposed within the*

Contra Costa College Campus in San Pablo, California, dated March 24, 2011 (Project Number 112252/PWPORTABLES/PLE11L027). The green-zone was further adjusted to the southwest near the Liberal Arts and Health Sciences buildings.

As noted above, our concurrent geotechnical engineering study for the subject project (Kleinfelder, 2017) provided conclusions and recommendations pertaining to grading, drainage, foundation design, and earthwork recommendations. Seismic design recommendations were presented in the site-specific ground motions seismic analysis report attached hereto in Appendix E. The geotechnical report also presented recommendations to mitigate potential fill settlement any potentially adverse geologic conditions associated with soil expansion and corrosion. We understand that the existing Liberal Arts building has sustained some distress, which may be related to the presence of undocumented fill or the magnitude of grading and type of foundations utilized.

This assessment was performed based on conclusions developed from the review of published studies and maps, nearby site-specific evaluations, a site reconnaissance visit by our project Engineering Geologist, results of geologic trenching studies referenced, review of subsurface information obtained from our concurrent preliminary geotechnical engineering study, and our experience with this college campus and similar projects.

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

Sincerely,

KLEINFELDER, INC.

Omar Khan
Project Geologist

Sadek M. Derrega, PG, CEG #2175
Senior Principal Engineering Geologist

OK/SMD/jmk

GEOLOGIC AND SEISMIC HAZARDS ASSESSMENT REPORT
C-4016 NEW SCIENCE BUILDING
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APPENDICES

1 INTRODUCTION

This report presents the results of Kleinfelder's geologic and seismic hazards assessment for the planned new science building at Contra Costa College in San Pablo, California. The approximate location of the school campus is shown on the Site Vicinity Map (Figure 1) and the approximate limits of the planned new science center are shown on the Site Plan and Geology Map (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.

Kleinfelder has vast experience at the campus. That experience, coupled with our concurrent geotechnical engineering study for the planned New Science building were relied on to characterize the subsurface conditions. For the concurrent geotechnical engineering study we drilled four soil borings at the planned New Science building site on August 11 and 18, 2017 to a depth of approximately 31 to 41½ feet deep. The approximate locations of the borings are shown on Figure 2. The subsurface conditions revealed by the borings drilled by Kleinfelder as part of the concurrent geotechnical study and our previous experience at the campus were utilized to characterize the potential for and magnitude of liquefaction at the project site and to generate engineering recommendations pertaining to grading, drainage, foundation design, and construction considerations for the planned new science center.

1.1 SITE LOCATION AND DESCRIPTION

The western part of the campus is located mostly on a level alluvial plain west of Rheem Creek. The eastern portion of the campus slopes upward to the northeast. The active Hayward fault, which crosses the campus, approximately separates the flat lying portion of the campus with the elevated/hillside portion of the campus. Rheem Creek flows through the campus in a northwesterly direction generally parallel to the base of the hillside. Most of the academic buildings on the campus are located on the hillside portion of the campus, while the flat lying portion of the campus contains mostly the athletic buildings and facilities. The ground surface elevation at the campus ranges from about 50 feet above mean sea level along the southwestern margin of the campus to about 130 feet in the northeast corner along Campus Drive.

We understand that the campus plans to demolish the existing abandoned Liberal Arts and Health Sciences buildings and construct a new 3-story building with an approximate footprint of up to about 20,000 square feet. It is anticipated that cuts up to about 20 to 40 fee can be anticipated to achieve finished grades. This could change since the project is currently in conceptual design phase. Structural loads are assumed to be less than 300 kips for column loads. It is anticipated that the structure will be founded on shallow spread footings. The final layout of the building has not been determined at this time.

The existing buildings are currently situated northeast of Rheem Creek along the elevated portion of the campus. As shown of Figure 2, the buildings are situated in between the Physical Sciences building (located to the northeast), Administrative and Applied Arts building (located to the southeast), and Library and Learning Resource Center (located to the west). In between the Library and Learning Resource Center and Liberal Arts and Health Sciences buildings there is an open, grass covered courtyard area gently sloping to the southwest. A fire access road runs parallel with the Liberal Arts and Health Sciences buildings along the northeastern end of the buildings, situated at a higher topographic level than the grass covered open area. The project site generally slopes to the southwest. Sloped walkways and stairways are located around the buildings.

According to the U.S. Geological Survey (USGS, 1993) 7½-Minute Richmond Topographic Quadrangle map, the existing ground elevation at the site ranges between about 70 and 100 feet above mean sea level. The coordinates at the center of the planned new science center location are approximately:

Latitude: 37.969664° N

Longitude: -122.336584° W

1.2 PURPOSE AND SCOPE OF SERVICES

The purpose of our geologic and seismic hazards assessment is to identify potential geologic and seismic hazards and conditions that could adversely impact development of the proposed new science center or restrict its overall use. Our scope of services included a site reconnaissance by a Certified Engineering Geologist (CEG), review of readily available published geotechnical data and unpublished site-specific geologic and seismic evaluations, and the subsurface exploration and laboratory data obtained during our concurrent geotechnical engineering investigation. The objectives of this report 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);
- CGS Note 48 (Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings); and
- DSA IR A-4.13 (Geohazard Report Requirements: 2013 & 2016 CBC).

Specifically, our scope of services included the following:

- Review of the regional and local geologic and seismic setting of the site and surrounding area, including research and review of available geologic/seismic reports published by the USGS and the CGS, and a review of available geologic and geotechnical site-specific studies performed by Kleinfelder.
- Performing a reconnaissance of the site and adjacent areas by our CEG.
- Reviewing subsurface geotechnical soil borings and geologic trench data including depth to groundwater, from the published literature and site-specific previous geotechnical investigations.
- Preparing this Geologic and Seismic Hazards Assessment report for the site that covers the checklist items in CGS Note 48, and which presents the conclusions and results of our study. The report may include the following:

  a) A site vicinity map;
  b) A site plan and geology map
  c) An area geologic map;
  d) A regional geology map;
  e) A geologic cross section(s);
  f) Regional fault and historic seismicity map;

g) A description of regional geology, area geology, and nearby seismic sources (faults);
h) Discussion of the site location as it pertains to the Alquist-Priolo Earthquake Fault Zone pertaining to liquefaction and slope stability;
i) A description of the site's seismicity;
j) Conducting a site specific ground motion analysis; and
k) Conclusions regarding:
    1. Fault-related ground surface rupture;
    2. Seismically-induced ground failures including liquefaction, lateral spreading, and dynamic compaction;
    3. Expansive soils, collapsible, peaty, or compressible soils;
    4. Presence of undocumented fill soils;
    5. Slope stability and landslides (seismically-induced or otherwise);
    6. Flooding, tsunami-related hazard, and seiches;
    7. Naturally-occurring asbestos;
    8. Radon gas; and
    9. Soil corrosion.

Our current scope excluded an assessment of pipeline locations within 1,500 feet of the project site. Our evaluation also specifically excluded the assessment of environmental spills and hazardous substances at the site.

2 GEOLOGIC SETTING

2.1 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 conditions of the site are depicted 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.

2.2 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. Based on information obtained from the extensive fieldwork at the campus during previous fault trench studies, we mapped the level areas of the campus as being underlain by Holocene basin deposits and Holocene fine- to coarse-grained channel deposits near Rheem Creek. The Holocene deposits 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.

According to Graymer (2000), the project site is underlain by late Miocene Orinda formation (map symbol Tor), as shown on Figure 4, Area Geology Map. The Orinda formation is described by Graymer (2000) as distinctly to indistinctly bedded, non-marine, pebble to boulder conglomerate, conglomeratic sandstone, coarse- to medium-grained lithic sandstone, and green and red siltstone and mudstone. Conglomerate clasts are subangular to well rounded, and contain a high percentage of detritus derived from the Franciscan complex.

2.3 SITE RECONNAISSANCE

A Certified Engineering Geologist with our firm performed a site reconnaissance of the project area during middle October 2017 and observed site conditions. The site and surrounding areas are occupied by structures and appear to have been developed completely as part of the college center as far back as the early 1990s on available aerial photographs. The Rheem Creek channel appears to have been shifted southwestward slightly between 1939 and 1993. The area remained essentially unaltered until the recent college center renovations.

2.4 SUBSURFACE CONDITIONS

The subsurface conditions described herein are based on the soil and groundwater conditions encountered during the current and previous geologic and geotechnical investigations in the vicinity of the site area. The project site subsurface consists mostly of fill and native soils underlain by claystone. The fill was encountered in borings B-3 and B-4 measuring between about 8 to 13 feet and generally consisting of very stiff to hard sandy clays. The native soil consisted stiff sandy clays interbedded with clayey sands and gravels, which in turn were underlain by weathered claystone. The claystone was generally weak to strong, moderately to highly weathered, and highly fractured.

Groundwater was not observed and encountered in our current borings. However, groundwater was observed in our previous borings and fault trenches at depths of about 9 to 23 feet below the ground surface. 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 our experience at the campus. More detailed descriptions of the subsurface conditions encountered are presented in the Boring Logs on Figures A-4 and A-7 in Appendix A, and on the Boring Logs, and fault trenches from our previous investigations presented in Appendix C.

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.

3 FAULTING AND SEISMICITY

The faulting and seismicity of the site and surrounding areas, including a site-specific ground motion analysis is discussed in Appendix E of this report.

4 CONCLUSIONS - GEOLOGIC AND SEISMIC HAZARDS

Discussion and conclusions regarding specific geologic hazards, which could impact the site, are included below. The hazards considered include: fault-related ground surface rupture, seismically-induced ground failures (liquefaction, lateral spreading, and dynamic compaction/seismic settlement), expansive soils, landslides, tsunami/seiches, flooding, naturally-occurring asbestos, soil corrosion, radon gas, and existing fill.

4.1 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 our project CEG. Therefore, it is our opinion that the potential for continued surface creep along the main fault trace located to the west/southwest of the project site is high. 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 campus cannot be ruled out. However, based on historic performance, the knowledge that the main trace is more than more than 400 feet to the southwest from the planned project site, and the knowledge that the Hayward fault ground surface rupture is generally contained along the trace itself and generally not extending for hundreds of feet laterally, we conclude that the potential for fault-related ground surface rupture to impact the planned project is considered low.

4.2 SEISMICALLY-INDUCED GROUND FAILURE

4.2.1 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 Hoose, 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 investigations at the campus, the project site subsurface consists mostly of interbedded layers of firm to hard fine-grained clayey soils underlain by bedrock. As a result, liquefaction potential at the site is considered minimal due to the soil types encountered.

4.2.2 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 since our borings indicate the fill to be comprised mostly of lean to fat clay soils with sand. For this reason we conclude that the potential for shaking related random ground cracking to affect the site and surrounding areas is low.

Furthermore, recommendations have been provided in our concurrent geotechnical engineering investigation (Kleinfelder, 2017) to address the presence of the reported undocumented fill.

4.3 EXPANSIVE SOILS

Based on the results of our concurrent field investigation and laboratory testing program, near-surface soils located within the building site are low to highly expansive. Pertinent mitigation measures addressing the potential presence of expansive soils at the site are presented in our concurrent geotechnical investigation report (Kleinfelder, 2017) for the site.

4.4 EXISTING FILL

Fill measuring between 8 to 13 feet was encountered in our borings B-3 and B-4 which was comprised of interbedded very stiff to hard sandy clays. Our concurrent geotechnical study evaluated the presence of the noted undocumented fill and presented recommendations to mitigate.

4.5 LANDSLIDES

No landslides are mapped in the project area nor did we observe any slope creep or cracks. Therefore, it is our opinion that the potential for seismically induced (or otherwise) landslides and slope failure to occur at the proposed site is considered low.

Rheem Creek is located approximately 200 feet southwest 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 top of banks. The creek banks do appear to exhibit evidence of soil creep and it is our opinion that soil creep will continue along these banks and could affect any improvements within 10 feet of the top of banks if not mitigated.

4.6 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 a mile southeast of the San Pablo Bay at an estimated elevation of about 80 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.

The East Bay Municipal Utility District North Reservoir, a ground level covered structure, is located approximately ½-mile northeast of the project site near Highland Elementary School. The San Pablo Reservoir/Dam is location approximately 4½ miles southeast of the project site. If these reservoirs were to fail during a seismic event, the project site would flood.

With respect to the 100-year storm events, the Federal Emergency Management Agency's (FEMA, 2009) Flood Insurance Rate Map, Community-Panel Number 06013C0227G, effective date September 30, 2015, 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.**

4.7 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 1½ miles to the southeast 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.

4.8 SOIL CORROSION

Kleinfelder has completed laboratory testing to provide data regarding corrosivity of onsite soils. The testing was performed by a State of California certified laboratory, CERCO Analytical of Concord, California on a selected sample of the near-surface soils. Our scope of services does not include corrosion engineering and, therefore, a detailed analysis of the corrosion test results is not included in this report. A qualified corrosion engineer should be retained to review the test results and design protective systems that may be required. Kleinfelder may be able to provide those services.

Laboratory chloride concentration, sulfate concentration, sulfide concentration, pH, oxidation reduction potential, and electrical resistivity tests were performed on the near surface soil sample. The results of the tests are presented in Appendix C and are summarized and are summarized below in Table 4.8-1. 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 4.8-1
Chemistry Laboratory Test Results

Boring	Depth, feet	Resistivity, ohm-cm		pH	Oxidation Reduction Potential, mV	Water-Soluble Ion Concentration, ppm		
		100% Saturated	In-Situ Moisture			Chloride	Sulfide	Sulfate
B-3	6	1,100	720	7.86	+440	N.D.	N.D.*	N.D.

*N.D. - None Detected

Ferrous metal and concrete elements in contact with soil, whether part of a foundation or part of the supported structure, are subject to degradation due to corrosion or chemical attack. Therefore, buried ferrous metal and concrete elements should be designed to resist corrosion and degradation based on accepted practices.

Based on the “10-point” method developed by the American Water Works Association (AWWA) in standard AWWA C105/A21.5, the soils at the site are corrosive to buried ferrous metal piping, cast iron pipes, or other objects made of these materials. We recommend that a corrosion engineer be consulted to recommend appropriate protective measures.

The degradation of concrete or cement grout can be caused by chemical agents in the soil or groundwater that react with concrete to either dissolve the cement paste or precipitate larger compounds within the concrete, causing cracking and flaking. The concentration of water-soluble sulfates in the soils is a good indicator of the potential for chemical attack of concrete or cement grout. The American Concrete Institute (ACI) in their publication “Guide to Durable Concrete” (ACI 201.2R-08) provides guidelines for this assessment. The sulfate tests indicated the sample had no sulfate detected. The results of sulfate test indicate the potential for deterioration of concrete is mild, no special requirements should be necessary for the concrete mix.

Concrete and the reinforcing steel within it are at risk of corrosion when exposed to water-soluble chloride in the soil or groundwater. Chloride tests indicated the sample had no chloride detected. The project structural engineer should review this data to determine if remedial measures are necessary for the concrete reinforcing steel.

4.9 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 Public Health services (2016) performed 52 tests within Zip Code 94806 (last updated on February 2016) where the school campus is located. Of the 52 tests, none reported a minimum of four (4) picocuries per liter (pCi/L). The maximum results reported was 2.3 pCi/L.

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 Public Health – Indoor Radon Program:
https://www.cdph.ca.gov/Programs/CEH/DRSEM/Pages/EMB/Radon/Radon.aspx

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

U.S. EPA: https://www.epa.gov/radon

4.10 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.

4.11 BEDROCK RIPPABILITY

Excavations can be performed by conventional earthmoving equipment. However, during site grading, foundation and utility trench excavation, localized zones of strong to very strong bedrock, resulting in hard digging, may be encountered. Contractor(s) and subcontractors should expect hard drilling, digging, and excavating and should be prepared to use heavy ripping and excavating equipment, including hydraulic hammers and/or hoe-ram equipment.

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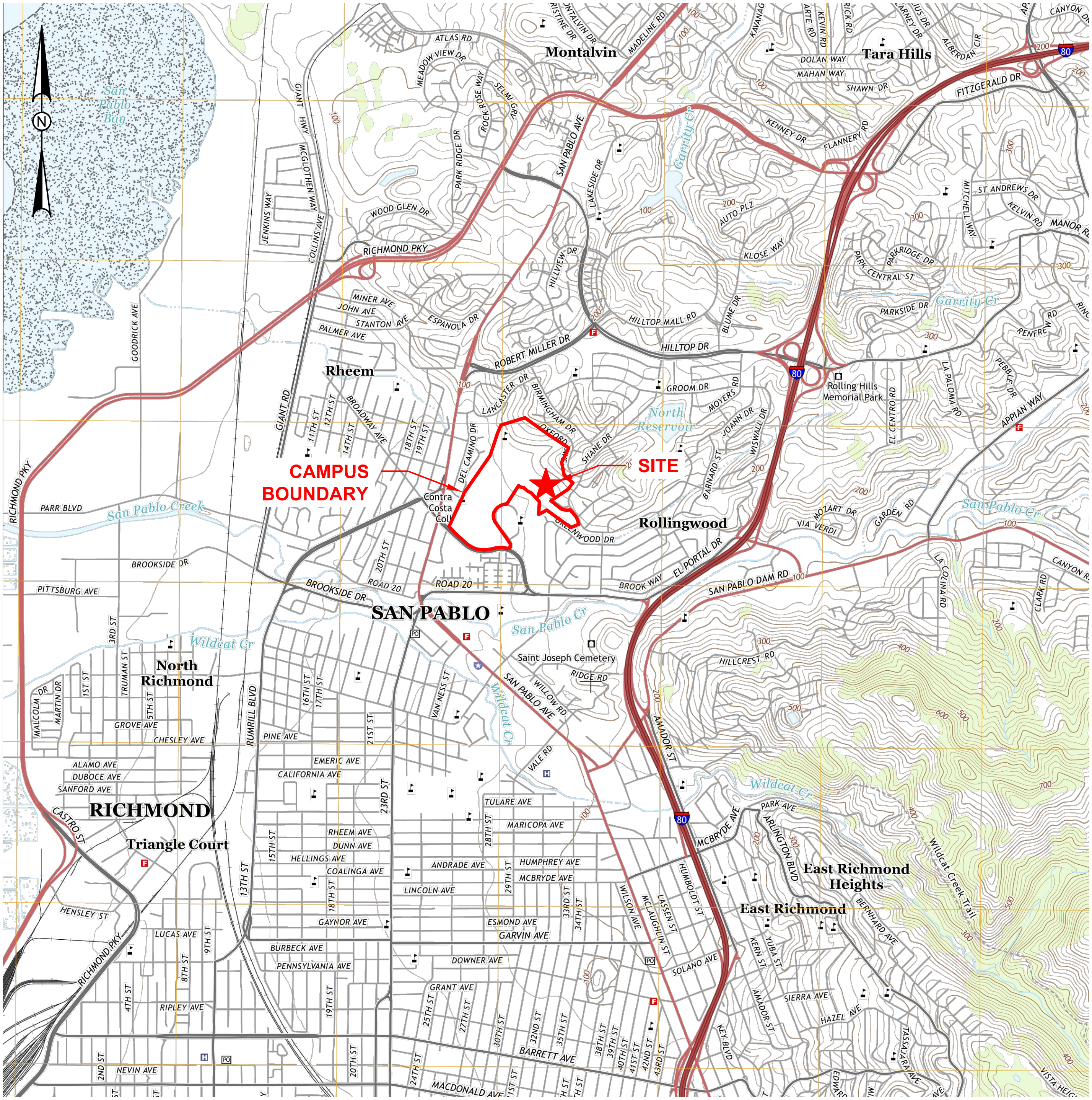
FIGURES

LIST OF ATTACHMENTS

The following figures are attached and complete this appendix.



REFERENCE:

VICINITY MAP CREATED FROM DATA COMPILED BY USGS US TOPO RICHMOND, CA. QUADRANGLE 7.5-MINUTE, 2015

0 3000 6000

SCALE: 1" = 3000' SCALE IN FEET

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PROJECT NO.	20181569
DRAWN BY:	JDS
CHECKED BY:	OK
DATE:	09/07/2017
REVISED:	

SITE VICINITY MAP

CONTRA COSTA COLLEGE
NEW SCIENCE BUILDING
2600 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA

FIGURE
1

PLOTTED: 9/7/2017 4:45 PM BY: jeff sala

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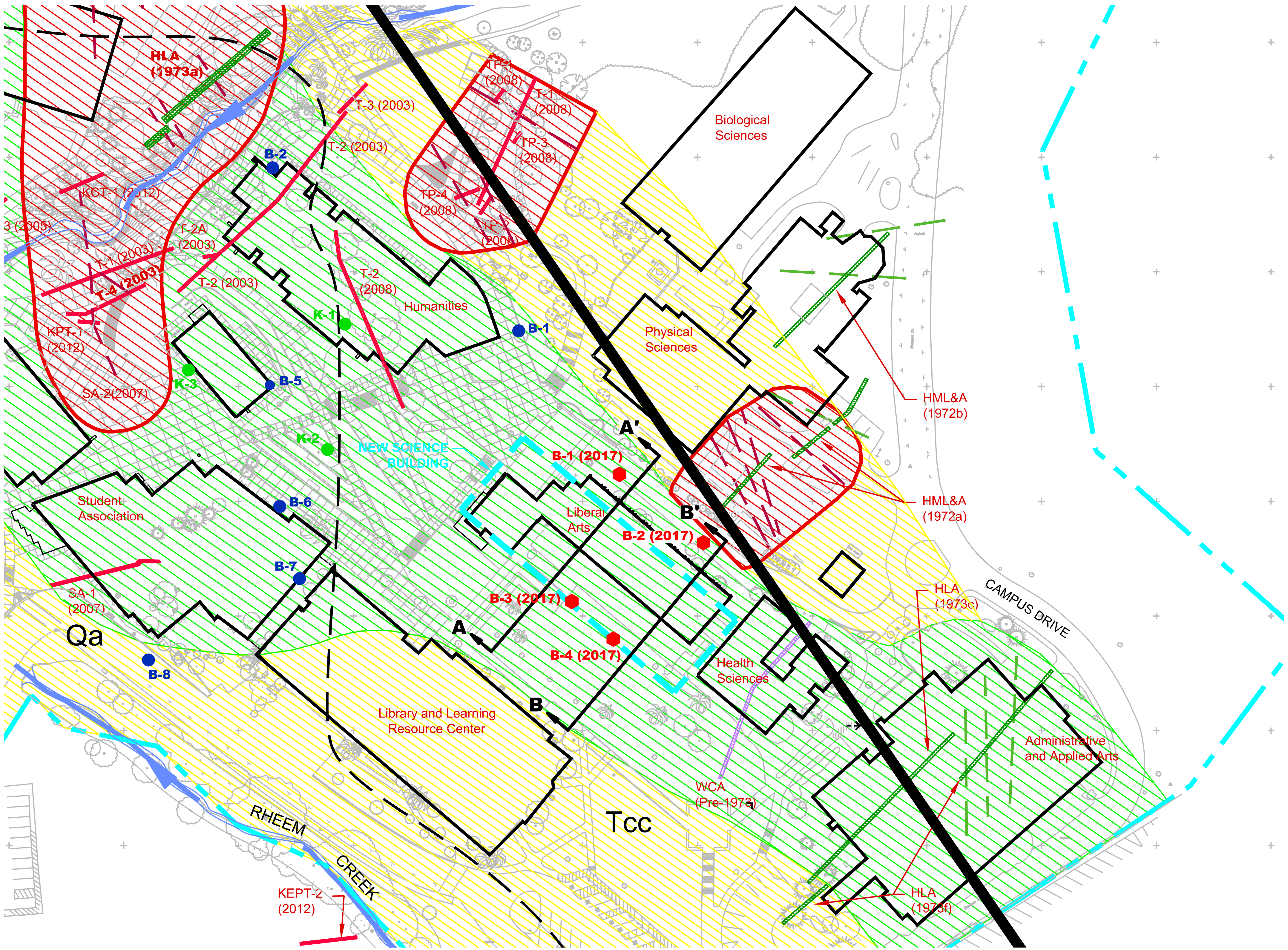
PLOTTED: 10/20/2017 10:48 AM BY: gabe gomez



LEGEND

CAMPUS BOUNDARY

APPROXIMATE GEOLOGIC CONTACT (BASED ON GRAYMER, 2000)

EASTERN LIMIT OF APEFZ - by California Geological Survey (CGS)

APPROXIMATE CREEK LOCATION

APPROXIMATE INACTIVE FAULT LOCATION

APPROXIMATE POTENTIALLY ACTIVE FAULT LOCATION

APPROXIMATE FAULT TRENCH LOCATION (report date in parentheses):

HARDING-LAWSON ASSOCIATES (HLA) or (HML&A)

WOODWARD-CLYDE & ASSOCIATES (WCA)

KLEINFELDER, INC.

A' A — GEOLOGIC CROSS SECTION LINE (See Figure 5 for cross section)

B-4 (2017) — SOIL BORING (BY KLEINFELDER, 2017)

B-8 — SOIL BORING (BY KLEINFELDER, 2010)

K-3 — SOIL BORING (BY KLEINFELDER, 2004)

Qa — QUATERNARY ALLUVIUM

Tcc — GARRITY MEMBER, CONTRA COSTA GROUP (TERTIARY)

EXCLUSION ZONE

AREAS NOT CLEARED OF SECONDARY FAULT TRACES (further investigation required)

HABITABLE STRUCTURE ZONE

Reference: Kleinfelder, 2017, Proposed Adjustment of Habitable Zone Boundaries within the Campus of Contra Costa Community College, San Pablo, California, dated August 24, 2017

N

100 50 0 100

APPROXIMATE SCALE (feet)

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DRAWN BY	JDS
CHECKED BY	OK
DATE:	10/04/2017
REVISED:	

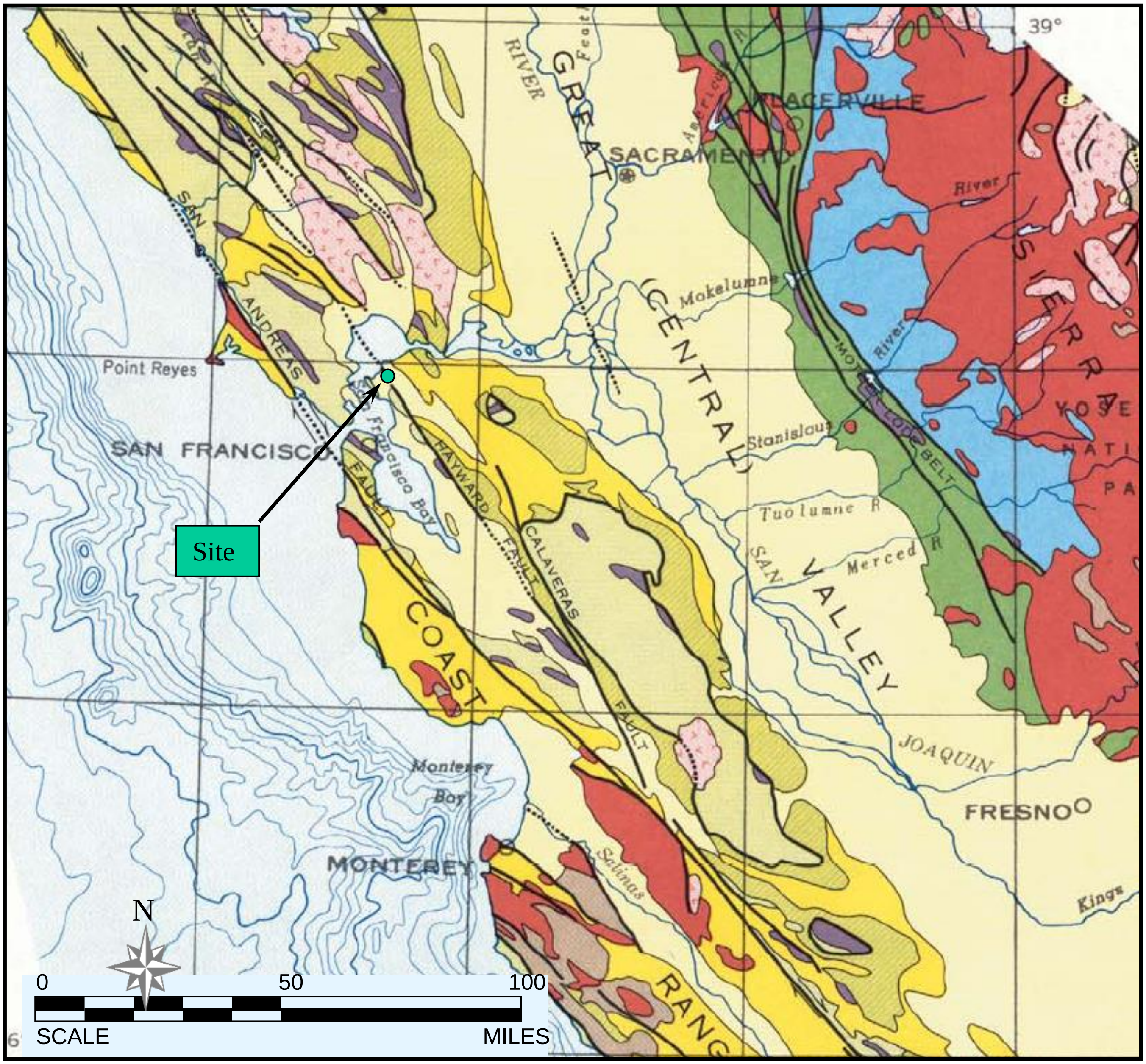
SITE PLAN AND GEOLOGY MAP

CONTRA COSTA COLLEGE
NEW SCIENCE BUILDING
2600 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA

FIGURE 2

CAD FILE: W:\2018\20181569\plans\SITE PLAN.dwg LAYOUT: F2

PLOTTED: 07 SEPT 2017,



EXPLANATION
of units in SF Bay Area

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

Map source: California Geological Survey, 2002, Geologic Map of California, California Department of Conservation.

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PROJECT NO.	20164720
DRAWN:	SEPT 2017
DRAWN BY:	P.H./J.S.
CHECKED BY:	O.K.
FILE NAME:	REGIONAL GEOLOGIC MAP.ppt

REGIONAL GEOLOGIC MAP

CONTRA COSTA COLLEGE
NEW SCIENCE BUILDING
2600 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA

FIGURE
3

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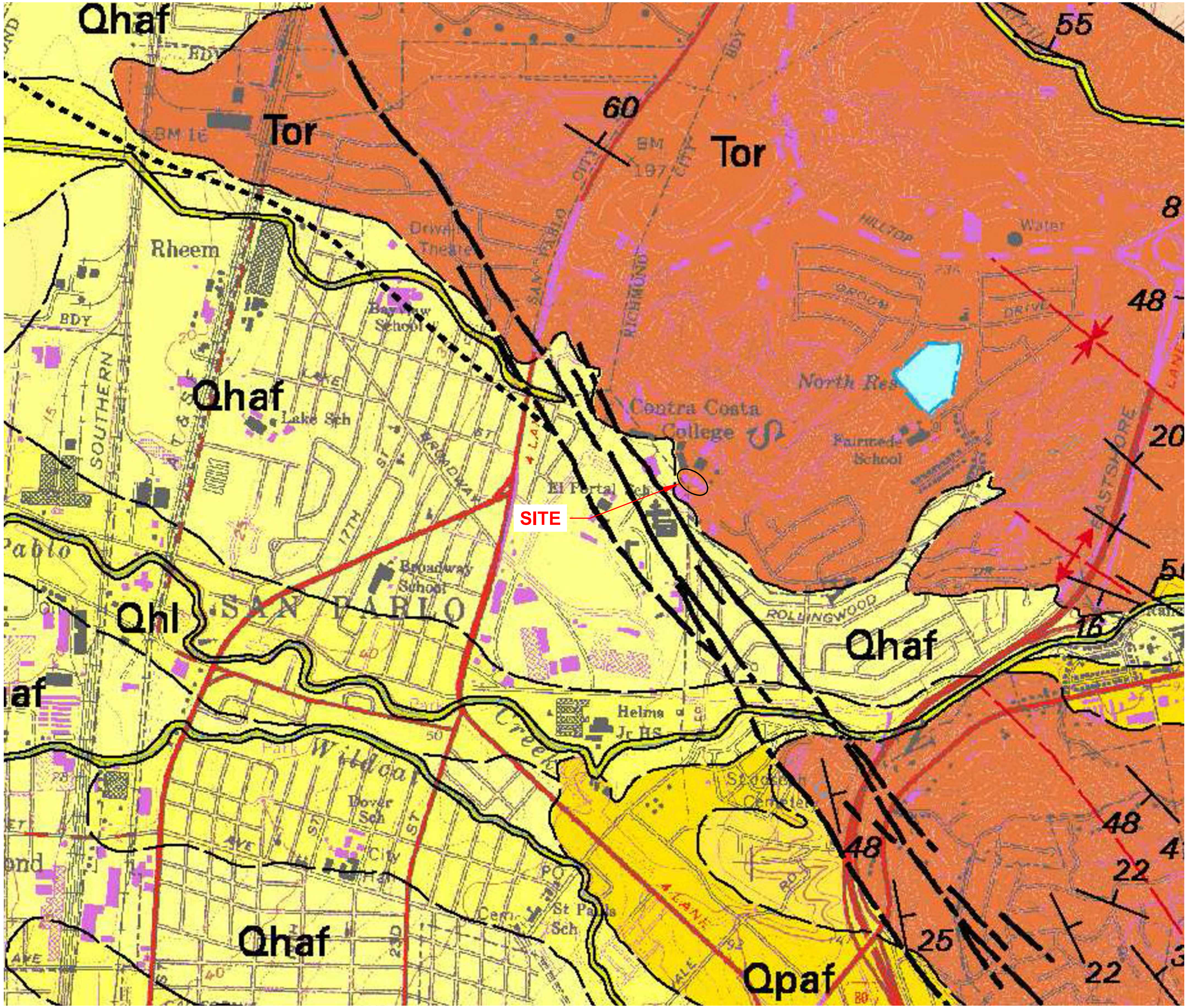

MAP UNITS

Qhaf	ALLUVIAL FAN AND FLUVIAL DEPOSITS (HOLOCENE)
Qhl	NATURAL LEVEE DEPOSITS (HOLOCENE)
Qpaf	ALLUVIAL FAN AND FLUVIAL DEPOSITS (PLEISTOCENE)
Tor	ORINDA FORMATION (LATE MIOCENE)

Contact-- Depositional or intrusive contact, dashed where approximately located, dotted where concealed

Fault-- Dashed where approximately located, small dashes where inferred, dotted where concealed, queried where location is uncertain.

Reverse or thrust fault-- Dashed where approximately located, dotted where concealed

Normal fault-- Dashed where approximately located, dotted where concealed

Anticline-- Shows fold axis, dashed where approximately located, dotted where concealed

Syncline

Strike and dip of bedding

Strike and dip of bedding, top indicator observed

Strike and dip of bedding, approximate

Overturned bedding

Overturned bedding, top indicator observed

Horizontal bedding

Vertical bedding

Vertical bedding, top indicator observed

Strike and dip of foliation

Vertical foliation

Strike and dip of joints

REFERENCE:

Graymer, R.W., 2000, Geologic Map and map database of the Oakland Metropolitan Area, Alameda, Contra Costa, And San Francisco Counties, California: U.S. Geological Survey, Miscellaneous Field Studies Map MF - 2342, scale 1:50,000.

1200 0 600 1200 2400

APPROXIMATE SCALE: 1 inch = 1,200 feet

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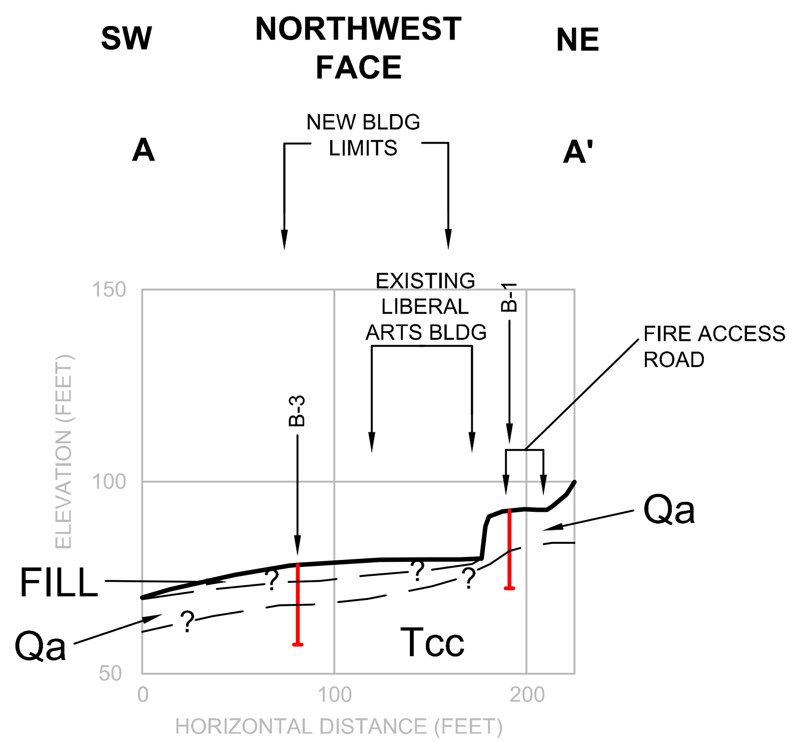
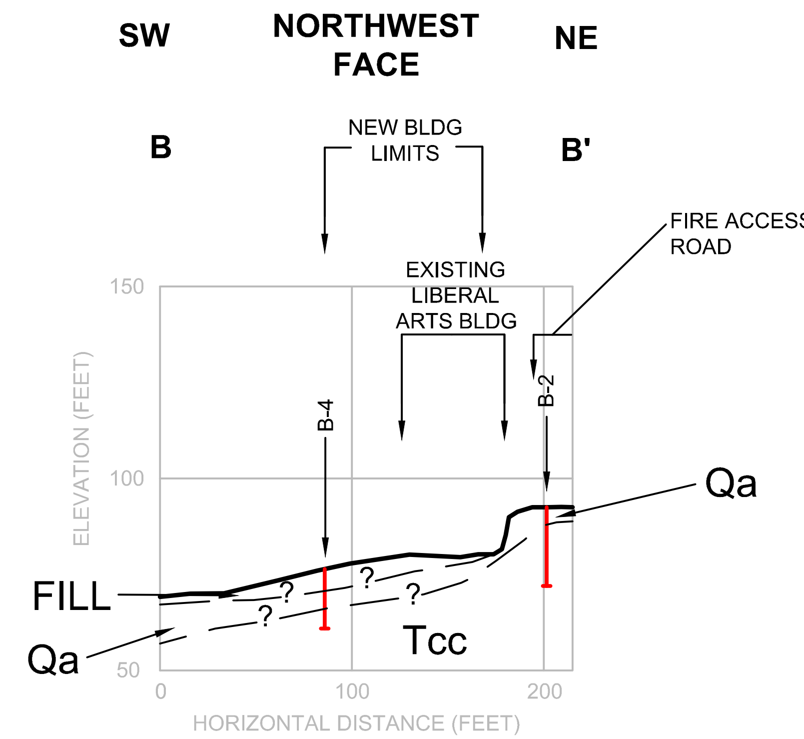
PROJECT NO. 20181569	AREA GEOLOGY MAP		FIGURE
DRAWN BY JDS			
CHECKED BY OK	CONTRA COSTA COLLEGE NEW SCIENCE BUILDING 2600 MISSION BELL DRIVE SAN PABLO, CALIFORNIA		4
DATE: 09/07/2017			
REVISED:			

PLOTTED: 10/5/2017 9:19 AM BY: jeff sala

LAYOUT: SECTION

CAD FILE: W:\2018\20181569.001A_CCCCD_C-4016 GeoSeis\GRAPHICs\GEOHAZ\SITE PLAN.dwg





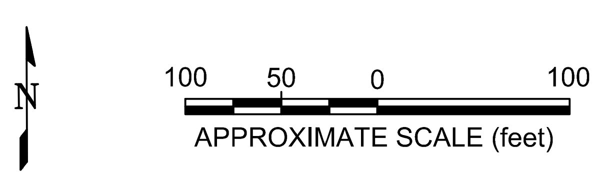
LEGEND

(heavy line)	GROUND SURFACE
— ? —	CONTACT LINE
Qa	QUATERNARY ALLUVIUM
Tcc	GARRITY MEMBER, CONTRA COSTA GROUP (TERTIARY)

NOTES:
1. LOCATIONS ARE APPROXIMATE.

2. SEE FIGURE 2 FOR APPROXIMATE CROSS SECTION LOCATIONS.

3. THESE CROSS SECTIONS ARE FOR ILLUSTRATIVE PURPOSES ONLY AND ARE BASED ON OUR INTERPRETATION OF THE SUBSURFACE CONDITIONS. THESE CROSS SECTIONS SHOULD NOT BE USED FOR CONSTRUCTION AND SHOULD BE CONSIDERED APPROXIMATE.



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PROJECT NO.	20181569
DRAWN BY	JDS
CHECKED BY	OK
DATE:	10/04/2017
REVISED:	

GEOLOGIC CROSS SECTION
A-A' AND B-B'

CONTRA COSTA COLLEGE
NEW SCIENCE BUILDING
2600 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA

FIGURE
5

APPENDIX A

Boring Logs

LIST OF ATTACHMENTS

The following figures are attached and complete this appendix.

SAMPLER AND DRILLING METHOD GRAPHICS

- BULK / GRAB / BAG SAMPLE
- MODIFIED CALIFORNIA SAMPLER (2 or 2-1/2 in. (50.8 or 63.5 mm.) outer diameter)
- CALIFORNIA SAMPLER (3 in. (76.2 mm.) outer diameter)
- STANDARD PENETRATION SPLIT SPOON SAMPLER (2 in. (50.8 mm.) outer diameter and 1-3/8 in. (34.9 mm.) inner diameter)
- SHELBY TUBE SAMPLER
- HOLLOW STEM AUGER
- SOLID STEM AUGER
- WASH BORING

GROUND WATER GRAPHICS

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

NOTES

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

ABBREVIATIONS

WOH - Weight of Hammer

WOR - Weight of Rod

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487)

COARSE GRAINED SOILS (More than half of material is larger than the #200 sieve)	**GRAVELS** (More than half of coarse fraction is larger than the #4 sieve)	CLEAN GRAVEL WITH <5% FINES	$Cu \geq 4$ and $1 \leq Cc \leq 3$	**GW**	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
			$Cu < 4$ and/ or $1 > Cc > 3$	**GP**	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
		GRAVELS WITH 5% TO 12% FINES	$Cu \geq 4$ and $1 \leq Cc \leq 3$	**GW-GM**	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GW-GC	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
			$Cu < 4$ and/ or $1 > Cc > 3$	**GP-GM**	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GP-GC	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
		GRAVELS WITH > 12% FINES		**GM**	SILTY GRAVELS, GRAVEL-SILT-SAND MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
				GC-GM	CLAYEY GRAVELS, GRAVEL-SAND-CLAY-SILT MIXTURES
	SANDS (More than half of coarse fraction is smaller than the #4 sieve)	CLEAN SANDS WITH <5% FINES	$Cu \geq 6$ and $1 \leq Cc \leq 3$	**SW**	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
			$Cu < 6$ and/ or $1 > Cc > 3$	**SP**	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
		SANDS WITH 5% TO 12% FINES	$Cu \geq 6$ and $1 \leq Cc \leq 3$	**SW-SM**	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SW-SC	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
			$Cu < 6$ and/ or $1 > Cc > 3$	**SP-SM**	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SP-SC	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
		SANDS WITH > 12% FINES		**SM**	SILTY SANDS, SAND-GRAVEL-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-GRAVEL-CLAY MIXTURES
				SC-SM	CLAYEY SANDS, SAND-SILT-CLAY MIXTURES

FINE GRAINED SOILS (More than half of material is smaller than the #200 sieve)	SILTS AND CLAYS (Liquid Limit less than 50)	**ML**	INORGANIC SILTS AND VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS, SILTS WITH SLIGHT PLASTICITY
		CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		CL-ML	INORGANIC CLAYS-SILTS OF LOW PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS (Liquid Limit greater than 50)	**MH**	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILT
		CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
		OH	ORGANIC CLAYS & ORGANIC SILTS OF MEDIUM-TO-HIGH PLASTICITY

KLEINFELDER
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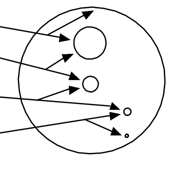
PROJECT NO.: 20181569	GRAPHICS KEY	FIGURE
DRAWN BY: MAP/JDS		A-1
CHECKED BY: OK	CONTRA COSTA COMMUNITY COLLEGE NEW SCIENCE BUILDING 2600 MISSION BELL DRIVE SAN PABLO, CALIFORNIA	
DATE: 9/19/2017		
REVISED: -		

GRAIN SIZE

DESCRIPTION		SIEVE SIZE	GRAIN SIZE	APPROXIMATE SIZE
Boulders		>12 in. (304.8 mm.)	>12 in. (304.8 mm.)	Larger than basketball-sized
Cobbles		3 - 12 in. (76.2 - 304.8 mm.)	3 - 12 in. (76.2 - 304.8 mm.)	Fist-sized to basketball-sized
Gravel	coarse	3/4 -3 in. (19 - 76.2 mm.)	3/4 -3 in. (19 - 76.2 mm.)	Thumb-sized to fist-sized
	fine	#4 - 3/4 in. (#4 - 19 mm.)	0.19 - 0.75 in. (4.8 - 19 mm.)	Pea-sized to thumb-sized
Sand	coarse	#10 - #4	0.079 - 0.19 in. (2 - 4.9 mm.)	Rock salt-sized to pea-sized
	medium	#40 - #10	0.017 - 0.079 in. (0.43 - 2 mm.)	Sugar-sized to rock salt-sized
	fine	#200 - #40	0.0029 - 0.017 in. (0.07 - 0.43 mm.)	Flour-sized to sugar-sized
Fines		Passing #200	<0.0029 in. (<0.07 mm.)	Flour-sized and smaller



SECONDARY CONSTITUENT

	AMOUNT	
Term of Use	Secondary Constituent is Fine Grained	Secondary Constituent is Coarse Grained
Trace	<5%	<15%
With	≥5 to <15%	≥15 to <30%
Modifier	≥15%	≥30%

MOISTURE CONTENT

DESCRIPTION	FIELD TEST
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

CEMENTATION

DESCRIPTION	FIELD TEST
Weakly	Crumbles or breaks with handling or slight finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

CONSISTENCY - FINE-GRAINED SOIL

CONSISTENCY	SPT - N_{60} (# blows / ft)	Pocket Pen (tsf)	UNCONFINED COMPRESSIVE STRENGTH (Q_u)(psf)	VISUAL / MANUAL CRITERIA
Very Soft	<2	PP < 0.25	<500	Thumb will penetrate more than 1 inch (25 mm). Extrudes between fingers when squeezed.
Soft	2 - 4	0.25≤ PP <0.5	500 - 1000	Thumb will penetrate soil about 1 inch (25 mm). Remolded by light finger pressure.
Medium Stiff	4 - 8	0.5≤ PP <1	1000 - 2000	Thumb will penetrate soil about 1/4 inch (6 mm). Remolded by strong finger pressure.
Stiff	8 - 15	1≤ PP <2	2000 - 4000	Can be imprinted with considerable pressure from thumb.
Very Stiff	15 - 30	2≤ PP <4	4000 - 8000	Thumb will not indent soil but readily indented with thumbnail.
Hard	>30	4≤ PP	>8000	Thumbnail will not indent soil.

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

REACTION WITH HYDROCHLORIC ACID

DESCRIPTION	FIELD TEST
None	No visible reaction
Weak	Some reaction, with bubbles forming slowly
Strong	Violent reaction, with bubbles forming immediately

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	SPT-N_{60} (# blows/ft)	MODIFIED CA SAMPLER (# blows/ft)	CALIFORNIA SAMPLER (# blows/ft)	RELATIVE DENSITY (%)
Very Loose	<4	<4	<5	0 - 15
Loose	4 - 10	5 - 12	5 - 15	15 - 35
Medium Dense	10 - 30	12 - 35	15 - 40	35 - 65
Dense	30 - 50	35 - 60	40 - 70	65 - 85
Very Dense	>50	>60	>70	85 - 100

FROM TERZAGHI AND PECK, 1948

PLASTICITY

DESCRIPTION	LL	FIELD TEST
Non-plastic	NP	A 1/8-in. (3 mm.) thread cannot be rolled at any water content.
Low (L)	< 30	The thread can barely be rolled and the lump or thread cannot be formed when drier than the plastic limit.
Medium (M)	30 - 50	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.
High (H)	> 50	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.

STRUCTURE

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

ANGULARITY

DESCRIPTION	CRITERIA
Angular	Particles have sharp edges and relatively plane sides with unpolished surfaces.
Subangular	Particles are similar to angular description but have rounded edges.
Subrounded	Particles have nearly plane sides but have well-rounded corners and edges.
Rounded	Particles have smoothly curved sides and no edges.

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PROJECT NO.: 20181569	SOIL DESCRIPTION KEY	FIGURE
DRAWN BY: MAP/JDS		
CHECKED BY: OK	CONTRA COSTA COMMUNITY COLLEGE NEW SCIENCE BUILDING 2600 MISSION BELL DRIVE SAN PABLO, CALIFORNIA	A-2
DATE: 9/19/2017		
REVISED: -		

INFILLING TYPE

NAME	ABBR	NAME	ABBR
Albite	Al	Muscovite	Mus
Apatite	Ap	None	No
Biotite	Bi	Pyrite	Py
Clay	Cl	Quartz	Qz
Calcite	Ca	Sand	Sd
Chlorite	Ch	Sericite	Ser
Epidote	Ep	Silt	Si
Iron Oxide	Fe	Talc	Ta
Manganese	Mn	Unknown	Uk

DENSITY/SPACING OF DISCONTINUITIES

DESCRIPTION	SPACING CRITERIA
Unfractured	>6 ft. (>1.83 meters)
Slightly Fractured	2 - 6 ft. (0.061 - 1.83 meters)
Moderately Fractured	8 in - 2 ft. (203.20 - 609.60 mm)
Highly Fractured	2 - 8 in (50.80 - 203.30 mm)
Intensely Fractured	<2 in (<50.80 mm)

ADDITIONAL TEXTURAL ADJECTIVES

DESCRIPTION	RECOGNITION
Pit (Pitted)	Pinhole to 0.03 ft. (3/8 in.) (>1 to 10 mm.) openings
Vug (Vuggy)	Small openings (usually lined with crystals) ranging in diameter from 0.03 ft. (3/8 in.) to 0.33 ft. (4 in.) (10 to 100 mm.)
Cavity	An opening larger than 0.33 ft. (4 in.) (100 mm.), size descriptions are required, and adjectives such as small, large, etc., may be used
Honeycombed	If numerous enough that only thin walls separate individual pits or vugs, this term further describes the preceding nomenclature to indicate cell-like form.
Vesicle (Vesicular)	Small openings in volcanic rocks of variable shape and size formed by entrapped gas bubbles during solidification.

ADDITIONAL TEXTURAL ADJECTIVES

DESCRIPTION	CRITERIA
Unweathered	No evidence of chemical / mechanical alternation; rings with hammer blow.
Slightly Weathered	Slight discoloration on surface; slight alteration along discontinuities; <10% rock volume altered.
Moderately Weathered	Discoloring evident; surface pitted and alteration penetration well below surface; Weathering "halos" evident; 10-50% rock altered.
Highly Weathered	Entire mass discolored; Alteration pervading most rock, some slight weathering pockets; some minerals may be leached out.
Decomposed	Rock reduced to soil with relic rock texture/structure; Generally molded and crumbled by hand.

RELATIVE HARDNESS / STRENGTH DESCRIPTIONS

GRADE		UCS (Mpa)	FIELD TEST
R0	Extremely Weak	0.25 - 1.0	Indented by thumbnail
R1	Very Weak	1.0 - 5.0	Crumbles under firm blows of geological hammer, can be peeled by a pocket knife.
R2	Weak	5.0 - 25	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer.
R3	Medium Strong	25 - 50	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single firm blow of a geological hammer.
R4	Strong	50 - 100	Specimen requires more than one blow of geological hammer to fracture it.
R5	Very Strong	100 - 250	Specimen requires many blows of geological hammer to fracture it.
R6	Extremely Strong	> 250	Specimen can only be chipped with a geological hammer.

ROCK QUALITY DESIGNATION (RQD)

DESCRIPTION	RQD (%)
Very Poor	0 - 25
Poor	25 - 50
Fair	50 - 75
Good	75 - 90
Excellent	90 - 100

APERTURE

DESCRIPTION	CRITERIA [in (mm)]
Tight	<0.04 (<1)
Open	0.04 - 0.20 (1 - 5)
Wide	>0.20 (>5)

BEDDING CHARACTERISTICS

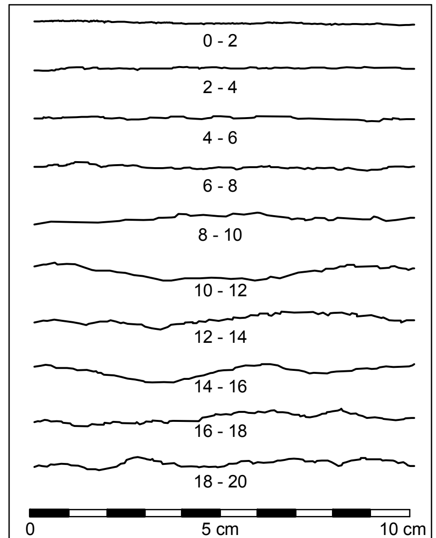
DESCRIPTION	Thickness [in (mm)]
Very Thick Bedded	>36 (>915)
Thick Bedded	12 - 36 (305 - 915)
Moderately Bedded	4 - 12 (102 - 305)
Thin Bedded	1 - 4 (25 - 102)
Very Thin Bedded	0.4 - 1 (10 - 25)
Laminated	0.1 - 0.4 (2.5 - 10)
Thinly Laminated	<0.1 (<2.5)

Bedding Planes — Planes dividing the individual layers, beds, or stratigraphy of rocks.

Joint — Fracture in rock, generally more or less vertical or traverse to bedding.

Seam — Applies to bedding plane with unspecified degree of weather.

JOINT ROUGHNESS COEFFICIENT (JRC)



From Barton and Choubey, 1977

RQD — Rock-quality designation (RQD) Rough measure of the degree of jointing or fracture in a rock mass, measured as a percentage of the drill core in lengths of 10 cm. or more.

CORE SAMPLER TYPE GRAPHICS

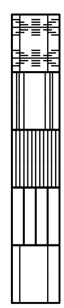
CORE SAMPLER

AQ CORE BARREL
(1.067 in. (27.1 mm.) core diameter)

AX CORE BARREL
(1.185 in. (30.1 mm.) core diameter)

BQ CORE BARREL
(1.433 in. (36.4 mm.) core diameter)

CONTINUOUS CORE SAMPLE
(2.000 in. (50.8 mm.) core diameter)



EX CORE BARREL
(0.846 in. (21.5 mm.) core diameter)

HQ CORE SAMPLE
(2.500 in. (63.5 mm.) core diameter)

NQ CORE SAMPLE
(1.874 in. (47.6 mm.) core diameter)

NO RECOVERY CORE SAMPLE

NX CORE SAMPLE
(2.154 in. (54.7 mm.) core diameter)

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20181569 DRAWN BY: MAP/JDS CHECKED BY: OK DATE: 9/19/2017 REVISED: -	ROCK DESCRIPTION KEY CONTRA COSTA COMMUNITY COLLEGE NEW SCIENCE BUILDING 2600 MISSION BELL DRIVE SAN PABLO, CALIFORNIA	FIGURE A-3

PLOTTED: 10/03/2017 01:39 PM BY: JSala

Date Begin - End:	8/11/2017	**Drilling Co.-Lic.#:**	Gregg - #CA107979		**BORING LOG B-1**
Logged By:	J. Anderson	**Drill Crew:**	Jeremy		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Truck Mounted M11	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Cloudy	**Exploration Diameter:**	Approx. 6 in.		

Latitude: 37.96986° N
Longitude: -122.33678° E
Approximate Ground Surface Elevation (ft.): 92.00
Surface Condition: Asphalt

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description (FIELD EXPLORATION)	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol (LABORATORY RESULTS)	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
	0		approximately 2-inches of asphalt											
90			**Sandy Lean CLAY with Gravel (CL)**: low plasticity, yellowish brown, moist, stiff to very stiff, subrounded to subangular gravel		BC=5 7 9	12"		18.9	109.7					
85	5		olive brown, stiff to very stiff		BC=5 6 8	12"		19.1	108.8					TXUU: c = 2.12 ksf
80	10		**Sandy Lean CLAY (CL)**: fine-grained sand, some gravel, medium plasticity, reddish yellow mottled, moist, very stiff		BC=6 10 14	12"		14.0	115.8					TXUU: c = 2.55 ksf
			some angular claystone fragments, yellowish brown, hard											
75	15				BC=12 18 22	12"								
70	20		**CLAYSTONE**: fine-grained, medium plasticity, yellowish brown, moderately weathered, weak to medium strong		BC=22 36 50/5"	11"								
65	25				BC=11 29 50	12"								
60	30		moderately weathered, weak to medium strong, interbedded with siltstone		BC=29 50/3"	8"								

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20181569	BORING LOG B-1	FIGURE
	DRAWN BY: MAP/JDS		A-4
	CHECKED BY: OK	CONTRA COSTA COMMUNITY COLLEGE NEW SCIENCE BUILDING 2600 MISSION BELL DRIVE SAN PABLO, CALIFORNIA	
	DATE: 9/19/2017		
	REVISED: -		PAGE: 1 of 2

gINT FILE: Klf_gint_master_2017 PROJECT NUMBER: 20181569.001A OFFICE FILTER: PLEASANTON
gINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2017.GLB [_KLF_BORING/TEST PIT SOIL LOG]

PLOTTED: 10/03/2017 01:39 PM BY: JSala

Date Begin - End:	8/11/2017	**Drilling Co.-Lic.#:**	Gregg - #CA107979		**BORING LOG B-1**
Logged By:	J. Anderson	**Drill Crew:**	Jeremy		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Truck Mounted M11	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Cloudy	**Exploration Diameter:**	Approx. 6 in.		

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 37.96986° N
Longitude: -122.33678° E
Approximate Ground Surface Elevation (ft.): 92.00
Surface Condition: Asphalt

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
55			**CLAYSTONE**: fine-grained, yellowish brown, moderately weathered, medium strong		BC=26 50	2"								
	40		- light brownish gray, slightly weathered, medium strong to strong		BC=44 50/2"	8"								
50														
	45													
45														
	50													
40														
	55													
35														
	60													
30														
	65													
25														

The boring was terminated at approximately 40.5 ft. below ground surface. The boring was backfilled with cement grout on August 11, 2017.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.
GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder.

gINT FILE: Klf_gint_master_2017 PROJECT NUMBER: 20181569.001A OFFICE FILTER: PLEASANTON
gINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2017.GLB [_KLF_BORING/TEST PIT SOIL LOG]

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PROJECT NO.: 20181569	BORING LOG B-1	FIGURE
DRAWN BY: MAP/JDS		
CHECKED BY: OK	CONTRA COSTA COMMUNITY COLLEGE NEW SCIENCE BUILDING 2600 MISSION BELL DRIVE SAN PABLO, CALIFORNIA	A-4
DATE: 9/19/2017		
REVISED: -		

Date Begin - End:	8/11/2017	**Drilling Co.-Lic.#:**	Gregg - #CA107979		**BORING LOG B-2**
Logged By:	J. Anderson	**Drill Crew:**	Jeremy		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Truck Mounted M11	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Cloudy	**Exploration Diameter:**	Approx. 6 in.		

FIELD EXPLORATION — LABORATORY RESULTS

Latitude: 37.96973° N
Longitude: -122.33647° E
Approximate Ground Surface Elevation (ft.): 93.00
Surface Condition: Asphalt

Approximate Elevation (feet)	Depth (feet)	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		approximately 2-inches of asphalt											
		Clayey SAND (SC): fine to medium-grained sand, low plasticity, mottled yellowish brown, dry, medium dense											
90		**Lean CLAY (CL)**: medium plasticity, yellowish brown, moist, very stiff		BC=10 12 14	12"		11.3	110.8					
	5	**CLAYSTONE**: fine-grained, yellowish brown, moderately weathered to highly weathered, weak to medium strong		BC=17 18 26	6"								
85		reddish yellow, fragmented moderately weathered, weak to medium strong											
	10			BC=16 14 50/4"	10"		9.5	118.9					
80													
	15	olive brown, weak to medium strong		BC=14 36 50/5"	2"								Very hard drilling
75													
	20	- yellowish brown with reddish brown stains, moderately weathered, intensely fractured medium strong		BC=23 50	4"								
70													
	25	weak		BC=13 14 20	2"								
65													
	30	medium-grained, yellow, moderately weathered, weak, highly fractured, interbedded with subrounded gravel		BC=11 18 34	10"								
60													

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PROJECT NO.:	20181569
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DATE:	9/19/2017
REVISED:	-

BORING LOG B-2

CONTRA COSTA COMMUNITY COLLEGE
NEW SCIENCE BUILDING
2600 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA

FIGURE
A-5

Date Begin - End:	8/11/2017	Drilling Co.-Lic.#:	Gregg - #CA107979		**BORING LOG B-2**
Logged By:	J. Anderson	**Drill Crew:**	Jeremy		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Truck Mounted M11	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Cloudy	**Exploration Diameter:**	Approx. 6 in.		

Latitude: 37.96973° N
Longitude: -122.33647° E
Approximate Ground Surface Elevation (ft.): 93.00
Surface Condition: Asphalt

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description (FIELD EXPLORATION)	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
			CLAYSTONE: fine-grained, yellowish brown, moderately weathered to highly weathered, weak to medium strong		BC=9 29 50/5"	3"								
55			fine-grained, light brownish gray, weak to medium strong, highly fractured											
	40				BC=21 50									

Elevation scale: 55, 50, 45, 40, 35, 30, 25. Depth scale: 40, 45, 50, 55, 60, 65.

The boring was terminated at approximately 41 ft. below ground surface. The boring was backfilled with cement grout on August 11, 2017.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.
GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder.

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PROJECT NO.: 20181569	BORING LOG B-2	FIGURE
DRAWN BY: MAP/JDS		
CHECKED BY: OK	CONTRA COSTA COMMUNITY COLLEGE NEW SCIENCE BUILDING 2600 MISSION BELL DRIVE SAN PABLO, CALIFORNIA	A-5
DATE: 9/19/2017		
REVISED: -		PAGE: 2 of 2

BORING LOG B-3

Date Begin - End:	8/18/2017	**Drilling Co.-Lic.#:**	Gregg - #CA107979
Logged By:	J. Anderson	**Drill Crew:**	Jeremy and Leo
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	D42
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger
Weather:	Overcast	**Exploration Diameter:**	Approx. 6 in.

Hammer Type - Drop: 140 lb. Auto - 30 in.

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 37.96965° N
Longitude: -122.33695° E
Approximate Ground Surface Elevation (ft.): 80.00
Surface Condition: Grass

Approximate Elevation (feet)	Depth (feet)	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		Sandy Lean CLAY (CL): medium plasticity, olive brown, moist, very stiff, (FILL)											
				BC=3 8 13	12"						27	12	
75	5	**Lean CLAY with Sand (CL)**: medium plasticity, olive brown, moist, very stiff, (FILL)		BC=4 8 12	11"								
		Sandy Lean CLAY (CL): medium plasticity, yellowish brown, moist, stiff											
70	10			BC=2 4 7	12"		26.8	94.7					TXUU: c = 1.25 ksf
65	15	**Clayey SAND (SC)**: non-plastic to low plasticity, yellowish brown, moist, loose		BC=4 4 5	12"	SC				49	33	18	
60	20	**CLAYSTONE**: fine-grained, olive brown, weak to medium strong, interbedded with siltstone		BC=20 42 50/5"	11"								
55	25	light gray, medium strong to strong		BC=40 50/5"	11"								
50	30	moderately to slightly weathered, weak, highly fractured		BC=20 25 26	12"								

PROJECT NO.:	20181569
DRAWN BY:	MAP/JDS
CHECKED BY:	OK
DATE:	9/19/2017
REVISED:	-

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BORING LOG B-3

CONTRA COSTA COMMUNITY COLLEGE
NEW SCIENCE BUILDING
2600 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA

FIGURE
A-6

BORING LOG B-3

Date Begin - End:	8/18/2017	**Drilling Co.-Lic.#:**	Gregg - #CA107979		
Logged By:	J. Anderson	**Drill Crew:**	Jeremy and Leo		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	D42	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Overcast	**Exploration Diameter:**	Approx. 6 in.		

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 37.96965° N
Longitude: -122.33695° E
Approximate Ground Surface Elevation (ft.): 80.00
Surface Condition: Grass

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
			CLAYSTONE: fine-grained, olive gray, weak		BC=18 27 30	12"								
40	40		olive, medium strong		BC=17 36 50/5"									
35	45													
30	50													
25	55													
20	60													
15	65													

The boring was terminated at approximately 41.5 ft. below ground surface. The boring was backfilled with cement grout on August 18, 2017.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.
GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder.

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PROJECT NO.: 20181569	BORING LOG B-3		FIGURE
DRAWN BY: MAP/JDS			
CHECKED BY: OK	CONTRA COSTA COMMUNITY COLLEGE NEW SCIENCE BUILDING 2600 MISSION BELL DRIVE SAN PABLO, CALIFORNIA		A-6
DATE: 9/19/2017			
REVISED: -			PAGE: 2 of 2

BORING LOG B-4

Date Begin - End:	8/18/2017	**Drilling Co.-Lic.#:**	Gregg - #CA107979		
Logged By:	J. Anderson	**Drill Crew:**	Jeremy and Leo		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	D42	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Overcast	**Exploration Diameter:**	Approx. 6 in.		

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 37.96953° N
Longitude: -122.33673° E
Approximate Ground Surface Elevation (ft.): 80.00
Surface Condition: Grass

Approximate Elevation (feet)	Depth (feet)	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
		Lean Fat CLAY with Sand (CL): medium to high plasticity, olive brown, moist, hard, (FILL)											
				BC=11 13 16 PP=4-4.5+	11"								
75	5	**Lean CLAY with Sand (CL)**: medium plasticity, olive brown, moist, hard, (FILL)		BC=9 12 23 PP=4.5	12"						43	28	
70	10	increase in sand content, very stiff, organics, brick fragments with gravel and brick at 11.5 feet		BC=9 11 12 PP=1.5-1.75									
		Clayey GRAVEL with Sand (GC): dark brown, moist, medium dense, fine to coarse gravel											
65	15			BC=17 18 12									
		Clayey SAND with Gravel (SC): medium to coarse-grained, olive brown, moist, medium dense								16			
60	20	**Sandy CLAYSTONE**: fine-grained, olive, weak to medium strong, moderately weathered, interbedded with siltstone		BC=20 27 25	12"								
55	25	medium strong		BC=18 33 48	12"								
50	30	medium strong to strong		BC=27 50/5"									

The boring was terminated at approximately 31 ft. below ground surface. The boring was backfilled with cement grout on August 18, 2017.

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

GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder.

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PROJECT NO.:	20181569
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DATE:	9/19/2017
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BORING LOG B-4

CONTRA COSTA COMMUNITY COLLEGE
NEW SCIENCE BUILDING
2600 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA

FIGURE
A-7

APPENDIX B

Laboratory Results

LIST OF ATTACHMENTS

The following figures are attached and complete this appendix.

Exploration ID	Depth (ft.)	Sample Description	Water Content (%)	Dry Unit Wt. (pcf)	Sieve Analysis (%)			Atterberg Limits			Additional Tests
					Passing 3/4"	Passing #4	Passing #200	Liquid Limit	Plastic Limit	Plasticity Index	
B-1	2.5	YELLOWISH BROWN SANDY LEAN CLAY WITH GRAVEL (CL)	18.9	109.7							
B-1	6.0	OLIVE BROWN SANDY LEAN CLAY (CL)	19.1	108.8							TXUU: c = 2.12 ksf
B-1	11.0	REDDISH YELLOW MOTTLED SANDY LEAN CLAY (CL)	14.0	115.8							TXUU: c = 2.55 ksf
B-2	2.5	YELLOWISH BROWN LEAN CLAY (CL)	11.3	110.8							
B-2	11.0	REDDISH YELLOW CLAYSTONE	9.5	118.9							
B-3	2.5	OLIVE BROWN CLAYEY SAND (SC)						27	15	12	
B-3	11.0	YELLOWISH BROWN SANDY LEAN CLAY (CL)	26.8	94.7							TXUU: c = 1.25 ksf
B-3	16.0	OLIVE BROWN CLAYEY SAND (SC)					49	33	15	18	
B-4	6.0	OLIVE BROWN LEAN CLAY WITH SAND (CL)						43	15	28	
B-4	16.0	OLIVE BROWN CLAYEY SAND WITH GRAVEL (SC)					16				

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

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PROJECT NO.: 20181569
DRAWN BY: MAP/JDS
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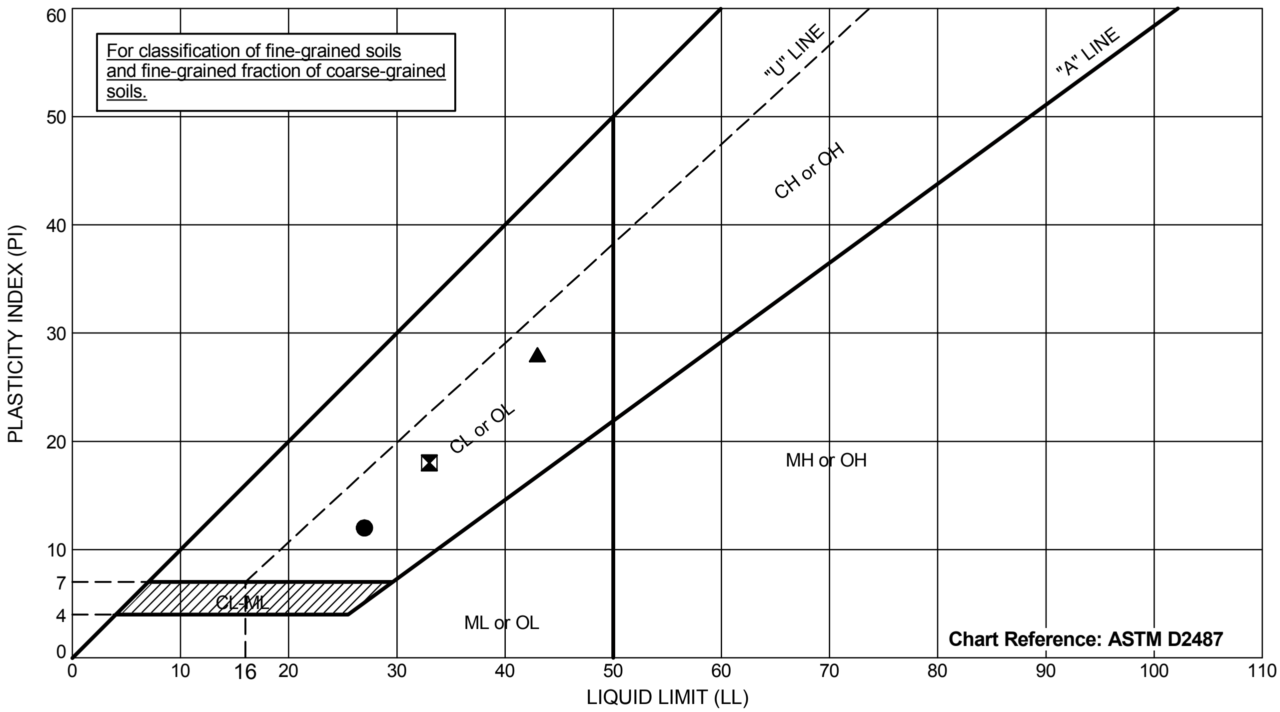
DATE: 9/19/2017
REVISED: -

LABORATORY TEST RESULT SUMMARY

CONTRA COSTA COMMUNITY COLLEGE
NEW SCIENCE BUILDING
2600 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA

FIGURE

B-1



	Exploration ID	Depth (ft.)	Sample Description	Passing #200	LL	PL	PI
●	B-3	2.5	OLIVE BROWN CLAYEY SAND (SC)	NM	27	15	12
⊠	B-3	16	OLIVE BROWN CLAYEY SAND (SC)	49	33	15	18
▲	B-4	6	OLIVE BROWN LEAN CLAY WITH SAND (CL)	NM	43	15	28

Testing perfomed in general accordance with ASTM D4318.
NP = Nonplastic
NM = Not Measured

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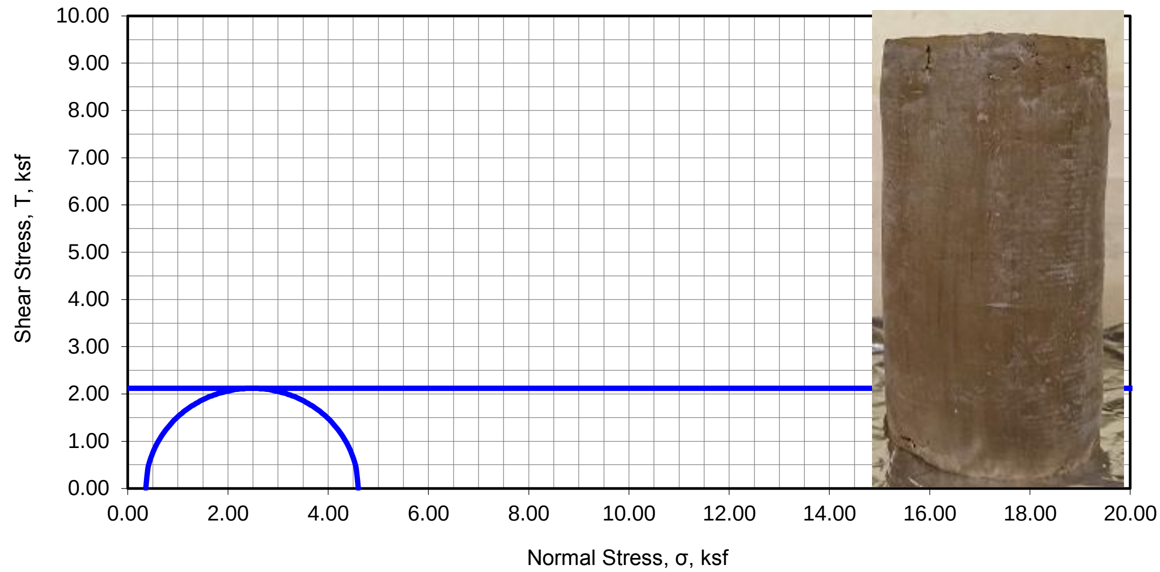
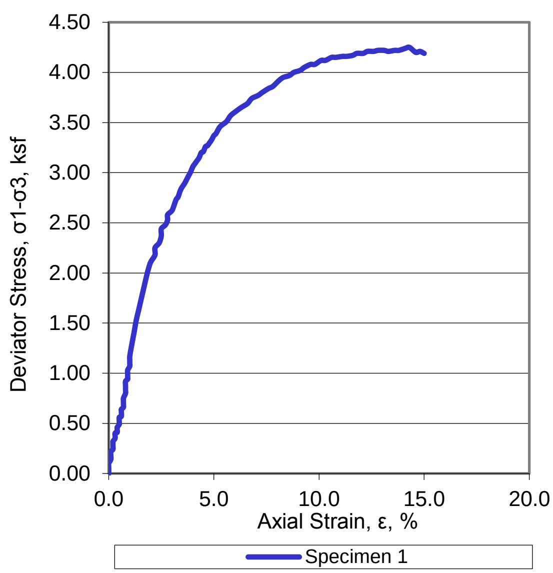
PROJECT NO.: 20181569	ATTERBERG LIMITS	FIGURE
DRAWN BY: MAP/JDS		
CHECKED BY: OK	CONTRA COSTA COMMUNITY COLLEGE NEW SCIENCE BUILDING 2600 MISSION BELL DRIVE SAN PABLO, CALIFORNIA	B-2
DATE: 9/19/2017		
REVISED: -		

Total	
c = 2.12	ksf

Specimen Shear Picture





Specimen No.			1
Initial	Diameter, in	D_O	2.39
Initial	Height, in	H_O	5.69
Initial	Water Content, %	ω_O	19.1
Initial	Dry Density, lbs/ft^3	γd_o	108.8
Initial	Saturation, %	S_O	97
Initial	Void Ratio	e_O	0.519
Minor Principal Stress, ksf		σ_3	0.36
Maximum Deviator Stress, ksf		$(\sigma 1-\sigma 3)_{max}$	4.25
Time to $(\sigma_1-\sigma_3)_{max}$, min		t_f	14.33
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	4.19
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		$'\varepsilon$	1.00
Axial Strain at Failure, %		ε_f	14.33

Description of Specimen: Olive Brown Sandy Lean Clay (CL)

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

Boring:	B-1
Sample:	2C
Depth, ft:	6.0
Test Date:	9/8/17

Remarks: nm= not measured, na = not applicable

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2601 Barrington Ct, Hayward, CA 94545

Project No.:	20181569
Date:	9/11/17
Entry By:	CP
Checked By:	CP
File Name:	HL10558

TRIAXIAL COMPRESSION TEST (UU)

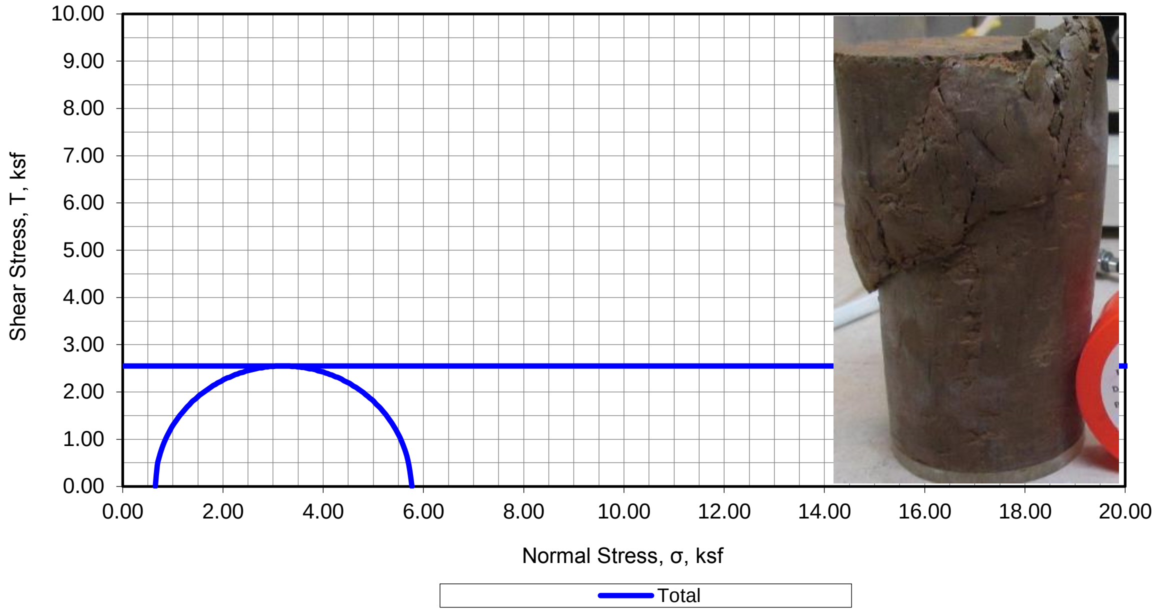
CONTRA COSTA COMMUNITY COLLEGE
NEW SCIENCE BUILDING
2600 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA

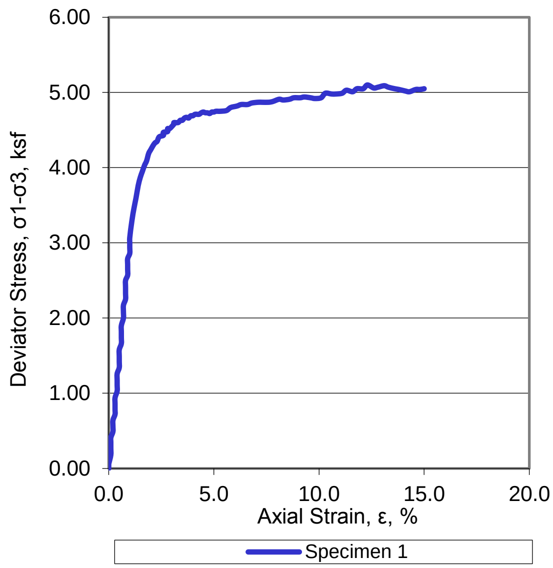
FIGURE

B-3

Total	
c = 2.55	ksf

Specimen Shear Picture





Specimen No.			1
Initial	Diameter, in	D_O	2.43
	Height, in	H_O	5.67
	Water Content, %	ω_O	14.0
	Dry Density, lbs/ft^3	γd_o	115.8
	Saturation, %	S_O	87
	Void Ratio	e_O	0.428
Minor Principal Stress, ksf		σ_3	0.65
Maximum Deviator Stress, ksf		$(\sigma1-\sigma3)_{max}$	5.10
Time to $(\sigma_1\text{-}\sigma_3)_{max}$, min		t_f	12.33
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	5.05
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		'ε	1.00
Axial Strain at Failure, %		ε_f	12.33

Description of Specimen: Brown Lean Clay with Sand (CL)

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

Boring:	B-1
Sample:	3C
Depth, ft:	11.0
Test Date:	9/8/17

Remarks: nm= not measured, na = not applicable

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Project No.:	20181569
Date:	9/11/17
Entry By:	CP
Checked By:	CP
File Name:	HL10558

TRIAXIAL COMPRESSION TEST (UU)

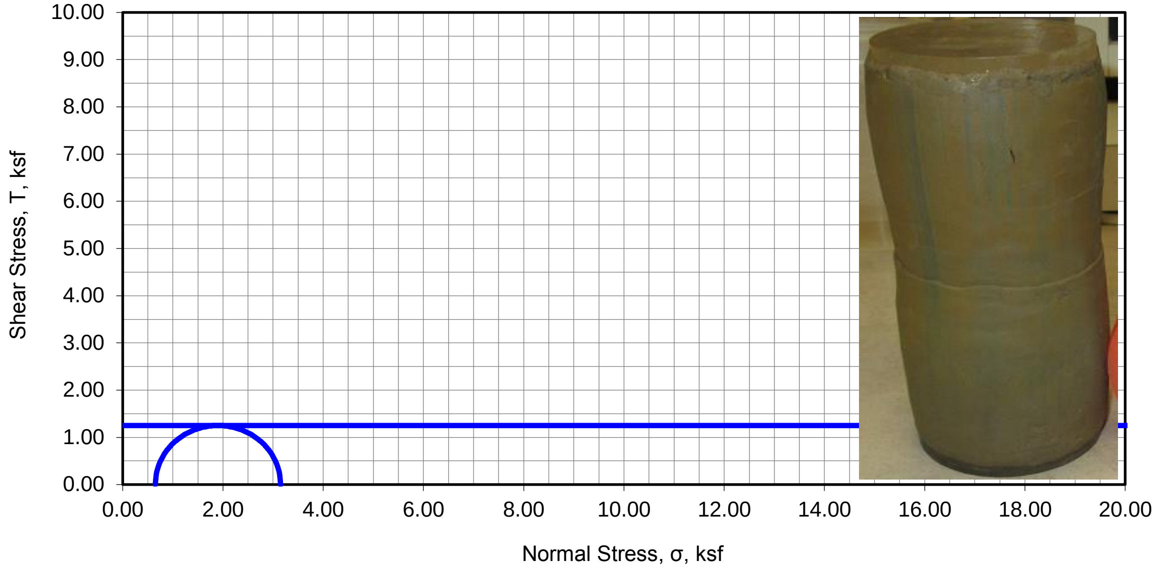
CONTRA COSTA COMMUNITY COLLEGE
NEW SCIENCE BUILDING
2600 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA

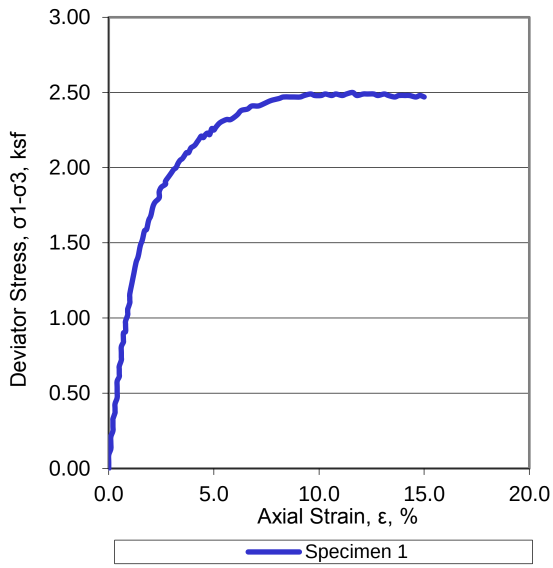
FIGURE

B-4

Total	
c = 1.25	ksf

Specimen Shear Picture





Specimen No.			1
Initial	Diameter, in	D_O	2.43
	Height, in	H_O	5.70
	Water Content, %	ω_O	26.8
	Dry Density, lbs/ft^3	γd_O	94.7
	Saturation, %	S_O	95
	Void Ratio	e_O	0.747
Minor Principal Stress, ksf		σ_3	0.65
Maximum Deviator Stress, ksf		$(\sigma 1-\sigma 3)_{max}$	2.50
Time to $(\sigma_1-\sigma_3)_{max}$, min		t_f	11.58
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	2.47
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		$'\varepsilon$	1.00
Axial Strain at Failure, %		ε_f	11.58

Description of Specimen: Brown Lean Clay (CL)

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

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

Membrane correction applied

Boring:	B-3
Sample:	3C
Depth, ft:	11.0
Test Date:	9/8/17

Remarks: nm= not measured, na = not applicable

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Project No.:	20181569
Date:	9/11/17
Entry By:	CP
Checked By:	CP
File Name:	HL10558

TRIAXIAL COMPRESSION TEST (UU)

CONTRA COSTA COMMUNITY COLLEGE
NEW SCIENCE BUILDING
2600 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA

FIGURE
B-5

APPENDIX C

Boring Logs and Trench Logs from Previous Kleinfelder Studies

- Kleinfelder, 2003, Subsurface Fault Investigation, Proposed Addition to the Student Activities Building
- Kleinfelder, 2004, Geotechnical Investigation Report, Student Activities Building Addition
- Kleinfelder, 2007, Subsurface Fault Investigation at the Existing Student Activities Building
- Kleinfelder, 2008, Subsurface Fault Investigation in Vicinity of the Existing Humanities Building
- Kleinfelder, 2011, Geotechnical Investigation Report, Campus Center

Kleinfelder, 2003, Subsurface Fault Investigation, Proposed Addition to the Student Activities Building

UNIT DESCRIPTION

① LEAN CLAY (CL) with sand, gravel, and cobbles, black (7.5YR 2.5/1), moist, hard, slightly porous, prismatic, contains a stoneline with clast lithologies comprised of subrounded Tertiary volcanics (basalt and rhyolite) measuring up to 6-inches in diameter (Holocene alluvium)

② Sandy LEAN/FAT CLAY with sand and gravel (CL-CH), dark gray (10YR 4/1), moist, hard, sand and gravel clasts are angular to subrounded and are comprised of sandstone, chert, vein quartz, siliceous shale and Tertiary volcanics (Holocene/Pleistocene alluvium)

③ LEAN/FAT CLAY (CL/CH) with sand and gravel, dark grayish brown (10YR 4/2), moist, hard, prismatic, contains angular to subrounded gravels that are composed of Mesozoic variegated chert and sandstone (Pleistocene alluvium)

④ Clayey SILT (ML) with sand and gravel, olive brown (2.5YR 4/4), moist, hard, contains veinlets of white carbonate, gravels are angular to subrounded Mesozoic clasts (Pleistocene alluvium)

⑤ SANDTONE with interbeds of siltstone, claystone, and conglomerate, laminated locally, moist, gray to light brown, highly weathered, widely fractured, weak to friable, oxide stained, horizontal to gently inclined bedding towards the south, contained Teleoceras (Rhinoceras) scapula fossil fragment similar to the late Hemphillian (Late Miocene) fauna from the Pinole Tuff. (Correlated to the Late Miocene Garrity Member of the Contra Costa Group Bedrock).

⑥ CLAYEY GRAVEL (GC) with sand, damp to moist, dense to very dense, oxidized clayey coating surrounds gravels, gravels are subrounded to rounded Mesozoic graywacke, variegated chert, siliceous shale and Tertiary volcanics (pleistocene channel deposit)

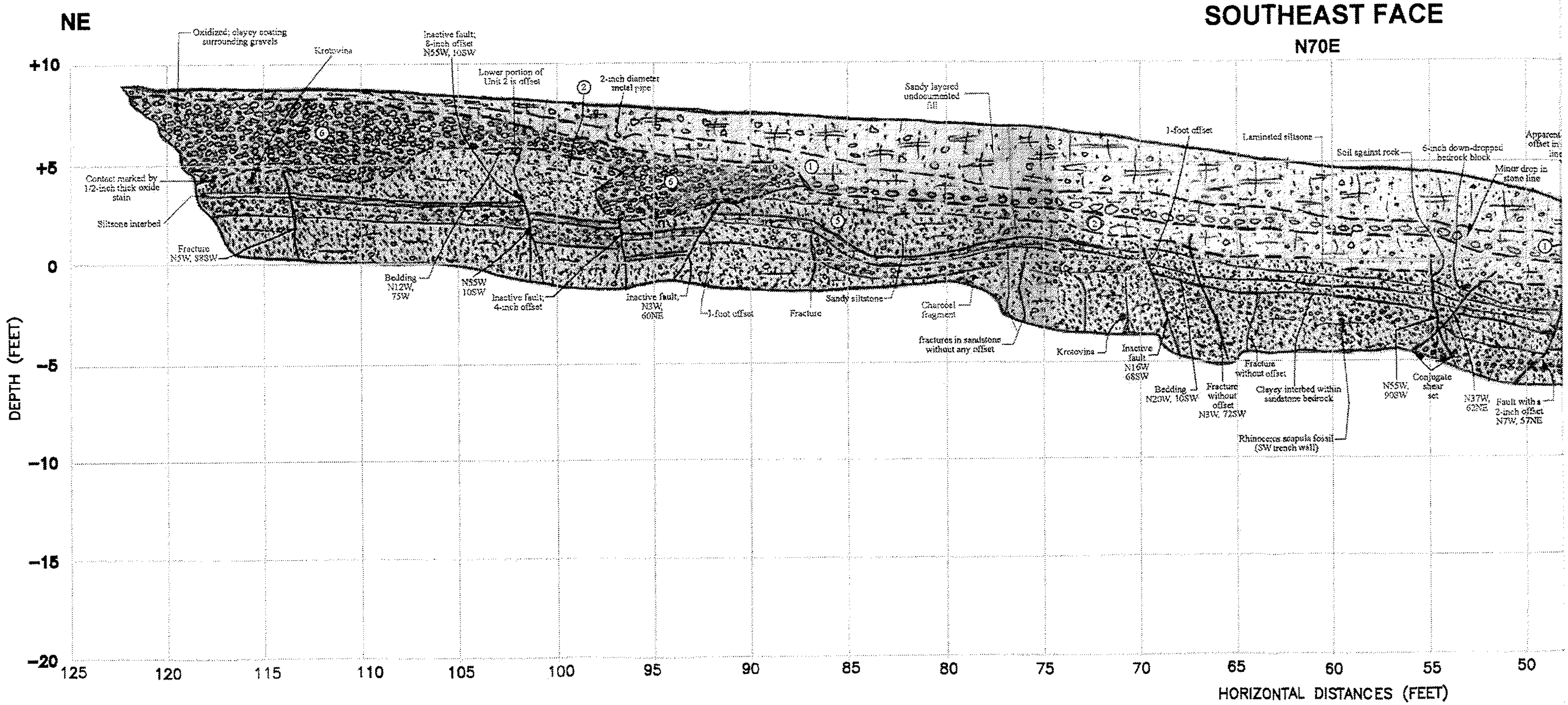
⑦ CLAYSTONE with trace gravel, olive, moist, foliated, plastic, reddish stain throughout, grades upward into sandstone, polished (Garrity Member of the Contra Costa Group Bedrock)

⑧ LEAN CLAY (CL) with sand and silt, very dark brown to black, moist, hard, prismatic, minor gravel within (Holocene/Pleistocene alluvium)

TRENCH T-1

SOUTHEAST FACE

N70E

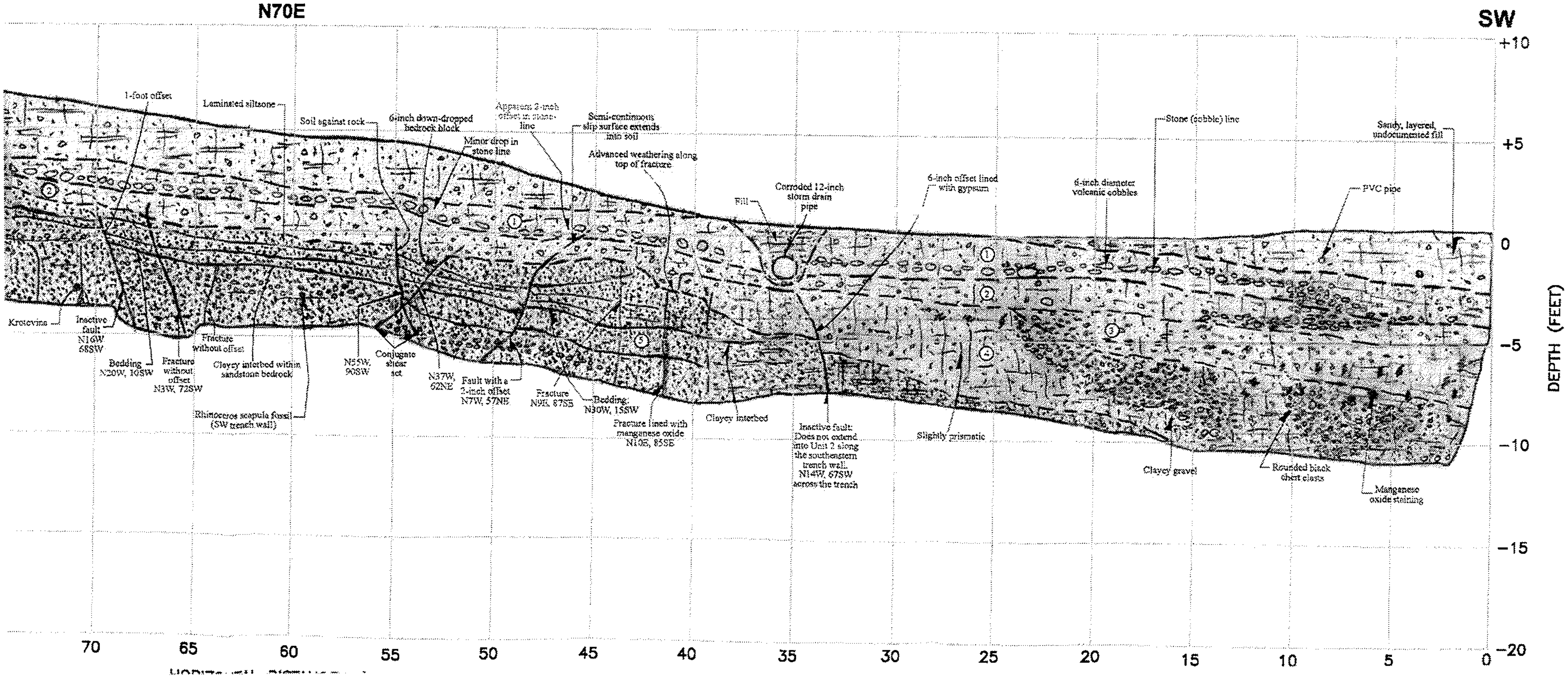


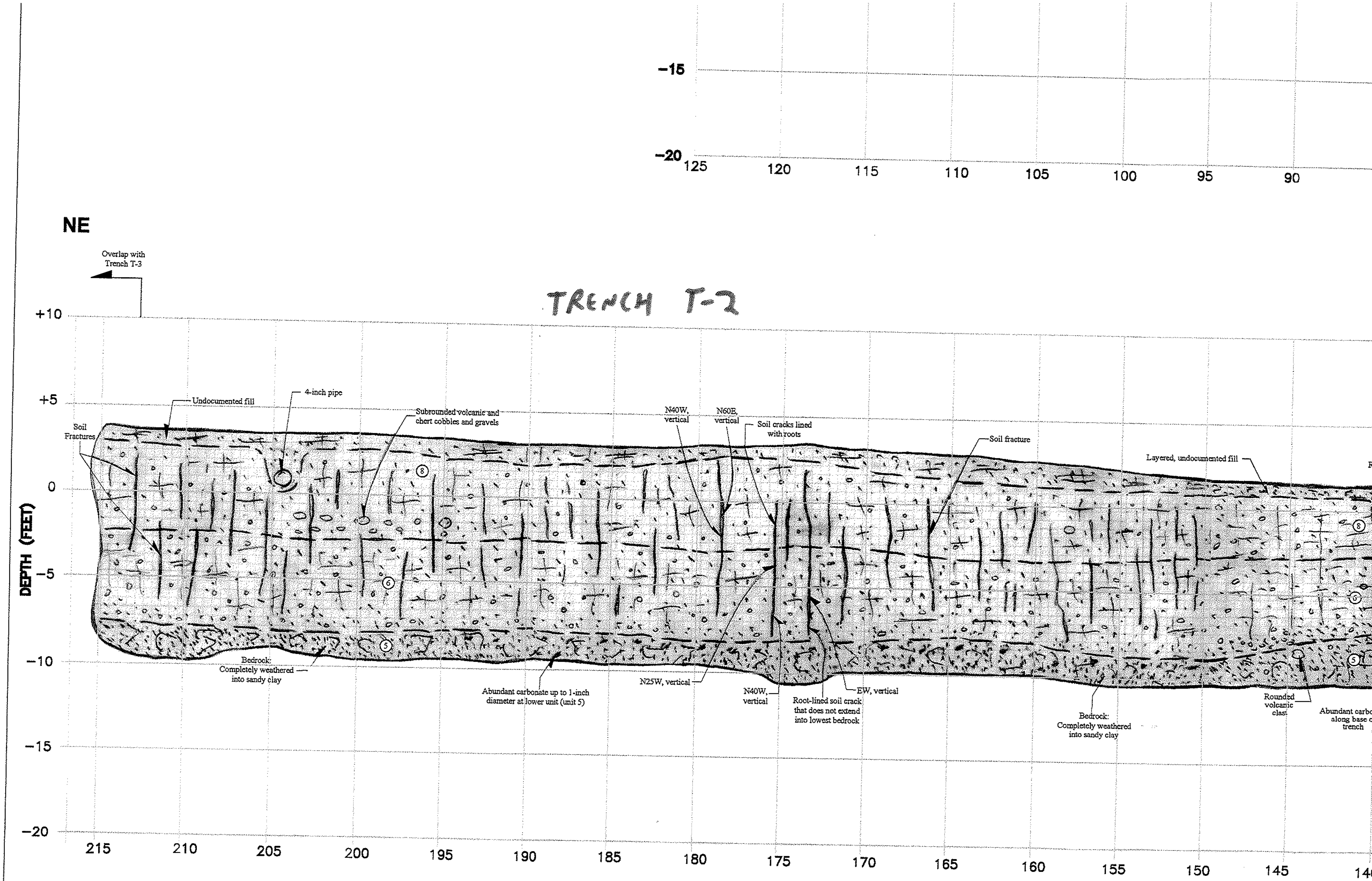
TRENCH T-2

TRENCH T-1

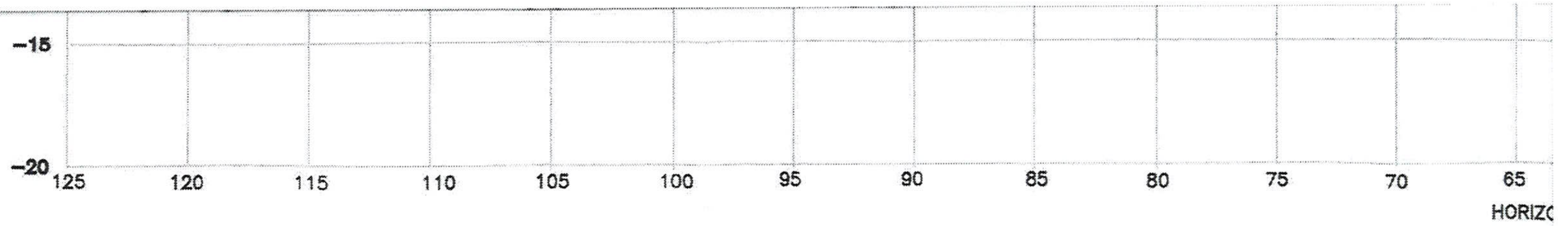
SOUTHEAST FACE

N70E





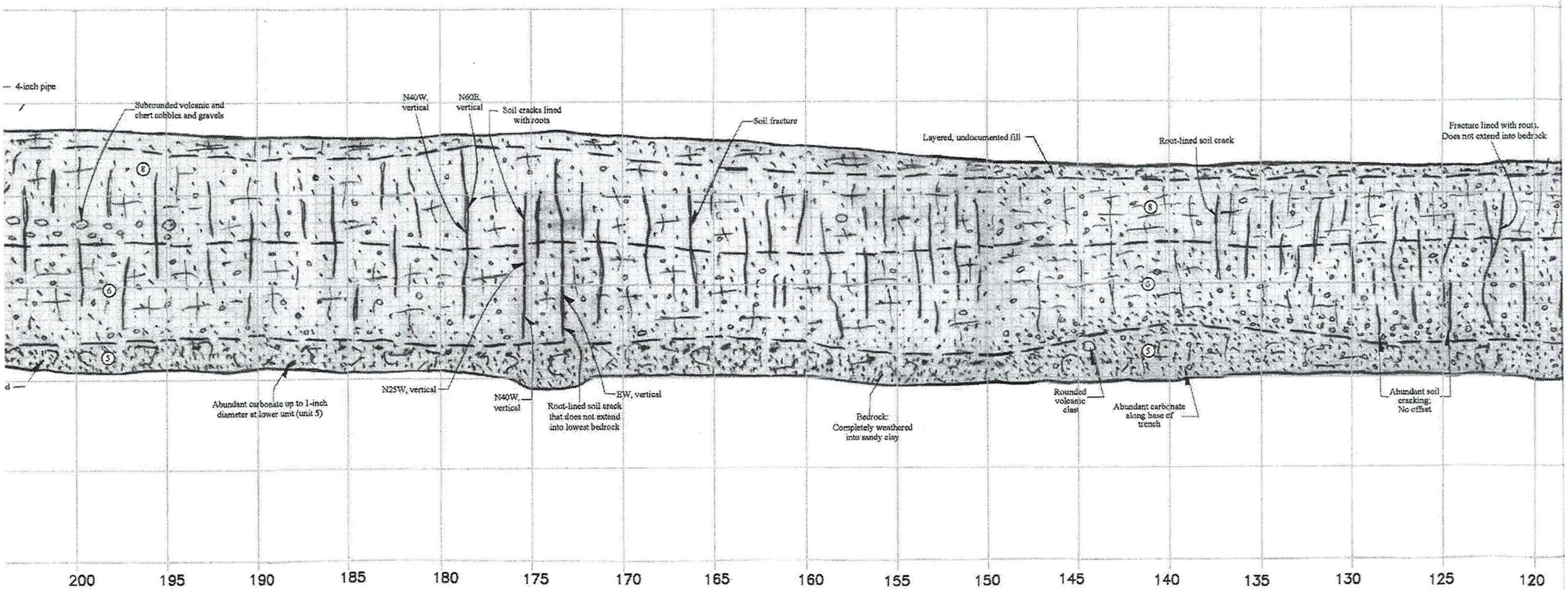
TRENCH T-2
NE
Overlap with Trench T-3
DEPTH (FEET)
+10
+5
0
-5
-10
-15
-20
215
210
205
200
195
190
185
180
175
170
165
160
155
150
145
140
125
120
115
110
105
100
95
90
Soil Fractures
Undocumented fill
4-inch pipe
Subrounded volcanic and chert cobbles and gravels
N40W, vertical
N60E, vertical
Soil cracks lined with roots
Soil fracture
Layered, undocumented fill
Bedrock: Completely weathered into sandy clay
Abundant carbonate up to 1-inch diameter at lower unit (unit 5)
N25W, vertical
N40W, vertical
Root-lined soil crack that does not extend into lowest bedrock
EW, vertical
Bedrock: Completely weathered into sandy clay
Rounded volcanic clast

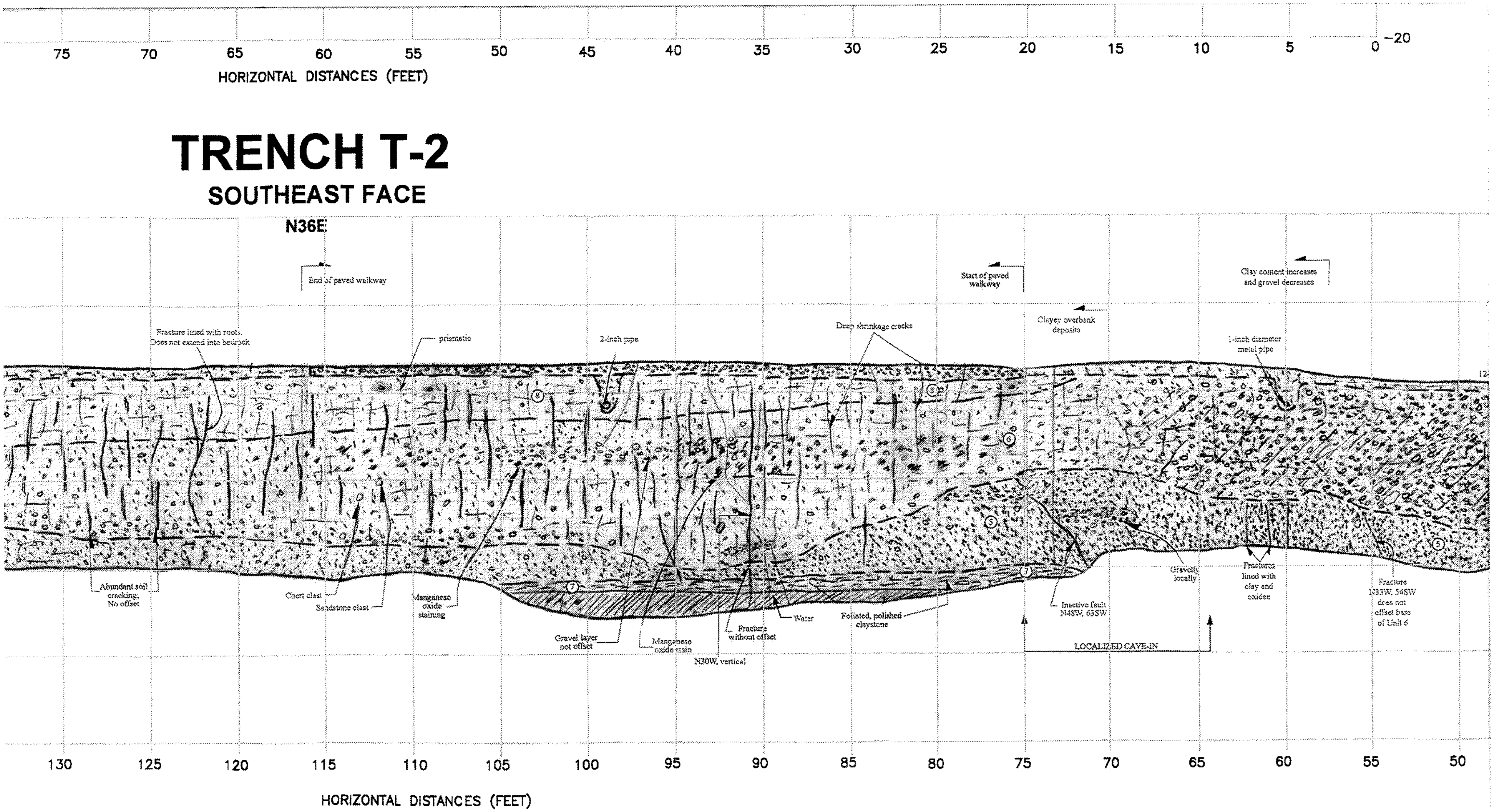


TRE

SOU

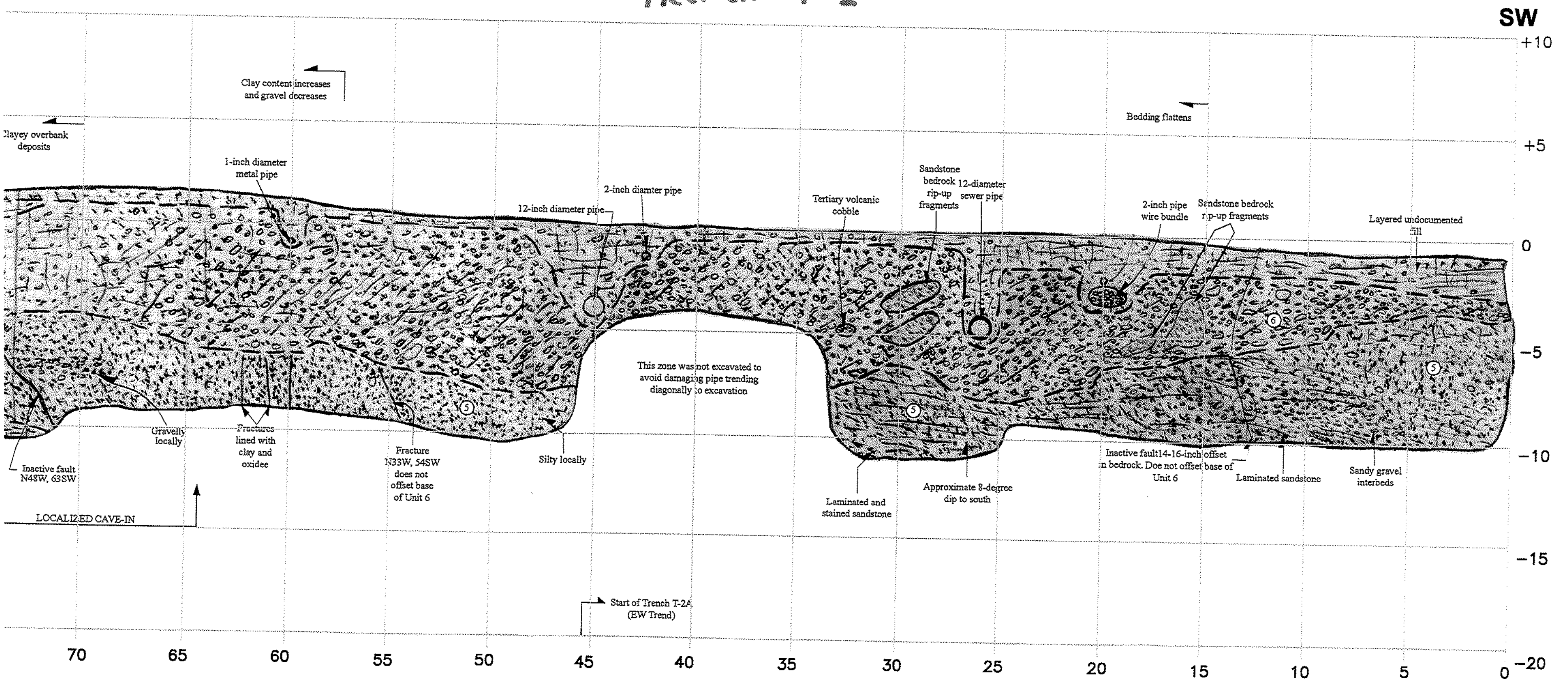
TRENCH T-2





75 70 65 60 55 50 45 40 35 30 25 20 15 10 5 0 −20
HORIZONTAL DISTANCES (FEET)
TRENCH T-2
SOUTHEAST FACE
N36E
End of paved walkway
Start of paved walkway
Clay content increases and gravel decreases
Clayey overbank deposits
Fracture lined with roots. Does not extend into bedrock
prismatic
2-inch pipe
Deep shrinkage cracks
1-inch diameter metal pipe
Abundant soil cracking, No offset
Chert clast
Sandstone clast
Manganese oxide staining
Gravel layer not offset
Manganese oxide stain
N30W, vertical
Fracture without offset
Water
Foliated, polished claystone
Inactive fault N48W, 63SW
LOCALIZED CAVE-IN
Gravelly locally
Fractures lined with clay and oxides
Fracture N33W, 54SW does not offset base of Unit 6
130 125 120 115 110 105 100 95 90 85 80 75 70 65 60 55 50
HORIZONTAL DISTANCES (FEET)

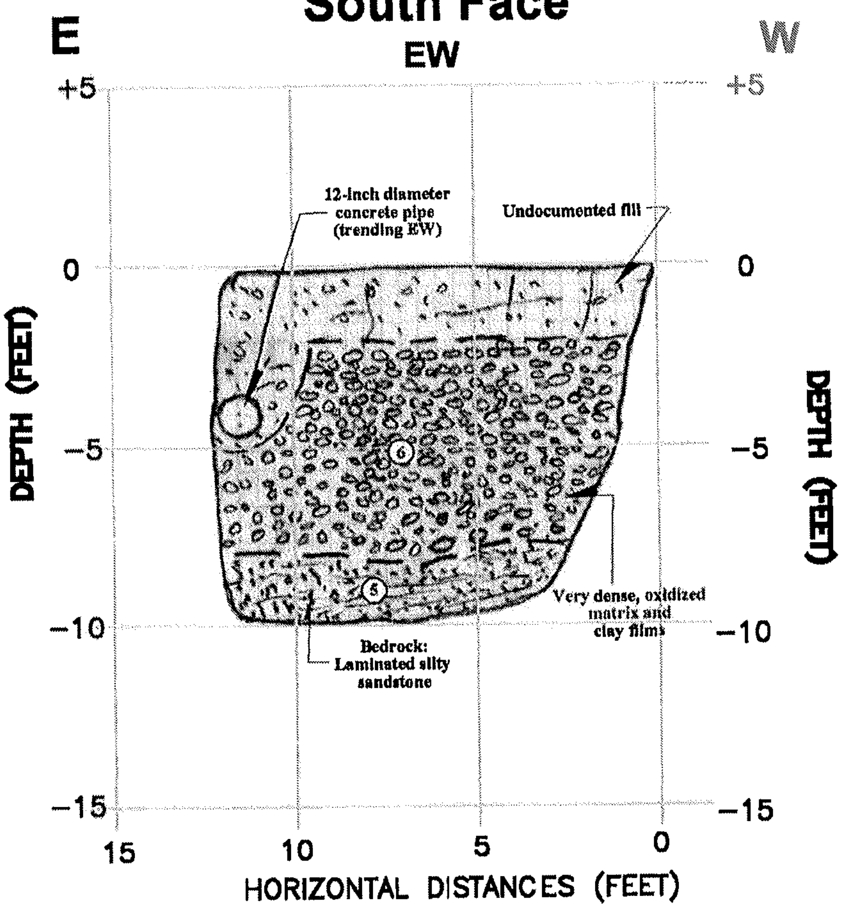
TRENCH T-2



SW
+10
+5
0
-5
-10
-15
-20
DEPTH (FEET)
Clay content increases and gravel decreases
Clayey overbank deposits
Bedding flattens
1-inch diameter metal pipe
2-inch diamter pipe
12-inch diameter pipe
Tertiary volcanic cobble
Sandstone bedrock rip-up fragments
12-diameter sewer pipe
2-inch pipe wire bundle
Sandstone bedrock rip-up fragments
Layered undocumented fill
6
5
This zone was not excavated to avoid damaging pipe trending diagonally to excavation
Inactive fault N48W, 63SW
Gravelly locally
Fractures lined with clay and oxidee
Fracture N33W, 54SW does not offset base of Unit 6
Silty locally
Laminated and stained sandstone
Approximate 8-degree dip to south
Inactive fault14-16-inch offset in bedrock. Doe not offset base of Unit 6
Laminated sandstone
Sandy gravel interbeds
LOCALIZED CAVE-IN
Start of Trench T-2A (EW Trend)
70
65
60
55
50
45
40
35
30
25
20
15
10
5
0

TRENCH T-2A

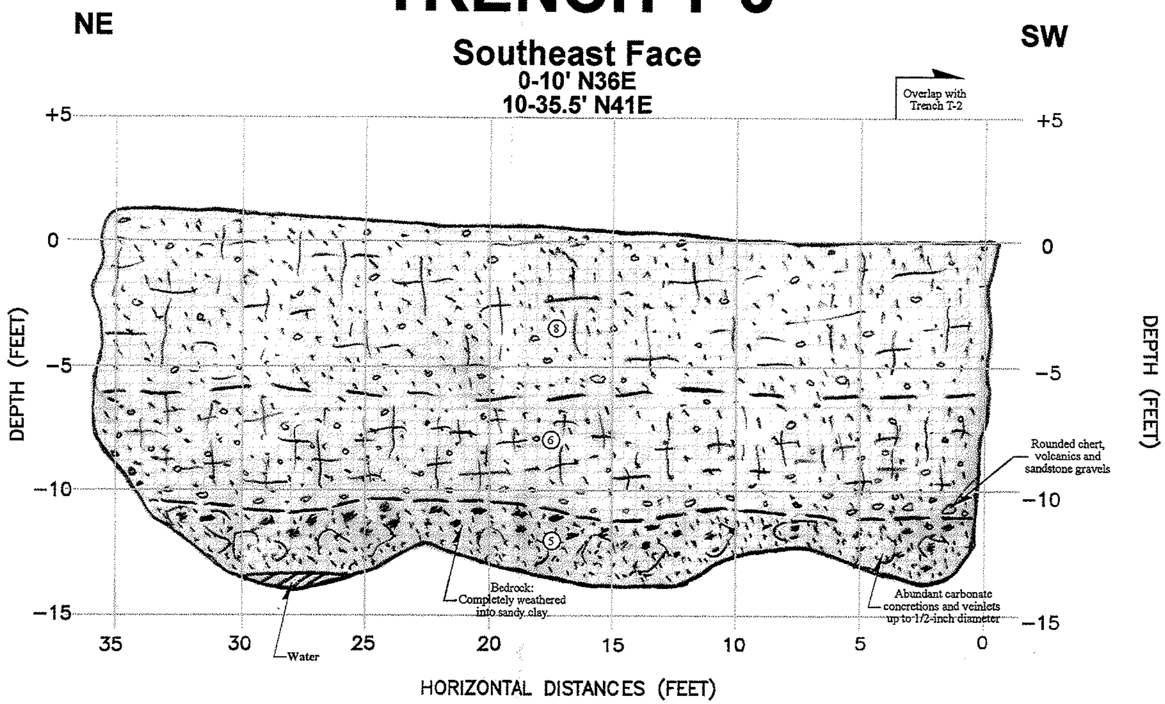
South Face



TRENCH T-3

Southeast Face

0-10' N36E
10-35.5' N41E



DEPTH (FEET)

PLATE NO.	DATE:

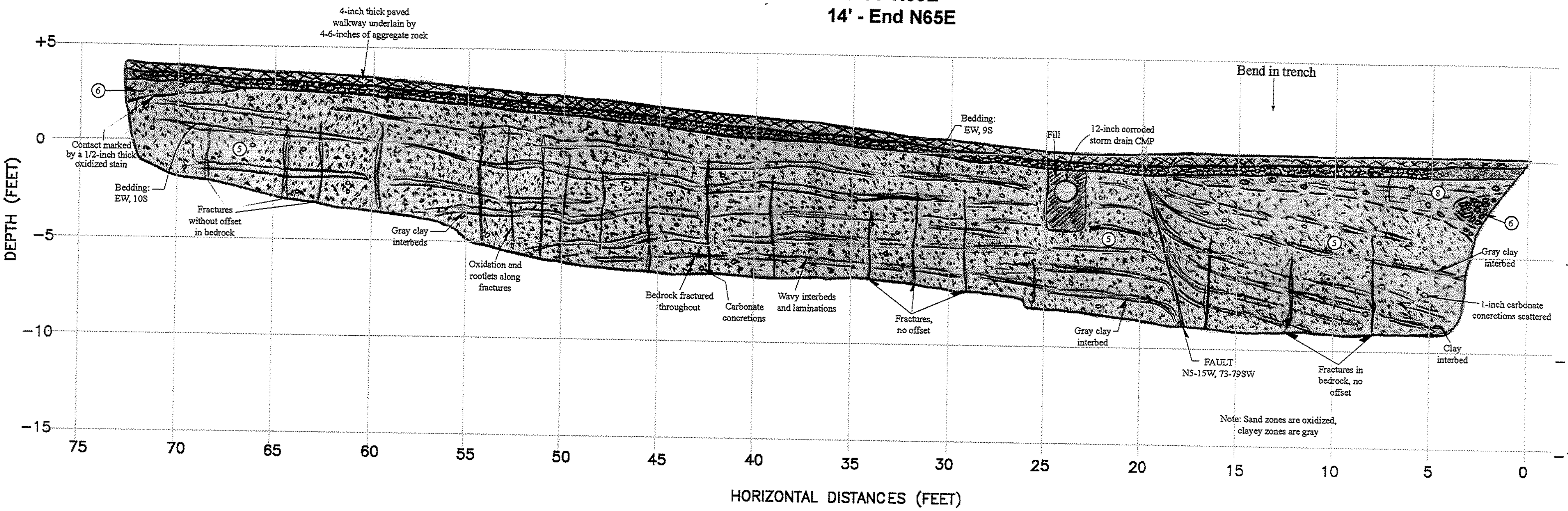
NE

TRENCH T-4

SOUTHEAST FACE

0-14' N86E
14' - End N65E

S\



LOGS OF TRENCH T-1, TRENCH T-2, TRENCH T-2A, TRENCH T-3 AND TRENCH T-4

PROPOSED STUDENT ACTIVITIES BUILDING ADDITION
CONTRA COSTA COMMUNITY COLLEGE
SAN PABLO, CALIFORNIA

Kleinfelder, 2004, Geotechnical Investigation Report, Student Activities Building Addition

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS		LTR	ID	DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY	GW		Well-graded gravels or gravel with sand, little or no fines.
		GP		Poorly-graded gravels or gravel with sand, little or no fines.
		GM		Silty gravels, silty gravel with sand mixture.
		GC		Clayey gravels, clayey gravel with sand mixture
	SAND AND SANDY	SW		Well-graded sands or gravelly sands, little or no fines.
		SP		Poorly-graded sands or gravelly sands, little or no fines.
		SM		Silty sand.
		SC		Clayey sand.

MAJOR DIVISIONS		LTR	ID	DESCRIPTION
FINE GRAINED SOILS	SILTS AND CLAYS	ML		Inorganic silts and very fine sands, rock flour or clayey silts with slight plasticity.
		CL		Inorganic lean clays of low to medium plasticity, gravelly clays, sandy clays, silty clays.
		OL		Organic silts and organic silt-clays of low plasticity.
	SILTS AND CLAYS	MH		Inorganic elastic silts, micaceous or diatomaceous or silty soils.
		CH		Inorganic fat clays (high plasticity).
		OH		Organic clays of medium high to high plasticity.
HIGHLY ORGANIC SOILS		Pt		Peat and other highly organic soils.

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.

▽ 0745, 5/31 Approximate water level first observed in boring. Time recorded in reference to a 24 hour clock.

▼ 0800, 5/31 Approximate water level observed in boring following drilling

PEN	Pocket Penetrometer reading, in tsf
TV:Su	Torvane shear strength, in ksf

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

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.

KLEINFELDER

PROJECT NO. **40698-GEO**

BORING LOG LEGEND

Student Activities Building Addition
Contra Costa College
San Pablo, California

PLATE **B-1**

4/13/04 3:38:39 PM

		Drilling method:	**6" Hollow Stem Auger**
Date Completed:	**2/17/04**		
Logged By:	**C. Melo**	*Hammer Wt:*	**140 lbs., 30" drop**
Total Depth:	**25.0 ft**	*Notes:*	**Drilled in barren landscaping**

Surface Elevation: **Estimated 80 feet (MSL)**

Depth, ft	FIELD: Sample	FIELD: Blows/ft	LABORATORY: Dry Density pcf	LABORATORY: Moisture Content %	LABORATORY: Compress. Strength tsf	LABORATORY: Other Tests	Pen, tsf	DESCRIPTION
								SANDY LEAN CLAY with SILT (CL) - dark brown mottled with gray-brown, low plasticity, fine grained sand, mottled, rootlets throughout, moist to very moist (Fill)
		34					2.8-3.7	**CLAYSTONE/SILTSTONE** - yellow-brown mottled with gray, completely weathered, friable, low to medium plasticity, fine grained sand, rootlets within, moist
5		75					>4.5	- gray with gray-brown mottling, calcium carbonate within, trace fine grained sand, slightly moist to moist
		54					3.0-4.5	
10			118	11		Direct Shear on Remold (see Plate C-2)		
		85/11"					>4.5	- yellow-brown, higher sand content
15			117	12				- gray with gray-brown mottling
		50/3"						- no recovery
20								
		43				LL=35; PI=21 (see Plate C-1)		- completely to highly weathered
25								Boring terminated at 25 feet. No free water observed. Boring backfilled with grout.
30								

Drilling method: **6" Hollow Stem Auger**

Date Completed: **2/17/04**

Logged By: **C. Melo**

Hammer Wt: **140 lbs., 30" drop**

Total Depth: **24.0 ft**

Notes: **Drilled in lawn**

Depth, ft	FIELD Sample	FIELD Blows/ft	LABORATORY Dry Density pcf	LABORATORY Moisture Content %	LABORATORY Compress. Strength tsf	LABORATORY Other Tests	Pen, tsf	DESCRIPTION (Surface Elevation: **Estimated 75 feet (MSL)**)
0								**CLAYEY SILT with SAND (ML)** - dark brown, mottled, low plasticity, fine grained sand, mottled, high organic content, very moist to wet (Fill)
		22					2.0	**SANDY LEAN CLAY (CL)** - yellow-brown with gray mottling, low to medium plasticity, stiff to very stiff, fine grained sand lenses, up to 1/4-inch subangular gravel, very moist
5		19					1.0-2.0	- stiff, high sand content, calcium carbonate within
		12					1.3-1.5	- rootlets within, black inclusions within
10								- gray-brown with yellow-brown mottling, low plasticity, fine grained sand, up to 1-inch subrounded gravel, black inclusions within, iron-oxide, moist to very moist
		17					1.3-1.6	
15			107	21		Passing -#200=61%		
								CLAYSTONE/SILTSTONE -olive-brown and gray mottled, completely to highly weathered, friable, medium plasticity, trace fine grained sand, slightly moist to moist
		82/11"					>4.5	
20								
		50/6"					>4.5	
25								Boring terminated at 24 feet. No free water observed. Boring backfilled with grout.
30								

KLEINFELDER

PROJECT NO. 40698-GEO

LOG OF BORING NO. K-2

Student Activities Building Addition
Contra Costa College
San Pablo, California

PLATE **B-3**

L:\2004\04PROJECTS\40698\40698.GPJ

Date Completed:	2/17/04	Drilling method:	6" Hollow Stem Auger
Logged By:	C. Melo	Hammer Wt:	140 lbs., 30" drop
Total Depth:	15.0 ft	Notes:	Drilled in Amphitheatre Concrete Slab

Surface Elevation: **Estimated 65 feet (MSL)**

Depth, ft	FIELD: Sample	FIELD: Blows/ft	LABORATORY: Dry Density pcf	LABORATORY: Moisture Content %	LABORATORY: Compress. Strength tsf	LABORATORY: Other Tests	Pen, tsf	DESCRIPTION
								CONCRETE - approximately 5 inches thick
								CLAYEY GRAVEL (GC) - approximately 2 inches thick
		27					2.0	**SANDY LEAN CLAY (CL)** - yellow-brown, low plasticity, very stiff, fine grained sand, moist to very moist
						LL=40; PI=26 (see Plate C-1)	2.0-4.0	**SILTY LEAN CLAY (CL)** - gray-brown to gray, medium plasticity, very stiff, trace fine grained sand, black inclusions within, moist
5		28	117	20			2.5-3.0	- gray, fine grained sand lenses within, possibly completely weathered claystone/siltstone - 1/4-inch rounded gravel
		46	98	20			3.0-4.5	**CLAYSTONE/SILTSTONE** - red and gray mottled, completely weathered, friable, low to medium plasticity, trace fine grained sand, calcium carbonate within, slightly moist to moist
10								
		43					>4.5	- gray, completely to highly weathered, calcium carbonate
15								Boring terminated at 15 feet. No free water observed. Boring backfilled with soil cuttings and 6 inches of Quikrete.
20								
25								
30								

KLEINFELDER

PROJECT NO. 40698-GEO

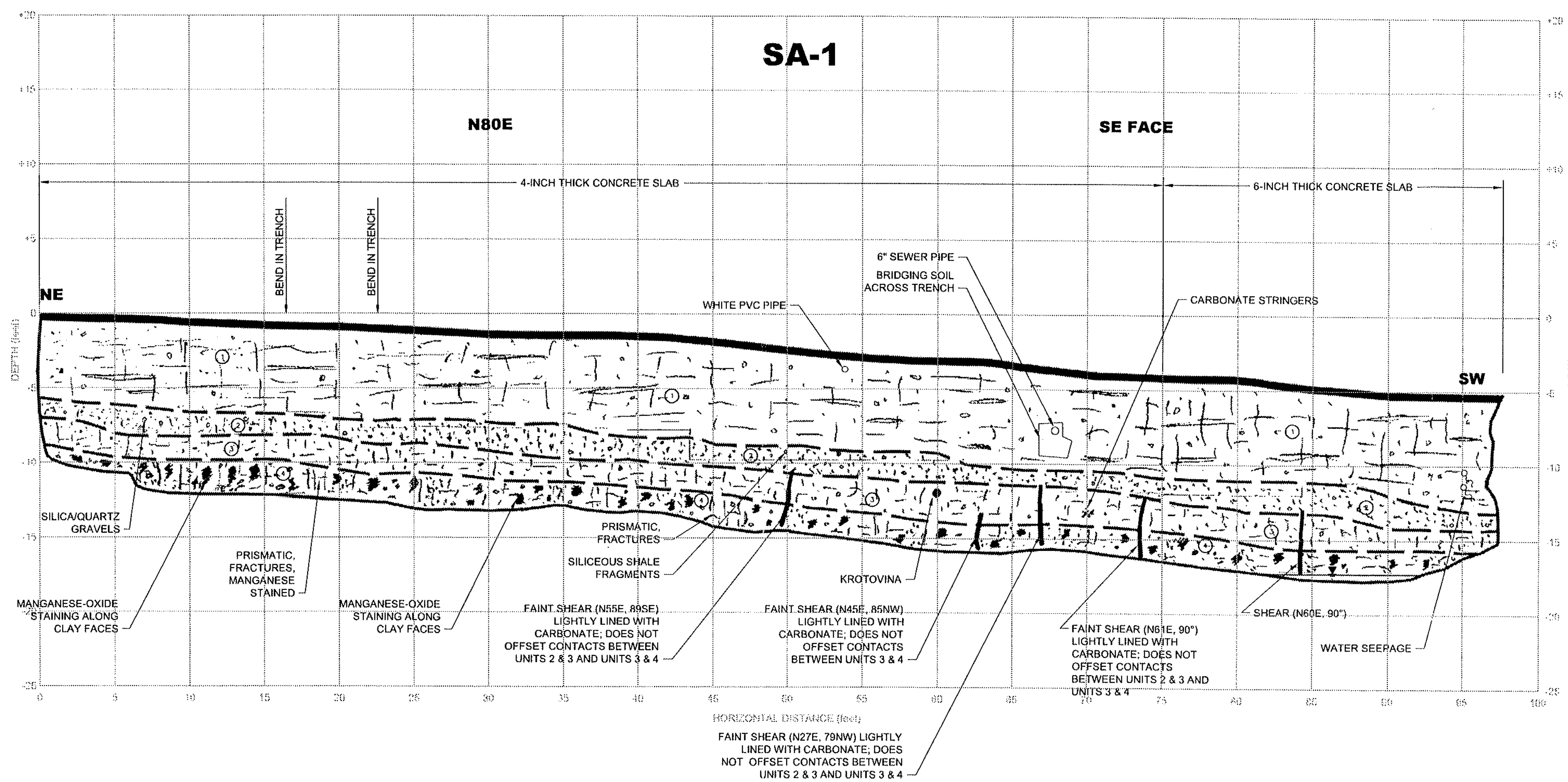
LOG OF BORING NO. K-3

Student Activities Building Addition
Contra Costa College
San Pablo, California

PLATE

B-4

Kleinfelder, 2007, Subsurface Fault Investigation at the Existing Student Activities Building



① **LEAN CLAY (CL)** - yellowish-brown to dark brown, moist, stiff, mottled, layered, trace sand and gravel (FILL)

② **SANDY LEAN CLAY (CL)** - black, moist, with silt, trace gravel (quartz and silica), rootlets (RESIDUAL SOIL)

③ **SANDY CLAYSTONE** - grayish blue to brown, highly to completely weathered, weak to plastic, trace sand and gravel (quartz, silica, volcanics and siliceous shale), subangular to subrounded gravel (UPPER PORTION OF GARRITY MEMBER BEDROCK)

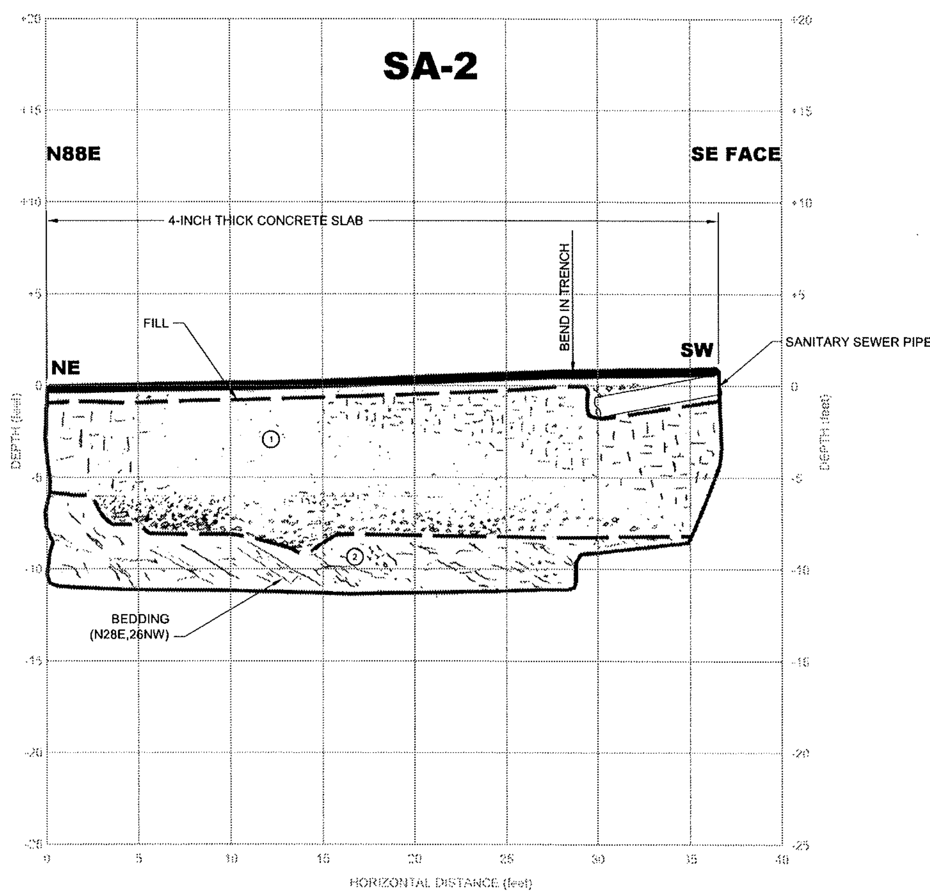
④ **CLAYSTONE** - brown, highly weathered, weak, plastic, sandy locally, prismatic texture, manganese staining along clay faces, slightly mottled due to staining (GARRITY MEMBER - CONTRA COSTA GROUP BEDROCK)

5 2.5 0 5
APPROXIMATE SCALE (feet)

DRAWN BY: LGS	LOG OF TRENCH SA-1	KLEINFELDER	PLATE
REVISED BY:		7133 Koll Center Parkway, Suite 100 Pleasanton, CA 94566-3101 PH. (925) 484-1700 FAX. (925) 484-5838 www.kleinfelder.com	A-1
CHECKED BY: SMD	STUDENT ACTIVITIES BUILDING CONTRA COSTA COMMUNITY COLLEGE SAN PABLO, CALIFORNIA		
DATE: AUGUST 2007 / APPROVED BY:	PROJECT NO. 82074/REPORT / FILE NAME: TRENCHES.dwg		

PLOTTED: 08 Aug 2007, 8:58am, jsala

ATTACHED IMAGES: Images: T-1.jpg Images: T-2.jpg Images: T-3.jpg
ATTACHED XREFS: XRef: Style A_22x34; XRef: Style A_22x34_GYM COMPLEX; XRef: Style A_22x34_STUDENT ACTIVITIES
Pfe-L:\2007\07PROJ CAD FILE: L:\2007\07Projects\82074\GRAPHICS\REPORT\ LAYOUT: T-1



① **SILTY SAND with GRAVELS & COBBLES** - reddish brown,moist, dense, gravel composed of Franciscan complex and Monterey Group clasts (PLEISTOCENE ALLUVIUM)

② **CLAYSTONE** - blue-green, moist, weak to plastic, iron-oxide staining on some surfaces, (GARRITY MEMBER - CONTRA COSTA GROUP BEDROCK)

5 2.5 0 5
APPROXIMATE SCALE (feet)

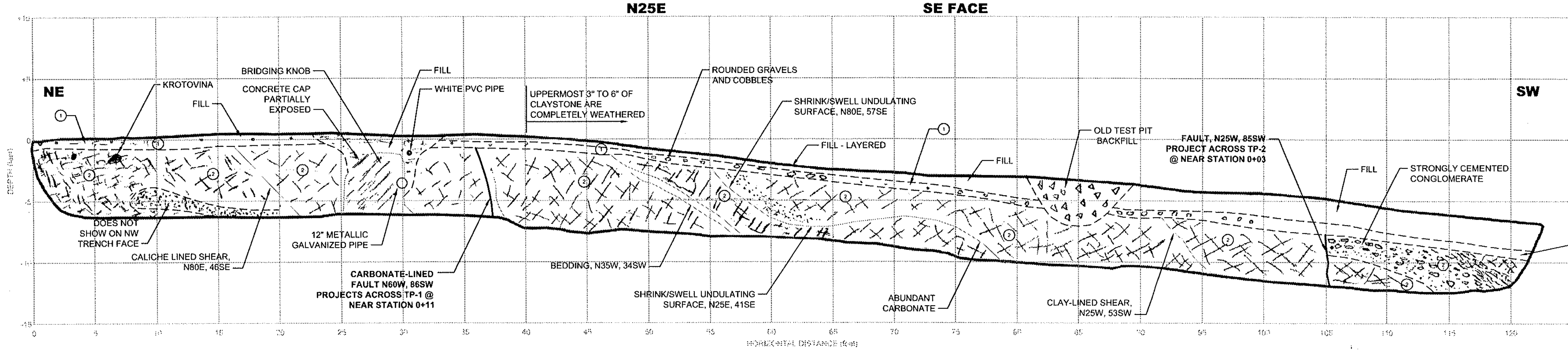
ATTACHED IMAGES: Images: T-1.jpg Images: T-2.jpg Images: T-3.jpg
ATTACHED XREFS: XRef: Style A_22x34; XRef: Style A_22x34_GYM COMPLEX; XRef: Style A_22x34_STUDENT ACTIVITIES
File: L:\2007\07PROJ CAD FILE: L:\2007\07Projects\82074\GRAPHICS\REPORT\ LAYOUT: T-2

DRAWN BY: LGS	LOG OF TRENCH SA-2	KLEINFELDER	PLATE
REVISED BY:		7133 Koll Center Parkway, Suite 100 Pleasanton, CA 94566-3101 PH. (925) 484-1700 FAX. (925) 484-5838 www.kleinfelder.com	A-2
CHECKED BY: SMD	STUDENT ACTIVITIES BUILDING CONTRA COSTA COMMUNITY COLLEGE SAN PABLO, CALIFORNIA		
DATE: AUGUST 2007 APPROVED BY:	PROJECT NO. 82074/REPORT FILE NAME: TRENCHES.dwg		

PLOTTED: 08 Aug 2007, 8:58am, jsala

Kleinfelder, 2008, Subsurface Fault Investigation in Vicinity of the Existing Humanities Building

TRENCH T-1



1. **LEAN CLAY with SAND (CL)** - very dark brown, moist, stiff, subrounded gravel, clay film coats gravels/cobbles, rootlets (Holocene Age Residual Soil)

2. **CLAYSTONE** - olive/gray, closely fractured, friable to weak, highly weathered, interbedded with sandstone, and conglomerate, abundant carbonate staining (Garrity Member - Contra Costa Group Bedrock)

NOTES:
Krotovina: Infilled animal burrow
Logged by : Sadek Derrega & Omar Khan

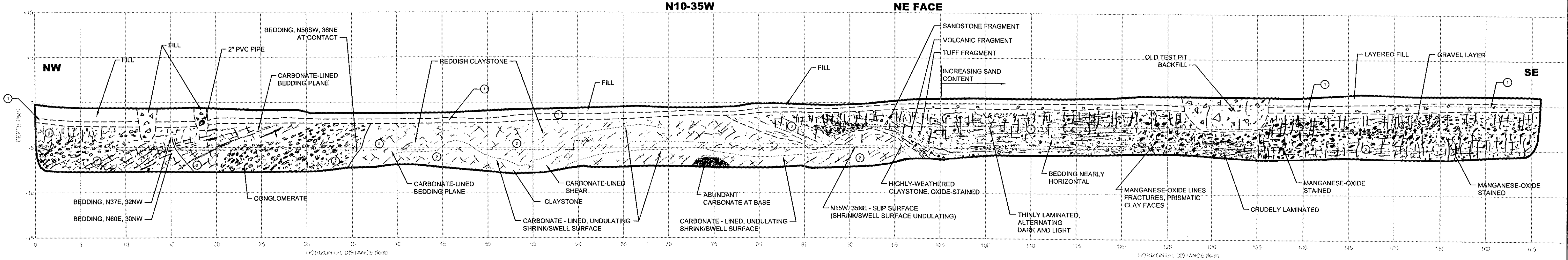
ATTACHED IMAGES: Images: T-1.jpg Images: T-2.jpg Images: TP-1.jpg Images: TP-2.jpg Images: TP-3.jpg Images: TP-4.jpg
ATTACHED XREFS: XRef: Style A_08x11: XRef: Style A_11x36: XRef: Style A_11x30
File-L:\2008\08PROJ CAD FILE: D:_PROJECTS\86352\ LAYOUT: T-1 and T-2

DRAWN BY: LGS/JDS	**LOG OF GEOLOGIC TRENCH T-1**	KLEINFELDER	PLATE
REVISED BY:		7133 Koll Center Parkway, Suite 100 Pleasanton, CA 94566-3101 PH. (925) 484-1700 FAX. (925) 484-5838 www.kleinfelder.com	**A-1**
CHECKED BY: SMD	HUMANITIES BUILDING TRENCHING CONTRA COSTA COLLEGE 2600 MISSION BELL DRIVE SAN PABLO, CALIFORNIA		
DATE: JAN 2008 / APPROVED BY:	PROJECT NO. 86352/REPORT / FILE NAME: TRENCH.dwg		

PLOTTED: 17 Jan 2008, 11:01am, jsala

TRENCH T-2

N10-35W **NE FACE**



① **LEAN CLAY with SAND (CL)** - very dark brown, moist, stiff, subrounded gravel, clay film coats gravels/cobbles, rootlets (Holocene Age Residual Soil)

② **CLAYSTONE** - olive/gray, closely fractured, friable to weak, highly weathered, interbedded with sandstone, and conglomerate, abundant carbonate staining (Garrity Member - Contra Costa Group Bedrock)

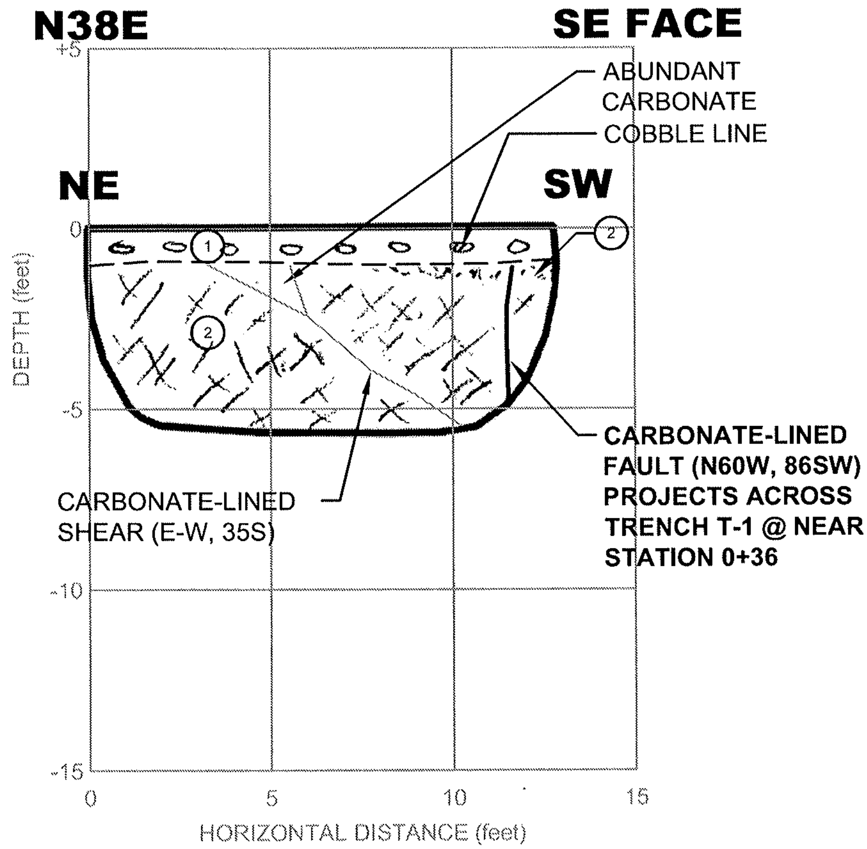
③ **SANDY LEAN CLAY with GRAVEL (CL)** - reddish brown, moist, hard, prismatic near top of unit, subrounded gravels composed of chert, sandstone, basalt, tuff, silicious shale, laminated, oxide stained (Pleistocene Age Channel Deposit)

LOGGED BY: Sadek Derrega & Omar Khan

DRAWN BY: LGS/JDS	LOG OF GEOLOGIC TRENCH T-2	KLEINFELDER	PLATE
REVISED BY:		7133 Koll Center Parkway, Suite 100 Pleasanton, CA 94566-3101	A-2
CHECKED BY: SMD	HUMANITIES BUILDING CONTRA COSTA COLLEGE 2600 MISSION BELL DRIVE SAN PABLO, CALIFORNIA	PH. (925) 484-1700 FAX. (925) 484-5838 www.kleinfelder.com	
DATE: JAN 2008	APPROVED BY:	PROJECT NO. 86352/REPORT	FILE NAME: TRENCH.dwg	

PLOTTED: 17 Jan 2008, 10:58am, jsala

TEST PIT TP-1



1. **LEAN CLAY with SAND (CL) -** very dark brown, moist, stiff, subrounded gravel, clay film coats gravels/cobbles, rootlets (Holocene Age Residual Soil)

2. **CLAYSTONE -** olive/gray, closely fractured, friable to weak, highly weathered, interbedded with sandstone, and conglomerate, abundant carbonate staining (Garrity Member - Contra Costa Group Bedrock)

LOGGED BY: Sadek Derrega & Omar Khan

KLEINFELDER
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PH. (925) 484-1700 FAX. (925) 484-5838
www.kleinfelder.com

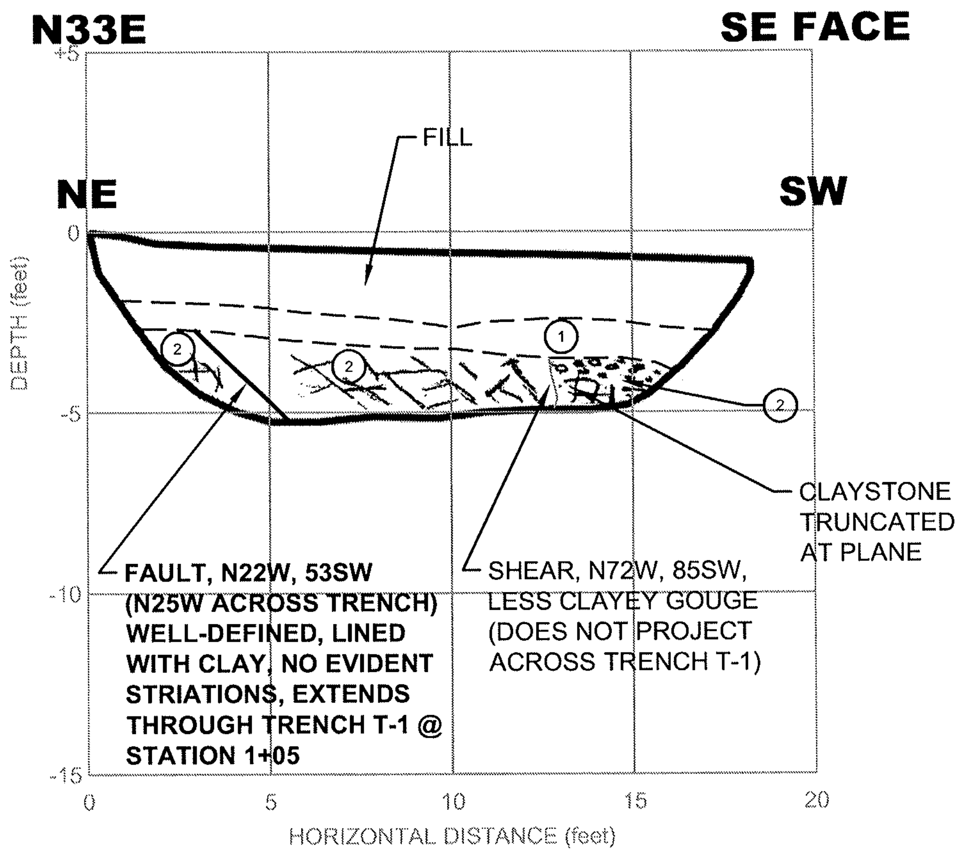
LOG OF GEOLOGIC TEST PIT TP-1

HUMANITIES BUILDING
CONTRA COSTA COMMUNITY COLLEGE
2900 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA

DRAWN BY:	LGS/JDS
REVISED BY:	
CHECKED BY:	SMD
PLATE	**A-3**

DRAWN: JAN 2008 | APPROVED BY: | PROJECT NO. 86352 | FILE NAME: TRENCH.dwg

TEST PIT TP-2



1 **LEAN CLAY with SAND (CL) -** very dark brown, moist, stiff, subrounded gravel, clay film coats gravels/cobbles, rootlets (Holocene Age Residual Soil)

2 **CLAYSTONE -** olive/gray, closely fractured, friable to weak, highly weathered, interbedded with sandstone, and conglomerate, abundant carbonate staining (Garrity Member - Contra Costa Group Bedrock)

ATTACHED IMAGES: Images: T-1.jpg Images: T-2.jpg Images: TP-1.jpg Images: TP-2.jpg Images: TP-3.jpg Images: TP-4.jpg
ATTACHED XREFS: XRef: Style A_08x11: XRef: Style A_11x36: XRef: Style A_11x30
L:\2008\08PROJ CAD FILE: D:_PROJECTS\86352\ LAYOUT: TP-2

KLEINFELDER

7133 Koll Center Parkway, Suite 100
Pleasanton, CA 94566
PH. (925) 484-1700 FAX. (925) 484-5838
www.kleinfelder.com

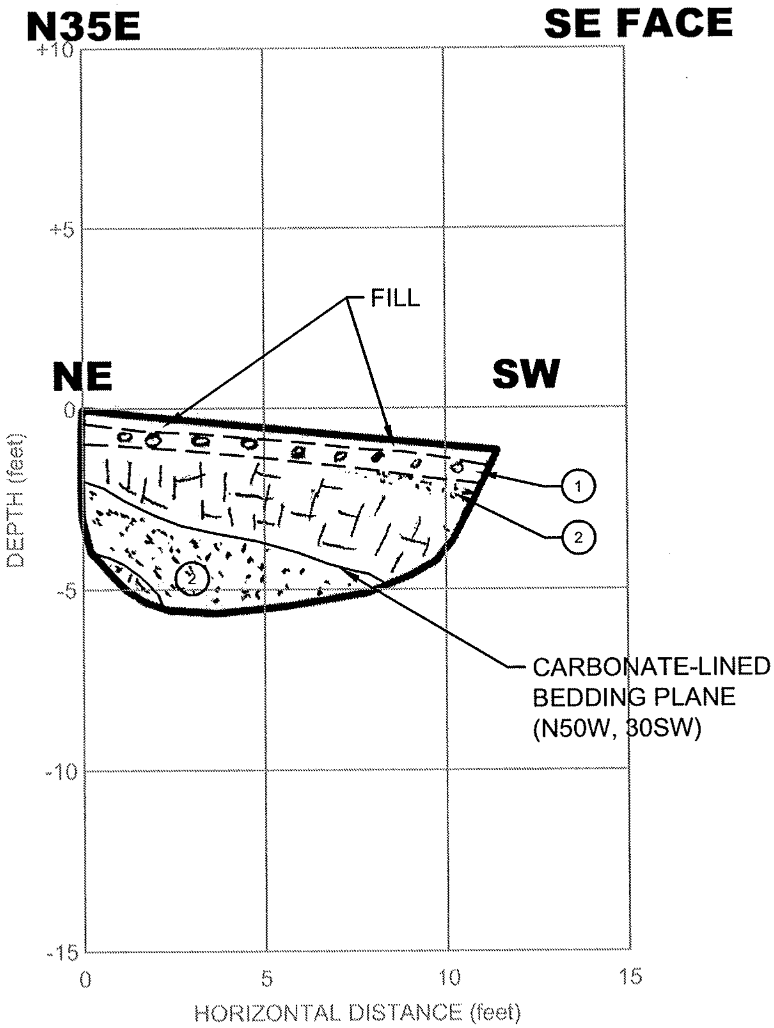
LOG OF GEOLOGIC TEST PIT TP-2

HUMANITIES BUILDING
CONTRA COSTA COMMUNITY COLLEGE
2900 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA

DRAWN BY:	LGS/JDS
REVISED BY:	
CHECKED BY:	SMD
PLATE	**A-4**

DRAWN: JAN 2008 | **APPROVED BY:** | **PROJECT NO.** 86352 | **FILE NAME:** TRENCH.dwg

TEST PIT TP-3



1. **LEAN CLAY with SAND (CL) -** very dark brown, moist, stiff, subrounded gravel, clay film coats gravels/cobbles, rootlets (Holocene Age Residual Soil)

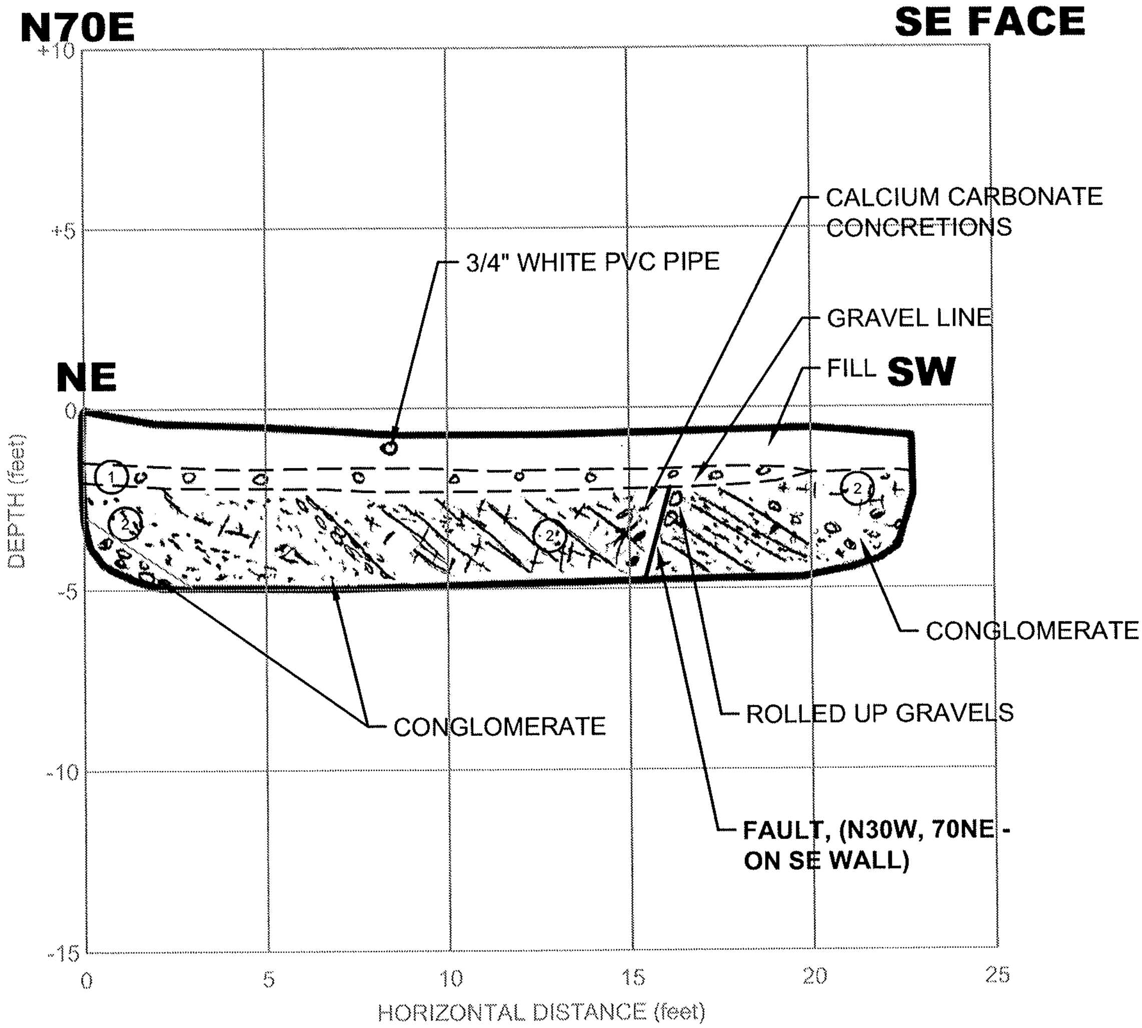
2. **CLAYSTONE -** olive/gray, closely fractured, friable to weak, highly weathered, interbedded with sandstone, and conglomerate, abundant carbonate staining (Garrity Member - Contra Costa Group Bedrock)

LOGGED BY: Sadek Derrega & Omar Khan

KLEINFELDER 7133 Koll Center Parkway, Suite 100 Pleasanton, CA 94566 PH. (925) 484-1700 FAX. (925) 484-5838 www.kleinfelder.com	**LOG OF GEOLOGIC TEST PIT TP-3** HUMANITIES BUILDING CONTRA COSTA COMMUNITY COLLEGE 2900 MISSION BELL DRIVE SAN PABLO, CALIFORNIA	DRAWN BY: LGS/JDS REVISED BY: CHECKED BY: SMD PLATE **A-5**
DRAWN: JAN 2008 APPROVED BY:	PROJECT NO. 86352 FILE NAME: TRENCH.dwg	

ATTACHED IMAGES: Images: T-1.jpg Images: T-2.jpg Images: TP-1.jpg Images: TP-2.jpg Images: TP-3.jpg Images: TP-4.jpg
ATTACHED XREFS: XRef: Style A_08x11: XRef: Style A_11x36: XRef: Style A_11x30
L:\2008\08PROJ CAD FILE: D:_PROJECTS\86352\ LAYOUT: TP-3

TEST PIT TP-4



1. **LEAN CLAY with SAND (CL) -** very dark brown, moist, stiff, subrounded gravel, clay film coats gravels/cobbles, rootlets (Holocene Age Residual Soil)

2. **CLAYSTONE -** olive/gray, closely fractured, friable to weak, highly weathered, interbedded with sandstone, and conglomerate, abundant carbonate staining (Garrity Member - Contra Costa Group Bedrock)

LOGGED BY: Sadek Derrega & Omar Khan

ATTACHED IMAGES: Images: T-1.jpg Images: T-2.jpg Images: TP-1.jpg Images: TP-2.jpg Images: TP-3.jpg Images: TP-4.jpg
ATTACHED XREFS: XRef: Style A_08x11: XRef: Style A_11x36: XRef: Style A_11x30
L:\2008\08PROJ CAD FILE: D:_PROJECTS\86352\ LAYOUT: TP-4

KLEINFELDER 7133 Koll Center Parkway, Suite 100 Pleasanton, CA 94566 PH. (925) 484-1700 FAX. (925) 484-5838 www.kleinfelder.com		**LOG OF GEOLOGIC TEST PIT TP-4** HUMANITIES BUILDING CONTRA COSTA COMMUNITY COLLEGE 2900 MISSION BELL DRIVE SAN PABLO, CALIFORNIA	DRAWN BY: LGS/JDS REVISED BY: CHECKED BY: SMD PLATE **A-6**
DRAWN: JAN 2008	APPROVED BY:	PROJECT NO. 86352 FILE NAME: TRENCH.dwg	

Kleinfelder, 2011, Geotechnical Investigation Report, Campus Center

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS		LTR	ID	DESCRIPTION
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY	GW		Well-graded gravels or gravel with sand, little or no fines.
		GP		Poorly-graded gravels or gravel with sand, little or no fines.
		GM		Silty gravels, silty gravel with sand mixture.
		GC		Clayey gravels, clayey gravel with sand mixture
	SAND AND SANDY	SW		Well-graded sands or gravelly sands, little or no fines.
		SP		Poorly-graded sands or gravelly sands, little or no fines.
		SM		Silty sand.
		SC		Clayey sand.
FINE GRAINED SOILS	SILTS AND CLAYS	ML		Inorganic silts and very fine sands, rock flour or clayey silts with slight plasticity.
		CL		Inorganic lean clays of low to medium plasticity, gravelly clays, sandy clays, silty clays.
		OL		Organic silts and organic silt-clays of low plasticity.
	SILTS AND CLAYS	MH		Inorganic elastic silts, micaceous or diatomaceous or silty soils.
		CH		Inorganic fat clays (high plasticity).
		OH		Organic clays of medium high to high plasticity.
HIGHLY ORGANIC SOILS		Pt		Peat and other highly organic soils.

Physical Properties Criteria for Rock Descriptions

FRACTURE SPACING

VERY WIDELY FRACTURED	Greater than 6.0 feet
WIDELY FRACTURED	2.0 to 6.0 feet
MODERATELY FRACTURED	8.0 inches to 2.0 feet
CLOSELY FRACTURED	2.5 to 8.0 inches
INTENSELY FRACTURED	0.75 to 2.5 inches
CRUSHED	Less than 0.75 inches

BEDDING OR LAYERING

VERY THICK OR MASSIVE	Greater than 4.0 feet
THICK	2.0 to 4.0 feet
THIN	0.2 to 2.0 feet
VERY THIN	0.05 to 0.2 feet
LAMINATED	0.01 to 0.05 feet
THINLY LAMINATED	Less than 0.01 feet

WEATHERING

FRESH - No visible sign of rock material weathering; perhaps slight discoloration on major discontinuity surfaces.

SLIGHTLY WEATHERED - Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discolored by weathering and may be somewhat weaker than in its fresh condition.

MODERATELY WEATHERED - Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a discontinuous framework or as corestones.

HIGHLY WEATHERED - More than half of the rock material is decomposed and/or distintegrated to a soil. Fresh or discolored rock is present either as a discontinuous framework or as corestones.

COMPLETELY WEATHERED - All rock material is decomposed and/or disintegrated to a soil. The original mass structure is still largely intact.

STRENGTH

PLASTIC - Can be remolded with hands.

FRIABLE - Can be crumbled between fingers or peeled by pocket knife.

WEAK - Can be peeled by a knife with difficulty, shallow indentations made by firm blow with point of geological hammer.

MEDIUM STRONG - Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single firm blow of geological hammer.

STRONG - Specimen requires more than one blow of geological hammer to fracture it.

VERY STRONG - Specimen withstands several blows of geological hammer without breaking.

EXTREMELY STRONG - Specimen can only be chipped with a geological hammer.

RQD (Rock Quality Designation) = $\frac{\text{(Length of Solid Core Pieces 4" or Longer)}}{\text{(Total Length of Core Run)}} \times 100$

Key to Test Data

Bulk Sample

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.

Shelby Tube 3.0 inch O.D.

Continuous Rock Core

California Sampler, 3.0 inch O.D., 2.5 inch I.D.

Pitcher Barrel

101 Method (Modified Pitcher Barrel)

0745, 5/31 — Approximate water level first observed in boring. Time recorded in reference to a 24-hour clock.

0800, 5/31 — Approximate water level observed in boring following drilling. Time recorded in reference to a 24-hour clock.

PEN Pocket Penetrometer reading, in tsf

TV:Su Torvane shear strength, in ksf

LL	LIQUID LIMIT
PI	PLASTICITY INDEX
%-#200	SIEVE ANALYSIS (MINUS #200 SCREEN)
R-Value	RESISTANCE VALUE
SE	SAND EQUIVALENT
C	COHESION (PSF)
PHI	FRICTION ANGLE
TX	TRIAXIAL SHEAR
CONSOL	CONSOLIDATION
DS	DIRECT SHEAR

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.

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PROJECT NO. **112252**

ROCK AND SOIL LEGEND

CONTRA COSTA COLLEGE
CAMPUS CENTER
SAN PABLO, CALIFORNIA

PLATE

B-1

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GRAPHIC ROCK SYMBOLS

	SHALE OR CLAYSTONE		CHERT		SERPENTINITE
	SILTSTONE		PYROCLASTIC		METAMORPHIC ROCKS
	SANDSTONE		VOLCANIC FLOWS		
	CONGLOMERATE		PLUTONIC		SHEARED ROCKS

WEATHERING INDEX

W1 - *FRESH - No visible sign of rock material weathering; perhaps slight discoloration on major discontinuity surfaces.*

W2 - *SLIGHTLY WEATHERED - Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discolored by weathering and may be somewhat weaker than in its fresh condition.*

W3 - *MODERATELY WEATHERED - Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a discontinuous framework or as corestones.*

W4 - *HIGHLY WEATHERED - More than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a discontinuous framework or as corestones.*

W5 - *COMPLETELY WEATHERED - All rock material is decomposed and/or disintegrated to a soil. The original mass structure is still largely intact.*

STRENGTH INDEX

R0 - *EXTREMELY WEAK - Indented by thumbnail*

R1 - *VERY WEAK - Crumbles under firm blows with a point of geological hammer, can be peeled by pocket knife*

R2 - *WEAK - Can be peeled by a knife with difficulty, shallow indentations made by firm blow with point of geological hammer.*

R3 - *MEDIUM STRONG - Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single firm blow of geological hammer.*

R4 - *STRONG - Specimen requires more than one blow of geological hammer to fracture it.*

R5 - *VERY STRONG - Specimen withstands several blows of geological hammer without breaking.*

R6 - *EXTREMELY STRONG - Specimen can only be chipped with a geological hammer.*

FRACTURE SPACING

VERY WIDELY FRACTURED	*Greater than 6.0 feet*
WIDELY FRACTURED	*2.0 to 6.0 feet*
MODERATELY FRACTURED	*8.0 inches to 2.0 feet*
CLOSELY FRACTURED	*2.5 to 8.0 inches*
INTENSELY FRACTURED	*0.75 to 2.5 inches*
CRUSHED	*Less than 0.75 inches*

BEDDING OR LAYERING

VERY THICK OR MASSIVE	*Greater than 4.0 feet*
THICK	*2.0 to 4.0 feet*
THIN	*0.2 to 2.0 feet*
VERY THIN	*0.05 to 0.2 foot*
LAMINATED	*0.01 to 0.05 foot*
THINLY LAMINATED	*Less than 0.01 foot*

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PROJECT NO. 112252

ROCK CLASSIFICATION SYSTEM

CONTRA COSTA COLLEGE
CAMPUS CENTER
SAN PABLO, CALIFORNIA

PLATE

B-2

8/5/2010 6:10:31 PM

Date Completed: 7/26/10

Logged By: O. Khan

Total Depth: Approximately 14.0 ft

Drilling method: 4" Flight Auger

Driller: Hillside Geotectnical

Hammer Wt: 140 lbs., 30" drop

Notes: Surface Condition: Asphalt

Depth, ft	FIELD: Sample	FIELD: Blows/ft	LABORATORY: Dry Density pcf	LABORATORY: Moisture Content %	LABORATORY: Compress. Strength tsf	LABORATORY: Other Tests	Pen, tsf	DESCRIPTION (Surface Elevation: Estimated ~91 feet (MSL))
								ASPHALT CONCRETE - approximately 1.5 inches thick
		26				Ø= 34 Degrees; C= 100 psf (See Plate D-7)	3.5-4.0	**CLAYSTONE** - light olive green, slightly moist to moist, highly weathered to decomposed, weak, low to medium plasticity (GARRITY MEMBER - Contra Costa Group Bedrock)
5		49	113	14.1			>4.5	- red brown
10		50/6"					>4.5	
		50/6"					>4.5	- light brown
15								Boring terminated at approx. 14 feet below ground surface. No free water observed. Boring backfilled with cement grout and capped with asphalt patching.
20								
25								
30								
35								

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PROJECT NO. 112252

LOG OF BORING NO. B- 1

CONTRA COSTA COLLEGE
CAMPUS CENTER
SAN PABLO, CALIFORNIA

PLATE

B-3

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Date Completed:	7/26/10	*Drilling method:*	4" Flight Auger
			Driller: Hillside Geotectnical
Logged By:	O. Khan		
		Hammer Wt:	140 lbs., 30" drop
Total Depth:	Approximately 30.0 ft	*Notes:*	Surface Condition: Landscape

Surface Elevation: **Estimated ~77 feet (MSL)**

Depth, ft	FIELD Sample	FIELD Blows/ft	LABORATORY Dry Density pcf	LABORATORY Moisture Content %	LABORATORY Compress. Strength tsf	LABORATORY Other Tests	Pen, tsf	DESCRIPTION
		43					>4.0	**LEAN CLAY (CL)** - brown to dark brown, dry to moist, hard, low to medium plasticity, trace fine grained gravel, rootlets (POSSIBLY FILL)
5		49	119	10.8	11.39 @ 6.0%	Corrosivity (See Appendix E)	>4.5	- very hard, increase in fine gravel
								GRAVELLY LEAN CLAY (CL)- dark olive brown, moist, firm, medium plasticity, fine grained gravel
10		35					2.0	
15		18	104	23.5		Consolidation Test (See Plate D-8)	1.0	**LEAN CLAY (CL)** - olive, wet, firm, medium plasticity, trace fine gravel and fine grained sand, mottled with manganese oxide staining
20		39						**POORLY-GRADED SAND with GRAVEL (SP)**- brown, wet, medium dense, medium to coarse grained sand, fine gravel (white quartz, chert)
								SILTY CLAYSTONE - olive, highly weathered, weak to plastic, trace fine grained sand (GARRITY MEMBER - Contra Costa Group Bedrock)
25		28						- moist
30		50-5"						Boring terminated at approx. 30 feet below ground surface. Perched water encountered at apprximately 13.5 feet. Backfilled with cement grout.
35								

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PROJECT NO. 112252

LOG OF BORING NO. B- 2

CONTRA COSTA COLLEGE
CAMPUS CENTER
SAN PABLO, CALIFORNIA

PLATE

B-4

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Date Completed:	**8/2/10**	*Drilling method:*	**4" Solid Stem Auger**
			Driller: Hillside Geotectnical
Logged By:	**O. Khan**		
		Hammer Wt:	**140 lbs., 30" drop**
Total Depth:	**Approximately 20.0 ft**	*Notes:*	**Surface Condition: Asphalt**

Depth, ft	FIELD: Sample	FIELD: Blows/ft	LABORATORY: Dry Density pcf	LABORATORY: Moisture Content %	LABORATORY: Compress. Strength tsf	LABORATORY: Other Tests	Pen, tsf	DESCRIPTION (*Surface Elevation:* **Estimated ~78 feet (MSL)**)
								ASPHALT CONCRETE - approximately 4 inches thick
		29					2.0-3.0	**SANDY LEAN CLAY (CL)** - olive, moist, firm to hard, medium plasticity, fine grained sand
5		28	97	26.3	2.40 @ 12.4%		2.8	**SANDY CLAYSTONE** - olive, weak to plastic, completely to highly weathered, fine grained sand (GARRITY MEMBER - Contra Costa Group Bedrock)
								- carbonate nodules, increase in sand content and gravel
10		40						
								- highly weathered, mottled with fine grained sand lens
15		33						
20		68						Boring terminated at approx. 20 feet below ground surface. No free water observed. Boring backfilled with cement grout and capped with asphalt patching.
25								
30								
35								

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PROJECT NO. 112252

LOG OF BORING NO. B- 5

CONTRA COSTA COLLEGE
CAMPUS CENTER
SAN PABLO, CALIFORNIA

PLATE **B-7**

Date Completed: **8/2/10**

Logged By: **O. Khan**

Total Depth: **Approximately 20.0 ft**

Drilling method: **4" Solid Stem Auger**
Driller: Hillside Geotectnical

Hammer Wt: **140 lbs., 30" drop**

Notes: **Surface Condition: Asphalt**

Depth,ft	FIELD: Sample	FIELD: Blows/ft	LABORATORY: Dry Density pcf	LABORATORY: Moisture Content %	LABORATORY: Compress. Strength tsf	LABORATORY: Other Tests	Pen, tsf	DESCRIPTION (*Surface Elevation:* **Estimated ~74 feet (MSL)**)
								ASPHALT CONCRETE - approximately 2 inches thick
		27					2.0	**LEAN CLAY (CL)** - dark olive brown, moist, firm, medium plasticity, rootlets
			106	13.9				**SANDY LEAN CLAY (CL)** - olive to light yellowish brown, moist, firm, low to medium plasticity, fine to coarse grained sand, trace fine gravel
5		**50**						**CLAYEY GRAVEL with SAND (GC)**- brown, moist, dense, fine angular to sub angular gravel
			93	30.4			1.8	**SANDY LEAN CLAY (CL)** - light olive, moist, firm, low to medium plasticity, fine grained sand, iron oxide staining
10		**33**						**CLAYEY GRAVEL with SAND (GC)**- brown, moist, medium dense, fine to medium grained sand, fine gravel
15		**48**						**SANDY CLAYSTONE** - light yellowish brown, weak, highly weathered, low to medium plasticity, manganese oxide staining, trace silt (GARRITY MEMBER - Contra Costa Group Bedrock)
20		**54**						- fine grained sand lens
								Boring terminated at approx. 20 feet below ground surface. No free water observed. Boring backfilled with cement grout and capped with asphalt patching.
25								
30								
35								

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PROJECT NO. *112252*

LOG OF BORING NO. B- 6

CONTRA COSTA COLLEGE
CAMPUS CENTER
SAN PABLO, CALIFORNIA

PLATE

B-8

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Date Completed: 8/2/10

Logged By: O. Khan

Total Depth: Approximately 19.5 ft

Drilling method: 4" Solid Stem Auger
Driller: Hillside Geotectnical

Hammer Wt: 140 lbs., 30" drop

Notes: Surface Condition: Lawn

Depth, ft	FIELD Sample	FIELD Blows/ft	LABORATORY Dry Density pcf	LABORATORY Moisture Content %	LABORATORY Compress. Strength tsf	LABORATORY Other Tests	Pen, tsf	DESCRIPTION (Surface Elevation: Estimated ~73 feet (MSL))
		14				LL=36; PI=21 (See Plate D-1) Corrosivity (See Appendix E)	1.0	**LEAN CLAY (CL)** - dark brown, moist, firm, mottled, low to medium plasticity, trace fine grained sand, fine subangular gravel, rootlets (FILL) - soft
5		14	95	27.6	0.99 @ 15.0%		0.5	**LEAN CLAY (CL)** - greenish gray, moist, firm, medium plasticity, trace fine gravel
10		24					1.0	**SANDY LEAN CLAY (CL)**- light yellowish brown, moist, firm, low to medium plasticity, fine grained sand, manganese oxide staining, higher sand content with depth
15		39	110	17.1				**GRAVELLY LEAN CLAY (CL)**- dark yellowish brown, moist, hard, low plasticity, iron oxide staining, fine subrounded to rounded gravel (chert and quartz) - increase in gravel and sand content
		50/4"						
20								Boring terminated at approx. 19.5 feet below ground surface. No free water observed. Boring backfilled with cement grout.
25								
30								
35								

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PROJECT NO. 112252

LOG OF BORING NO. B- 7

CONTRA COSTA COLLEGE
CAMPUS CENTER
SAN PABLO, CALIFORNIA

PLATE

B-9

Date Completed: **8/2/10**

Logged By: **O. Khan**

Total Depth: **Approximately 20.0 ft**

Drilling method: **4" Solid Stem Auger**

Driller: Hillside Geotectnical

Hammer Wt: **140 lbs., 30" drop**

Notes: **Surface Condition: Asphalt**

Surface Elevation: **Estimated ~65 feet (MSL)**

Depth, ft	FIELD: Sample	FIELD: Blows/ft	LABORATORY: Dry Density pcf	LABORATORY: Moisture Content %	LABORATORY: Compress. Strength tsf	LABORATORY: Other Tests	Pen, tsf	DESCRIPTION
0–0.5								**ASPHALT CONCRETE** - approximately 6 inches thick
0.5–1								**GRAVELLY MATERIAL** - approximately 6 inches thick (Possibly Aggregate Base)
		18					1.5-1.8	
5		20	96	30.0			1.3-1.8	**SANDY LEAN CLAY (CL)** - dark gray to yellowish brown, firm, low to medium plasticity, fine grained sand, trace fine gravel (FILL)
								LEAN CLAY (CL) - light gray, moist, firm, medium plasticity
10		18	103	22.3	1.61 @ 15.0%		1.5-2.0	- grayish brown, soft to firm, trace fine grained sand
15		39					2.5	- dark yellowish brown, firm to hard, manganese oxide staining
18			14:10 (groundwater)					**SILTY SAND (SM)** - yellowish brown, wet, medium dense to dense, fine grained sand
20		43				Passing -#200=23%		- angular coarse gravel (Volcanic Clasts)
								Boring terminated at approx. 20 feet below ground surface. Boring backfilled with cement grout and capped with asphalt patching.

Depth scale: 5, 10, 15, 20, 25, 30, 35

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PROJECT NO. 112252

LOG OF BORING NO. B- 8

CONTRA COSTA COLLEGE
CAMPUS CENTER
SAN PABLO, CALIFORNIA

PLATE

B-10

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APPENDIX D

Corrosion Results

California State Certified Laboratory No. 2153

CERCO analytical
1100 Willow Pass Court, Suite A
Concord, CA 94520-1006
925 462 2771 Fax. 925 462 2775
www.cercoanalytical.com

Client: Kleinfelder
Client's Project No.: 20181569
Client's Project Name: Contra Costa College-New Allied Science Bldg. (C-4016)
Date Sampled: 08/11 & 18/17
Date Received: 8-Sep-2017
Matrix: Soil
Authorization: Signed Chain of Custody

Date of Report: 21-Sep-2017

Job/Sample No.	Sample I.D.	Redox (mV)	pH	Resistivity (As Received) (ohms-cm)	Resistivity (100% Saturation) (ohms-cm)	Sulfide (mg/kg)*	Chloride (mg/kg)*	Sulfate (mg/kg)*
1709047-001	B-3 2C @ 6'	+440	7.86	720	1,100	N.D.	N.D.	N.D.

Method:	ASTM D1498	ASTM D4972	ASTM G57	ASTM G57	ASTM D4658M	ASTM D4327	ASTM D4327
Reporting Limit:	-	-	-	-	50	15	15
Date Analyzed:	14-Sep-2017	14-Sep-2017	13-Sep-2017	13-Sep-2017	20-Sep-2017	14-Sep-2017	14-Sep-2017

* Results Reported on "As Received" Basis
N.D. - None Detected

Cheryl McMillen
Laboratory Director

Quality Control Summary - All laboratory quality control parameters were found to be within established limits

APPENDIX E

Site-Specific Seismic Analysis

APPENDIX E
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 New Science Building 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 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 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, southwest of this site. However, the subsurface soil conditions are not similar at these two sites. The Campus Safety Center is located over relatively thick alluvium, whereas, this site has relatively shallow bedrock. Therefore, we had to perform PSHA and DSHA for this site and could not use the results from previous studies.

PROJECT LOCATION

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

Latitude: 37.9697° N
Longitude: -122.3369° 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. Figure E-1 shows the known faults within 100 km of 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 E-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 E-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 E-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 E-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 E-1: SIGNIFICANT FAULTS IN THE SEISMIC SOURCE MODEL

Fault Name	Closest Distance* (km)	Fault Length (km)	Magnitude of Characteristic Earthquake **	Slip Rate (mm/yr)
Hayward-Rodgers Creek	0	150	7.33	9.0
West Napa	23	30	6.70	1.0
Green Valley Connected	25	56	6.80	4.7
Northern San Andreas	28	473	8.05	17-24
Mount Diablo Thrust	29	25	6.70	2.0
San Gregorio - Connected	33	176	7.50	5.5
Calaveras	34	123	7.03	6-15
Great Valley 4b, Gordon Valley	39	28	6.80	1.3
Point Reyes	42	47	6.90	0.4
Great Valley 5, Pittsburg Kirby Hills	44	32	6.70	1.0
Greenville Connected	46	51	7.00	2.0
Hunting Creek-Berryessa	55	60	7.10	6.0
Monte Vista-Shannon	60	45	6.50	0.4
Great Valley 4a, Trout Creek	61	19	6.60	1.3
Great Valley 7	74	45	6.90	1.5
Maacama-Garberville	74	221	7.40	9.0
Great Valley 3, Mysterious Ridge	77	55	7.10	1.3
Collayomi	95	28	6.70	0.6

* 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:

$$\mathrm{Log}(N) = a - b*M$$

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 E-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 using appropriate correlations between penetration resistance and Vs and/or undrained shear strength and Vs, the site is estimated to have average shear wave velocity in the upper 100 feet (V_{S30}) varies from about 1,050 feet/sec (320 m/s) in boring B-3 to about 1,475 (450 m/s) in boring B-2, thus making this site as Site Class D (i.e., Stiff soil) on one side to Site Class C (soft rock) on the other based on Table 20.3-1 of ASCE 7-10. Conservatively, we have assumed V_{S30} of 320 m/s for this site, thus making it Site Class D. 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 (return period of about 2,475 years) adjusted for risk factors and for the maximum direction; and (2) greater of 84th percentile (median + 1 standard deviation) deterministic values (adjusted for the maximum direction) from the controlling fault and deterministic lower limit (DLL) of Figure 21.2-1 of ASCE 7-10. The DE is defined as two-thirds of the MCE_R. In addition, for site-specific response spectra, procedures provided in Chapter 21 of ASCE 7-10 should be used and the design spectral accelerations at any period from site-specific analyses should not be less than the 80 percent of the code spectrum based on S_{DS} and S_{D1} values from Chapter 11, ASCE 7-10.

Both probabilistic and deterministic seismic hazard analyses were used to estimate the spectral accelerations for the MCE_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 320 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

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

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

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

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

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

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

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

A simplified mathematical expression for these steps is provided below:

$$\nu\ (Sa > z) = \sum_{i=1}^{Nsource} N_i(M_{\min}) \int_{r=0}^{\infty} \int_{m=M_{\min}}^{M_{\max_i}} f_{m_i}(M) f_{r_i}(r) P(Sa > z \mid M, r) dr dM$$

Where $\nu(Sa>z)$ is the mean annual rate of a spectral acceleration (Sa) exceeding a test value (z); N_{source} is the number of seismic sources; $N_i(M_{min})$ is the rate of earthquakes with magnitude greater than M_{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$_R$ 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 E-2.

TABLE E-2: GENERAL GROUND MOTION PARAMETERS BASED ON ASCE 7-10

PARAMETER	VALUE	ASCE 7-10 REFERENCE
S_S	2.478g	Fig 22-1
S_1	1.030g	Fig 22-2
Site Class	D	Table 20.3-1
F_a	1.00	Table 11.4-1
F_v	1.50	Table 11.4-2
C_{RS}	0.988	Fig 22-3
C_{R1}	0.969	Fig 22-4
S_{MS}	2.478g	Eq. 11.4-1
S_{M1}	1.545g	Eq. 11.4-2
S_{DS}	1.652g	Eq. 11.4-3
S_{D1}	1.030g	Eq. 11.4-4
PGA	0.960	Fig 22-7
F_{pga}	1.00	Table 11.8-1
PGA_M	0.960	Eq. 11-8-1

As discussed earlier, the MCE$_R$ response spectrum is developed by comparing probabilistic, deterministic, DLL, and 80% of the code values. These NGA GMPEs present the spectral accelerations in terms of geometric mean values of the rotated two horizontal ground motions. To estimate both the deterministic and probabilistic the spectral accelerations in the direction of the maximum horizontal response at each period from geometric mean values, we have used the scale factors as used by USGS. To obtain spectral acceleration values in the maximum direction, a factor of 1.1 for periods of 0.2s and less, a factor of 1.3 for period of 1.0s and greater were used.

Linear interpolation was used between 1.1 and 1.3 for periods between 0.2s and 1.0s. In addition, the probabilistic spectrum was adjusted for targeted risk using risk coefficients C_{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 E-2. Spectral values are also compared in Table E-3 for some specific periods. Figure E-2 and Table E-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 E-3: COMPARISON OF SPECTRAL ACCELERATION (G)

Period (s)	Deterministic Max Rot	DLL	Probabilistic Max Rot Risk Adj	DE	80% Code DE
PGA (0.01)	1.030	0.600	1.282	0.687	0.529
0.2	1.891	1.500	2.769	1.387	1.322
0.3	2.298	1.500	2.913	1.532	1.322
0.5	2.398	1.500	2.851	1.599	1.322
1.0	1.942	0.900	2.086	1.295	0.824
2.0	1.104	0.450	1.121	0.736	0.412

Site-specific probabilistic spectrum is compared with the controlling deterministic spectrum on Figure E-3. Spectral values are also compared in Table E-3 for some specific periods. Figure E-3 and Table E-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_R spectrum is developed by enveloping the controlling deterministic and probabilistic spectra. The DE spectrum was developed by taking two-thirds of the MCE_R spectrum. Comparison of the DE spectrum with the 80% of the code spectrum is shown on Figure E-4. Spectral values are also compared in Table E-3 for some specific periods. Figure E-4 and Table E-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 0.15 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_R spectrum and 80% of the code spectrum. Site-specific MCE_R spectrum is taken as 1.5 times the DE spectrum. Figure E-5 shows the site-specific 5% damped DE and MCE_R spectra. Site-specific horizontal spectral acceleration values in terms of g for the DE and MCE_R are presented in Table E-4.

TABLE E-4: SITE-SPECIFIC HORIZONTAL MCE_R AND DE SPECTRAL ACCELERATIONS (g)

Period (sec)	DE	MCE_R
	5% Damping	5% Damping
0.01	0.687	1.031
0.125	1.322	1.983
0.2	1.387	2.080
0.25	1.491	2.237
0.3	1.532	2.298
0.4	1.590	2.385
0.5	1.599	2.398
0.75	1.489	2.234
1	1.295	1.942
1.5	0.969	1.453
2	0.736	1.104
2.5	0.573	0.860
3	0.460	0.690
4	0.323	0.485
5	0.258	0.387

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 E-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 90% of the spectral acceleration at 0.5 seconds as shown in Table E-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 E-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 E-5.

TABLE E-5: SITE-SPECIFIC DESIGN ACCELERATION PARAMETERS

Parameter	Value (5% Damping)
S_{DS}	1.439 g
S_{D1}	1.472 g
S_{MS}	2.158 g
S_{M1}	2.208 g

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_G 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 0.936g and is controlled by the deterministic results. Therefore, the associated earthquake magnitude is 7.3.

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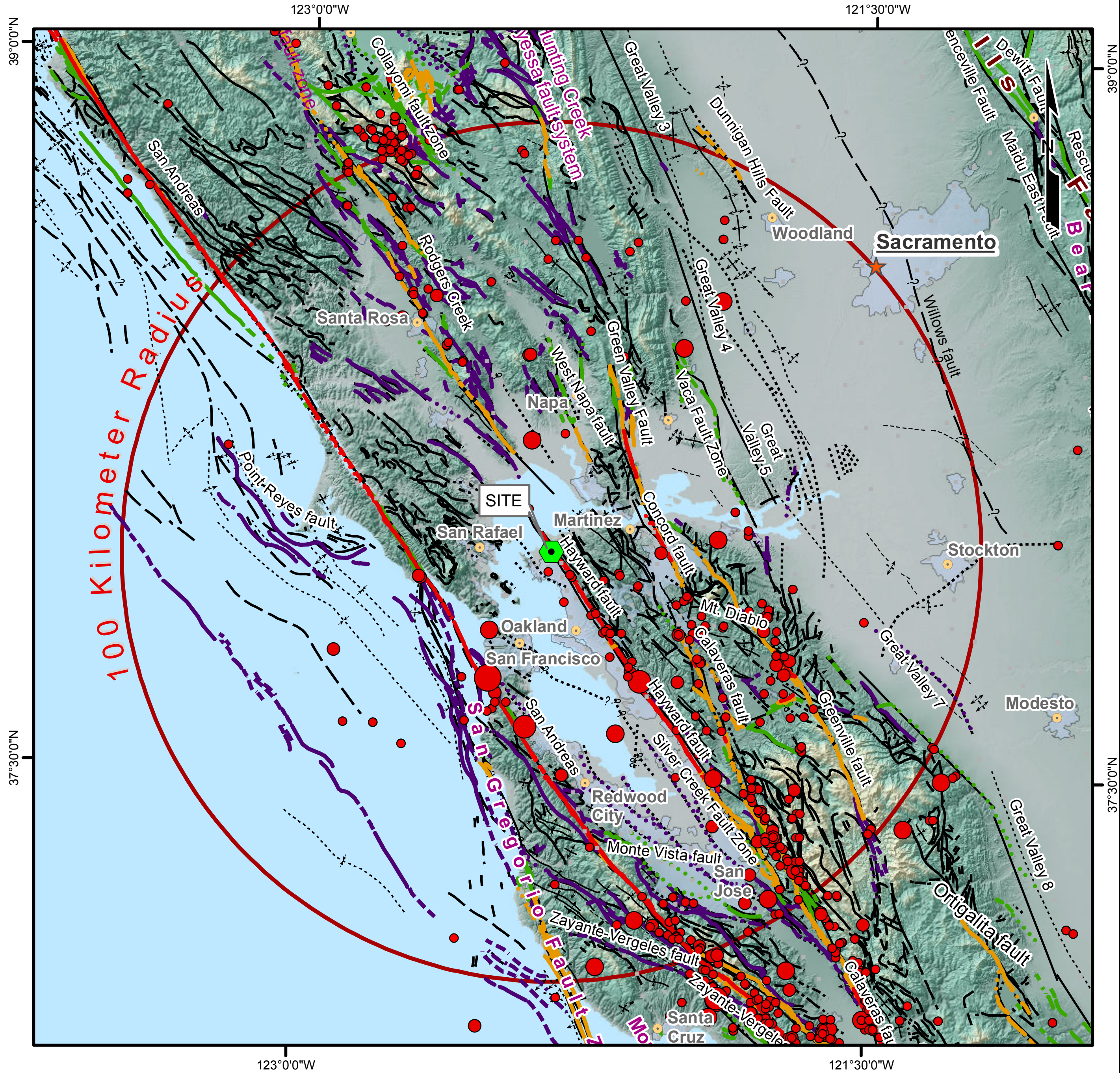

Faulting Legend

Quaternary Faults (Bryant, 2005; USGS, 2009)

Historic displacement (< 200 years)
- Mapped Fault Location
- Dashed where Approximated
- Concealed

Holocene displacement (< 11,000 years)
- Mapped Fault Location
- Dashed where Approximated
- Concealed

Late Quaternary displacement (< 750,000 years)
- Mapped Fault Location
- Dashed where Approximated
- Concealed

Quaternary displacement (< 1,600,000 years)
- Mapped Fault Location
- Dashed where Approximated
- Concealed

Pre-Quaternary Geologic Structures (CGS, 2000)
- fault, approx. located
- fault, approx. located, queried
- fault, certain
- fault, concealed
- fault, concealed, queried
- fault, inferred, queried

ANSS Earthquakes

Magnitude
- 4.0 - 4.9
- 5.0 - 5.9
- 6.0 - 6.9
- 7.0 - 7.9
- 8.0 - 8.9

35 17.5 0 35 Kilometers

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DRAWN BY:	NP
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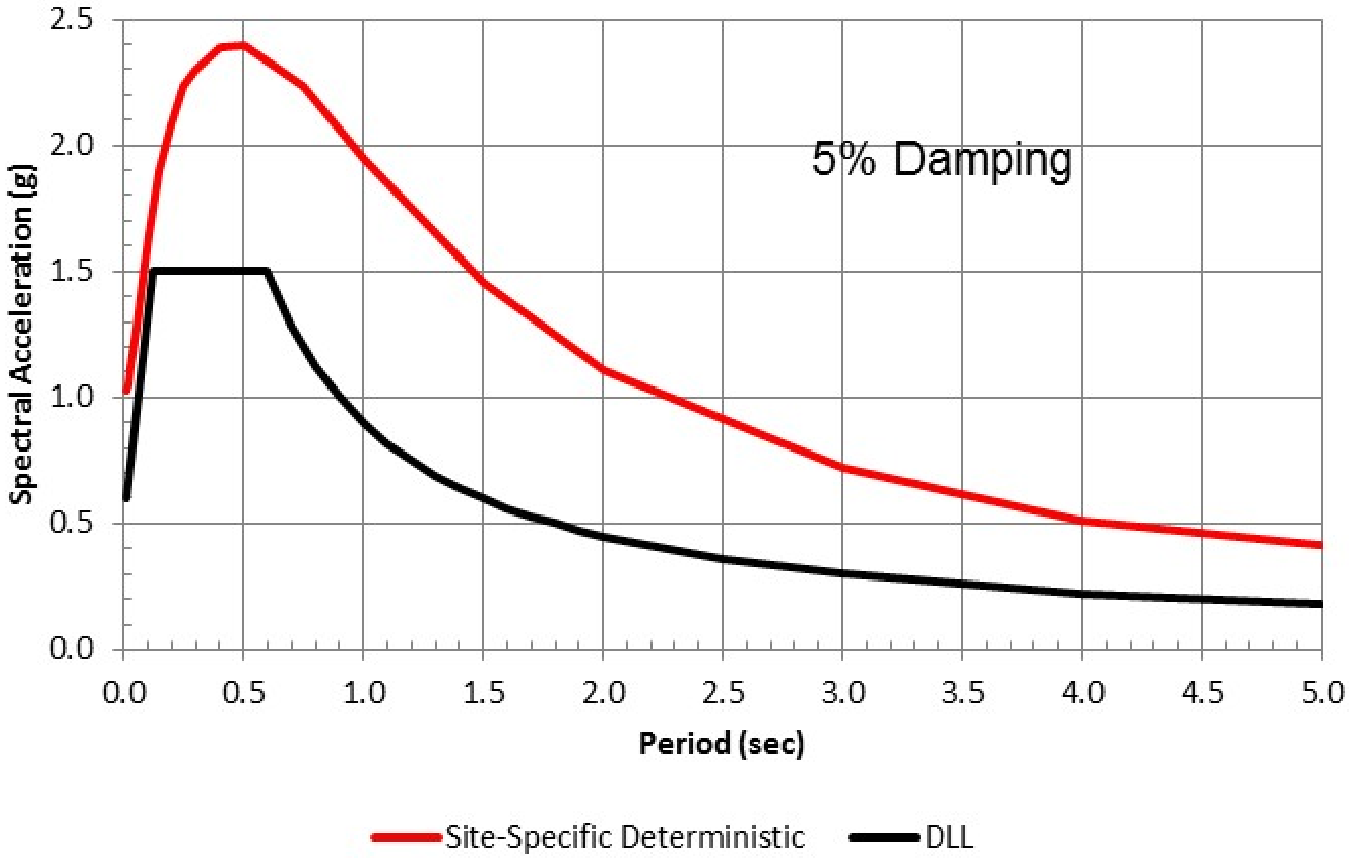
FILE NAME:	20181569_ContraCosta.mxd

REGIONAL FAULT MAP AND EARTHQUAKE EPICENTERS (1800 - OCTOBER 2017)

NEW SCIENCE BUILDING PROJECT
CONTRA COSTA COMMUNITY COLLEGE
SAN PABLO, CALIFORNIA

FIGURE

E-1



KLEINFELDER Bright People. Right Solutions. www.kleinfelder.com	PROJECT NO. 20181569	**COMPARISON OF DETERMINISTIC SPECTRA—5% DAMPING**	FIGURE
	DRAWN: 10/10/17		**E-2**
	DRAWN BY: M. Galouei		
	CHECKED BY: Z. Zafir	NEW SCIENCE BUILDING PROJECT CONTRA COSTA COMMUNITY COLLEGE SAN PABLO, CALIFORNIA	
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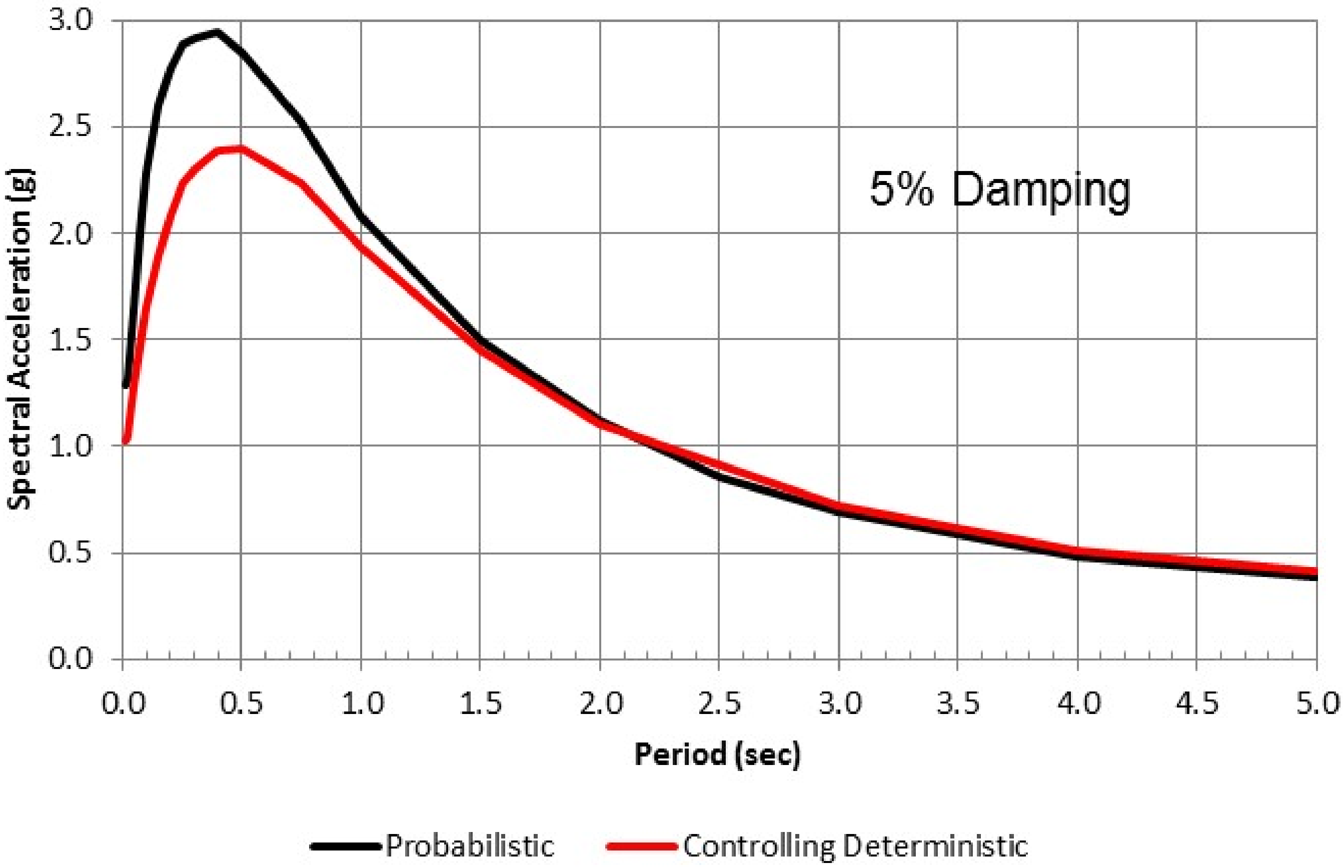

KLEINFELDER Bright People. Right Solutions. www.kleinfelder.com	PROJECT NO. 20181569	**COMPARISON OF DETERMINISTIC AND PROBABILISTIC SPECTRA**	FIGURE
	DRAWN: 10/10/17		**E-3**
	DRAWN BY: M. Galouei		
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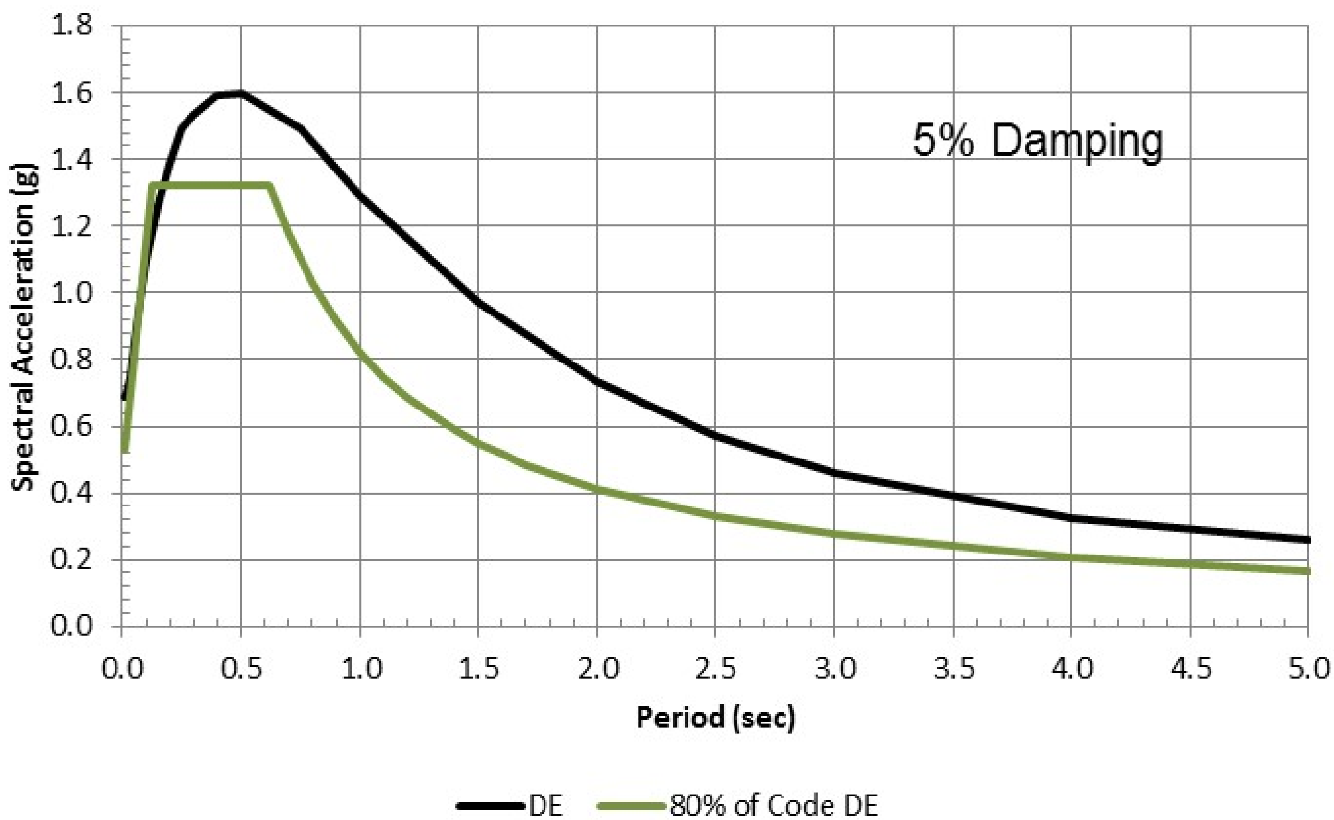

KLEINFELDER Bright People. Right Solutions. www.kleinfelder.com	PROJECT NO. 20181569	**COMPARISON OF DE SPECTRUM WITH THE CODE SPECTRUM**	FIGURE
	DRAWN: 10/10/17		**E-4**
	DRAWN BY: M. Galouei		
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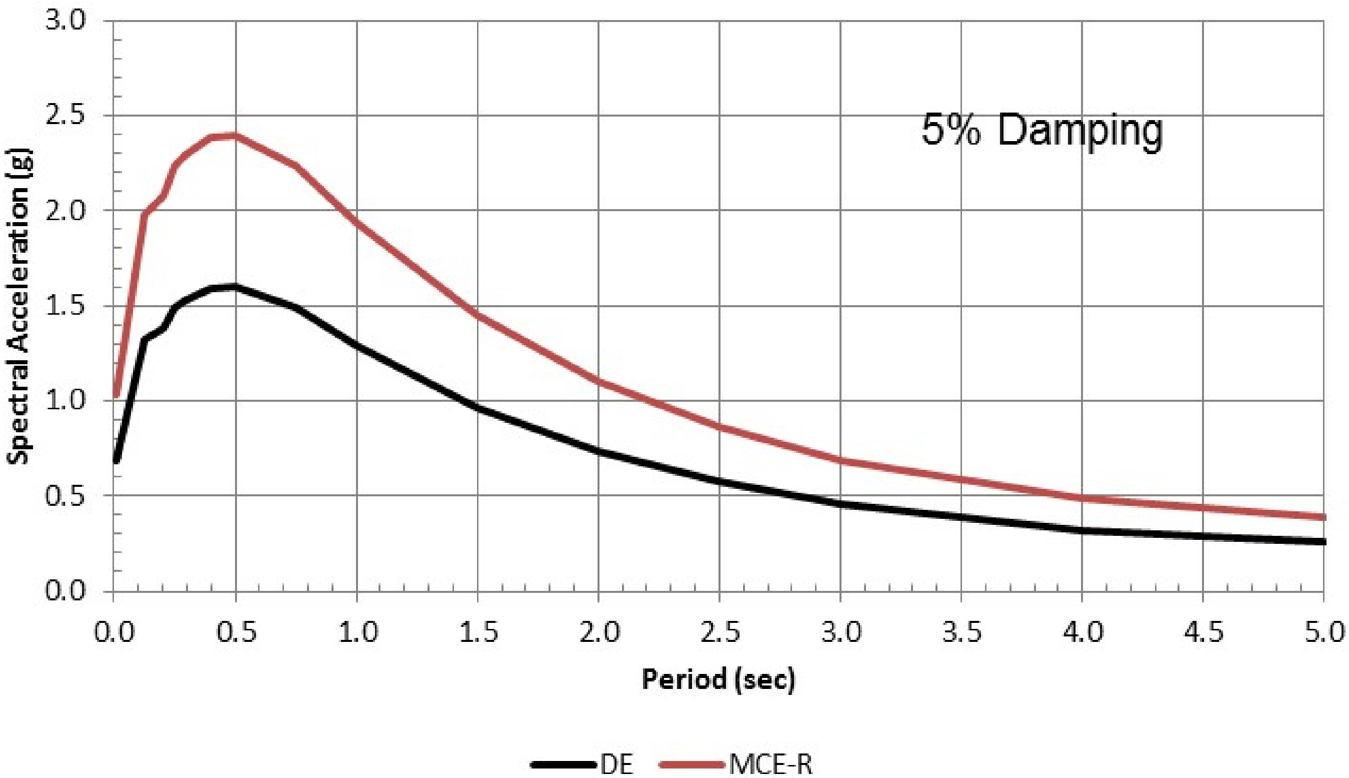

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DRAWN BY:	M. Galouei
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FILE NAME:	

SITE-SPECIFIC RESPONSE SPECTRUM—MCE$_R$ AND DE

NEW SCIENCE BUILDING PROJECT
CONTRA COSTA COMMUNITY COLLEGE
SAN PABLO, CALIFORNIA

FIGURE
E-5

March 2, 2018
File No: 20181569.002A

Mr. Ron Johnson
Contra Costa Community College District
2600 Mission Bell Drive
San Pablo, California 94806
Email: ronj@cslpm.com

SUBJECT: Response to Comments and Addendum Letter No. 1
Foundation Uplift Anchors
C-4016 New Allied Science Building
Contra Costa College
2600 Mission Bell Drive
San Pablo, California

Dear Mr. Johnson,

This addendum letter presents additional geotechnical recommendations pertaining to the C-4016 New Allied Science Building project located at Contra Costa College in San Pablo, California. This letter was prepared in response to comments received via email from Mr. Jeff Smith, structural engineer, of Rutherford + Chekene dated January 17, 2018 with regard to Kleinfelder's geotechnical report entitled "Geotechnical Engineering Investigation Report, C-4016 New Allied Science Building, Contra Costa College, 2600 Mission Bell Drive, San Pablo, California," dated October 17, 2017 (File No. 20181569.001A/PLE17R67485).

Recommendations provided herein address the construction of uplift anchors associated with the design of buckling-restrained braced frame (BRBF) seismic force-resisting system foundations. Our understanding of the proposed anchor design is based on telephone conversations with Mr. Smith and our review of project plans entitled "100 Percent Schematic Design, Phase 3 – DSA Increment 2," for Contra Costa College New Science Building, compiled by SmithGroupJJR, dated January 19, 2018. Recommendations for other elements needed for this project, including permanent soldier pile tie-back wall design, temporary shoring, and reuse of native soils for backfill are presently being considered and will be provided under separate cover. Recommendations provided in this report follow the 2016 California Building Code (CBC). The recommendations provided in the referenced geotechnical report should also be adhered to, as appropriate.

SITE SUBSURFACE CONDITIONS

The subsurface conditions at the project site are summarized in Table 1. This data is based on the soil borings included in the referenced geotechnical report (Kleinfelder, 2017).

Table 1 – Summary of Subsurface Conditions

Soil/Rock Description	Depth (ft)	Unit Weight, γ (pcf)	Assumed Friction Angle, ϕ (deg.)	Recommended Cohesion, c (psf)
Sandy CLAY	Varies 0 to 20	120	0	1,000
CLAYSTONE	20 to 40	130	0	4,000

FOUNDATION SOIL/ROCK ANCHOR DESIGN RECOMMENDATIONS

General

Based on the referenced design plans, we understand that uplift anchors are proposed to be constructed within isolated pad and strip footings throughout the building for the BRBF seismic force-resisting system.

Horizontal anchor spacing is presently proposed to be from 2 to 4 feet (center to center). The anchors are proposed to consist of an anchor bar/tendon in a grouted 6-inch minimum diameter hole with a minimum free stressing length of 10 feet and total anchor length of up to about 40 feet. Design loads on the order of 95 kips are anticipated for each individual anchor. Furthermore, we understand that these anchor elements will be used for uplift support only during potential seismic events.

Our geotechnical recommendations for the soil anchors with respect to Section 1811A of the 2016 CBC are provided below. The provided recommendations are based on guidelines presented in the Post-Tensioning Institute (PTI) "Recommendations for Prestressed Rock and Soil Anchors", Publication No. PTI DC35.1-14, dated 2014. We recommend soil anchor design and construction follow the PTI guidelines.

Minimum Diameter and Spacing

Uplift anchors should be at least 6 inches in diameter and sized to allow a minimum of 1 inch grout cover around the anchor tendon and its corrosion protection. Additionally, the hole diameter should be sized to allow for placement of a tremie grout tube alongside the tendon.

Upilft anchors should maintain a center to center spacing between bond zones of at least 5 feet. We are recommending this because deep, small diameter anchors can wander off a vertical in some cases. If that happens, the bond zones could end up being closer than anticipated. The minimum center to center spacing between installed bond zones must be greater than 3 anchor diameters. Staggering of the bond zone depths or varying the inclination of adjacent anchors should be adopted if closer spacing is necessary. Kleinfelder should evaluate that condition and its effect on anchor capacity on a case by case basis.

Grout to Ground Bond Stress

It should be noted that the exploratory borings drilled for this study extended to depths of about 40 feet and encountered claystone bedrock. At the time of the referenced geotechnical investigation, it was planned to support the building on spread footings. Therefore, deep borings and detailed characterization of the claystone bedrock were not performed.

For preliminary design of uplift anchors drilled in native claystone, the anchors should be designed for an ultimate grout to claystone bond stress of 2,000 psf, or 14 psi (taken as 50 percent of the estimated undrained shear strength for soil anchors). This is based on the recommendations contained in PTI (2014). Therefore, a maximum allowable bond stress of approximately 1,300 psf (9 psi) may be used for preliminary design. This value is derived using a factor of safety of 1.5 against pullout (for seismic events) and is based on the assumption that verification load testing will be performed on at least 3 sacrificial test anchors installed at locations selected by Kleinfelder and the project designer. Final design should be based on the results of pre-production verification testing performed by the Contractor prior to installation of the production anchors. Anchor load testing recommendations are provided in subsequent sections of this report.

Minimum Unbonded/Bonded Length of Tendon

Anchors should be designed with a minimum unbonded/free length of 10 feet for bar tendons. The bonded length should be a minimum of 15 feet in claystone. However, the minimum bonded length should be based on the required uplift capacity developed by skin friction of the grout to claystone bond. Based on a design load of 95 kips, a diameter of 6 inches, a factor of safety of 1.5 using a pressure grouted anchor, we estimate the minimum required bond length to be approximately 45.5 feet in claystone, which could extend over 25 feet below present boring depths.

Fractures and joints in the bedrock can cause excessive grout takes when using pressure-grouted anchors. The presence of fractures and joints is presently not well understood. For a gravity grouted anchor, the ultimate bond stress would be about half the value for a pressure grouted anchor, which would double the anchor bond length.

Geotechnical Considerations

Although it is likely in this geologic unit that other sedimentary rocks underlie the claystone unit encountered in the borings, their type and engineering properties have not been studied. Consideration could be given to increasing the anchor diameter to reduce the bond zone lengths in the claystone. If anchors must extend below depths of about 40 feet, we recommend performing at least 2 additional exploratory borings in the building pad area to depths of about 80 feet so that proper characterization of the bedrock unit can be performed. That will also enable us to further evaluate the appropriate anchor grouting method (i.e., gravity or pressure grouting).

Anchor Axial Tension Stiffness

Anchor axial tension stiffness should be provided by the structural engineer.

Grout Pressures

The recommended preliminary grout to ground bond stress provided above is based on pressure and/or post-grouted anchor types. Typical pressures vary from 50 to 400 psi for pressure grouting as the casing or auger is withdrawn from the hole and additional grout is pumped through the casing cap or grout swivel.

Corrosion Protection

Based on the 2016 CBC Section 1811A, Class I corrosion protection is required at a minimum for permanent anchors. Analytical lab testing performed for the referenced geotechnical report resulted in the site soils having an extreme to high corrosion potential when compared to American Water Works Association (AWWA) standards. Additionally, the environment is considered ‘aggressive’ by PTI due to a low soil electrical resistivity (less than 2,000 ohm-cm). Reference should be made to the above referenced guidelines for specific recommendations on corrosion protection. Additionally, a qualified corrosion engineer may be retained to provide corrosion protection requirements for the anchors.

Verification Load Testing

Sacrificial load tests (often termed pre-production load tests or verification tests) should be performed to verify the design and installation procedure for the anchors prior to final design and construction of production anchors. These load tests are also needed to evaluate the anchor grout to ground bond stress for final design. The tests should be performed at three (3) locations to be determined by the Structural and Geotechnical engineers.

Each anchor should be load tested in tension to at least 150 percent of the design load, per ASTM D 3689. The central reinforcing bar should be designed such that the maximum tensile stress does not exceed 80 percent of the yield strength of the steel. The jack should be positioned at the beginning of the test such that unloading and repositioning of the jack during the test will not be required. Upon completion of the load testing, the geotechnical engineer should evaluate the data obtained and provide final recommendations for the production anchors.

During production anchor construction, proof-load testing should be performed on all production anchors up to 133 percent of the design load, per PTI guidelines.

Lock-off Loading

The magnitude of the lock-off load shall be specified by the structural engineer and shall not exceed 70 percent of the steel yield strength.

Drilling Methods

The anchor drilling method should be selected by the Contractor and should be appropriate for the encountered soil and rock conditions and proposed grouting method. Caving conditions are not anticipated within the clayey on-site soils and claystone. Additionally, groundwater was not encountered within the explored borings to depths of approximately 40 feet. However, groundwater has been encountered throughout borings and trenches performed throughout the college campus at depths ranging between 9 to 23 feet below the ground surface.

Construction Observation and Monitoring

We recommend that all anchor construction and testing be monitored by a representative of Kleinfelder, including drilling, grout placement, and all verification and proof-load testing in accordance with Chapter 17 of the CBC and PTI (2014). The purpose of these services would be to provide Kleinfelder the opportunity to observe the subsurface conditions encountered during construction, evaluate the applicability of the recommendations presented in this addendum letter to the subsurface conditions encountered, and prepare recommendations for final anchor design and construction.

CLOSURE

Unless specifically superseded in this addendum, the recommendations presented in the above-referenced geotechnical report remain applicable. This document is intended to provide specific recommendations for preliminary design of uplift anchors for the subject project. Accordingly, it cannot be considered an independent document, as it does not contain adequate background information. This document is directed only to the personnel with detailed knowledge of the subject project. The conclusions and recommendations presented in this addendum were prepared under the conditions and limitations presented in our above-referenced October 2017 geotechnical investigation report.

We trust this information meets your current needs. We appreciate the opportunity to be of professional service to you on this project. If you have any questions, please do not hesitate to contact us at (916) 366-1701.

Respectfully submitted,

KLEINFELDER, INC.

REGISTERED PROFESSIONAL ENGINEER
REBECCA L. MONEY
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Revised Date: June 14, 2018
March 14, 2018
File No: 20181569.002A

Mr. Ron Johnson
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Email: ronj@cslpm.com

**SUBJECT: Response to Comments and Addendum Letter No. 2
Temporary Shoring and use of
Native Soil as Backfill
C-4016 New Allied Science Building
Contra Costa College
2600 Mission Bell Drive
San Pablo, California**

Dear Mr. Johnson,

This addendum letter presents additional geotechnical recommendations pertaining to the C-4016 New Allied Science Building project located at Contra Costa College in San Pablo, California. This letter was prepared in response to comments received via email from Mr. Jeff Smith, structural engineer, of Rutherford + Chekene dated January 17, 2018 with regard to Kleinfelder's geotechnical report entitled "Geotechnical Engineering Investigation Report, C-4016 New Allied Science Building, Contra Costa College, 2600 Mission Bell Drive, San Pablo, California," dated October 17, 2017 (File No. 20181569.001A/PLE17R67485). These previously prepared plans have been revised to eliminate the soldier pile wall. We previously prepared a letter report with recommendations for the soldier pile wall in our letter dated March 14, 2018. We have modified this report accordingly.

Recommendations provided herein address the temporary shoring and reuse of native soils for backfill. Our understanding of the proposed project is based on telephone conversations with Mr. Smith and our review of project plans entitled "100 Percent Schematic Design, Phase 3 – DSA Increment 2," for Contra Costa College New Science Building, compiled by SmithGroupJJR, dated January 19, 2018. We previously provided an addendum letter entitled, "Response to Comments and Addendum Letter No. 1, Foundation Uplift Anchors, C-4016 New Allied Science Building, Contra Costa College," dated March 2, 2018 (File No. 20181569.002A/SAC18L74494) which provided recommendations for uplift anchors associated with the design of buckling-restrained braced frame (BRBF) seismic force-resisting system foundations. Recommendations provided in this report are consistent with the requirements of the 2016 California Building Code (CBC). The recommendations provided in the referenced geotechnical report should also be adhered to, as appropriate.

SITE SUBSURFACE CONDITIONS

The subsurface data is based on the soil borings included in the referenced geotechnical report (Kleinfelder, 2017) which are summarized in Table 1.

Table 1 – Summary of Subsurface Conditions

Soil/Rock Description	Depth (ft)	Unit Weight, γ (pcf)	Assumed Friction Angle, ϕ (deg.)	Recommended Cohesion, c (psf)
Sandy CLAY	Varies 0 to 20	120	0	1,000
CLAYSTONE	20 to 40	130	0	4,000

SUPPLEMENTAL LABORATORY TESTING

In order to further characterize the soils within the area of the proposed permanent soldier pile wall, a representative of Kleinfelder performed a site reconnaissance to collect a bulk sample of the near-surface site soils near the wall location for additional geotechnical laboratory testing. The laboratory testing program included maximum dry density, Atterberg limits, sieve analysis, and one-dimensional swell tests in accordance with ASTM standards to evaluate the physical characteristics and engineering properties of the clay soils proposed to be retained by the soldier pile tie-back wall. A summary of the laboratory tests and results is presented in Table 2, below.

Table 2 – Summary of Laboratory Testing

Test	ASTM Standard	Result
Modified Proctor	D1557A	DD = 116.6 pcf, MC = 13.5 %
Atterberg Limits	D4318	LL = 45, PI = 23 (CL)
Sieve Analysis	C136 / C117	99 % passing No. 4 90.9 % passing No. 200
One-Dimensional Swell/Collapse	D4546	800 psf for 0 % Swell

Notes: DD=Dry Density, MC=Moisture content, LL=Liquid Limit, PI=Plasticity Index

EARTHWORK RECOMMENDATIONS

Expansive Soils

As discussed in the referenced geotechnical report, near-surface, clayey soils were encountered within the building footprint and are considered moderately to highly expansive. Expansive soils are characterized by their ability to undergo significant volume change (shrink or swell) due to variations in moisture content. Changes in soil moisture content due to rainfall, landscape irrigation, utility leakage, roof drainage, perched groundwater, drought, or other factors may cause unacceptable settlement or heave of structures, retaining walls, or concrete slabs supported-on-grade. Depending on the extent and location of expansive soils below finished subgrade, these soils could have a detrimental effect on the proposed construction.

Due to the expansive soil properties discussed above and the inability to adequately characterize the potential lateral pressures from expansion on the retaining walls, clayey soils are typically not recommended to be used as backfill adjacent to retaining structures. As such, the referenced geotechnical report recommended that all retaining wall backfill consist of imported, non-expansive fill. However, we understand that the proposed permanent soldier pile wall at the northwest corner of the building will retain a cut consisting of native clayey soil. As a result, we have provided recommendations in this letter for design of the proposed soldier pile tie-back wall supporting native clayey soils.

Reuse of Onsite Materials

On-site clay soils encountered across the site are considered suitable for reuse as general engineered fill provided that they are not placed within the upper 2 feet of areas supporting improvements (e.g. building pad subgrades, exterior flatwork, etc.) unless chemically treated with sufficient high-calcium quicklime to reduce the expansion potential to meet non-expansive fill requirements. Engineered fill consisting of native clayey soils placed deeper than 2 feet from finished soil grades should be compacted to between 88 and 92 percent relative compaction at a moisture content at least 3 percent above optimum. As stated above, the upper 2 feet of finished soil grades supporting improvements should consist of lime treated soil or imported non-expansive fill.

Chemical stabilization of the clay soils can be accomplished by mixing with high-calcium quicklime. For estimating purposes, quicklime application rates of 4 to 5 pounds per cubic foot of soil treated may be assumed. The actual quicklime application rate should be evaluated by laboratory testing of soil samples obtained from on-site borrow areas prior to construction. Lime treatment should be performed by a specialty contractor experienced in this work and should be performed in accordance with Caltrans Standard Specifications. Lime-treated areas can have significantly elevated pH levels (pH over 10) and may not be appropriate for use in landscaped areas. If used beneath improvements, appropriate corrosion protection should be provided. Final lime application rates should be determined such that a stabilized fill material with an expansion index of less than 20 (based on ASTM test method D4829) is achieved. The lime-stabilized soil should be compacted to at least 90 percent relative compaction at a moisture content of at least 2 percent above optimum, based on ASTM D1557. The upper 6 inches of subgrades supporting exterior slabs or pavements should be compacted to at least 93 percent of the maximum dry density.

TEMPORARY SHORING

General

As requested by the structural engineer, we are providing the following comments and recommendations regarding temporary shoring for the project. Shoring may be required where space or other restrictions do not allow a sloped excavation. This may include excavations within/near roadways and near or around existing utilities and structures. Since selection of appropriate shoring systems will be dependent on construction methods and scheduling, we recommend the Contractor be solely responsible for the design, installation, maintenance, and performance of temporary shoring systems. Shoring, bracing, or underpinning required for the project (if any) should be designed by a professional engineer registered in the State of California.

Discontinuous shoring systems are not recommended for excavations deeper than about 8 feet at the site based on the soils and groundwater conditions encountered. Continuous shoring systems such as Slide Rail, internally braced systems, trench boxes, or other applicable shoring systems may be suitable provided Cal OSHA regulations are met and damage to existing adjacent improvements does not result from their use.

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Where trenches are excavated in and/or near existing roadways, structures or underground utilities, we recommend shoring systems be designed to provide positive restraint of trench walls. Where positive restraint of trench walls is not provided, lateral deformation of the trench walls may result in ground cracks, settlement and/or other ground movements that may affect adjacent underground utilities as well as surface improvements. If trench walls deflect laterally in pavement areas, parallel cracks may develop in the pavement and underlying soils that may require repair. The Contractor should be made aware of this potential condition in order that preventative measures can be implemented or repair measures provided for.

Lateral Earth Pressures

Where there is insufficient space to lay back the slopes for the planned excavations, shoring will be required. For design of cantilevered shoring (i.e., soldier piles, sheet piles, or similar shoring systems), a triangular distribution of lateral earth pressure may be used. For design of braced shoring, a uniform distribution of earth pressure is recommended. Sheet pile, soldier pile, or similar shoring systems either incapable of deflection or which are fully constrained against deflection may be designed for an equivalent fluid at-rest pressure. Table 3, below, provides approximate lateral earth pressures for use in preliminary shoring design based on an angle of internal friction of 0 degrees, an apparent cohesion of 1,000 psf, and a moist soil unit weight of 120 pcf for native clay soils and a level ground surface (without surcharge loading) adjacent to the top of the shoring. The earth pressure values provided below are ultimate values. Therefore, a factor of safety of at least 1.5 should be used for design of the lateral force resisting system. Since this is a temporary system, seismic pressures are not provided. Final design of shoring systems should be performed by the contractor based on their review of the trench wall soil conditions.

Table 3 - Lateral Earth Pressures for Braced Shoring in Clay Soils

Condition	Level Backfill
Active Pressure (psf)	41H
At-rest/Restrained Pressure (psf)	85H
Passive Pressure (psf/ft) for Native Stiff Clay	275

Notes: 1. H is shored height in feet.

Lateral Deflection of Shoring Systems

Lateral deflection of a shored excavation is heavily dependent on the relative stiffness of the shoring system, the amount of bracing and/or tie-backs, and the quality of workmanship during installation. The limiting condition of maximum active earth pressure for soft to firm silts is generally reached when the shoring tilts or deflects laterally about 1 percent of the shoring wall height in stiff cohesive materials. If the shoring tilts or deflects less than the limiting condition, the lateral earth pressure will lie between the active and at-rest earth pressures. This soil movement can extend horizontally as far back as 2H back from the top of cantilever retaining structures, with vertical movements approximately equal to the horizontal. The movement tends to be greatest close to the excavation and becomes less with increasing distance away. Backfilling void spaces behind shoring with sand or pea gravel may reduce the potential for vertical and lateral movements around the excavation.

The shoring designer should perform a deflection analysis of the shoring system. If movements are greater than the tolerance of existing project features (utilities, pavements, structures, etc.) tie-backs, dead-man anchors, or cross bracing may be needed to reduce deflections. Design using the at-rest pressure and/or more stringent tie-back or bracing systems may be required in the vicinity of improvements that cannot withstand lateral movements.

Lateral Resistance

All soldier or sheet piles should extend to a sufficient depth below the excavation bottom to provide the required lateral resistance. Embedment depths should be determined using methods based on the principles of force and moment equilibrium. To account for three-dimensional effects on a soldier pile, the passive pressure may be assumed to act on an area 2 times the width of the embedded portion of the pile, provided adjacent piles are spaced at least 3 diameters, center-to-center. A minimum factor of safety of 1.2 should be applied to the calculated embedment depth and to determine the allowable passive pressure. The shoring professional should evaluate the final design conditions and shoring type to select the appropriate factor of safety for design.

The passive earth pressure, similar to active earth pressures, is mobilized when the shoring below the excavation bottom tilts or deflects laterally. The limiting condition of maximum passive earth pressure is generally reached when the shoring deflects laterally below the base of the excavation about 0.2 percent of the embedment depth below the bottom of the excavation in dense sands and about 2 percent of the embedment depth below the bottom of the excavation in stiff cohesive material. If the shoring system is restrained against movement, the lateral resistance below the base of the excavation will lie somewhere between the passive and at-rest earth pressure conditions. Accordingly, if lateral deflection at the base of the excavation is objectionable, the at-rest earth pressure should be used in design for lateral resistance.

Surcharge Pressures

Shoring systems should be designed to resist lateral pressures due to hydrostatic forces, if present, and surface loads adjacent to excavations. We anticipate surface loads will be imposed by construction equipment, foundations, exterior flatwork, etc. Actual surcharge pressures will depend upon the geometry (i.e., point-, strip- or rectangular-shaped loaded area), the size of the loaded area, the position of the loaded area relative to the shoring, and the magnitude of the load. Thirty-five and fifty-five percent of any areal surcharge placed adjacent to the shoring may be assumed to act as a uniform horizontal pressure against the shoring for active and at-rest earth pressure conditions, respectively. It is common in shoring design to use an appropriate Boussinesq theory solution to evaluate surcharge load pressures. Special cases, such as combinations of sloping and shoring or other surcharge loads (not specified above) may require an increase in the design values recommended above. These conditions should be evaluated by the shoring designer.

Protection of Existing Utilities, Structures, and Pavements

The shoring designer should complete a survey of existing utilities, pavements, and structures adjacent to those portions of the proposed excavation that will be shored. The purpose of this review would be to evaluate the ability of existing pipelines or conduits to withstand horizontal movements associated with a shored excavation. If existing utilities, pavements, and structures are not capable of withstanding anticipated lateral movements, alternative, more robust shoring systems may be required. It may be necessary to repair cracks in pavements adjacent to shored portions of excavations due to lateral displacements of the shoring systems and the ground that it retains.

Existing Trench Backfill Conditions

In areas where existing trench backfills are exposed in or located adjacent to excavations for the proposed improvements, the shoring design criteria presented above may not be valid. The shoring designer should consider the presence of existing utility trenches in and near the proposed excavation areas as well as methods to protect the utilities. If existing trench backfill materials are encountered in excavations on the site, the shoring designer should be notified immediately to observe and address the encountered conditions. It should be noted that trench wall collapses have occurred where these conditions were not recognized and addressed during construction.

Monitoring

Where existing facilities adjacent to an excavation must be protected, horizontal and vertical movements of the shoring system should be monitored by establishing survey points, installation of inclinometers, or a combination of both prior to excavation such that the vertical and horizontal positions of the monitoring points can be recorded to the nearest 0.01 feet. The results should be reviewed by a qualified Geotechnical Engineer on a daily basis for a period of at least one week during excavation and following construction of the shoring system. Measurements should be obtained on a weekly basis thereafter. Detailed recommendations for monitoring should be provided by a qualified Geotechnical Engineer after a review of the planned shoring system.

Construction Vibrations

The Contractor should use means and methods that will limit vibrations at the locations adjacent structures/facilities. Where construction operations such as sheet pile driving, demolition, or similar activities induce significant ground vibrations near critical facilities we recommend vibration monitoring be performed. As a guide, peak particle velocities from construction vibrations within adjacent structures/facilities should be limited to less than 1 inch/second when measured using an accelerometer. More stringent requirements may be needed adjacent to historic structures, buildings in poor conditions, or buildings where vibration sensitive equipment is being operated. We suggest the need for vibration monitoring be evaluated on a case-by-case basis.

Shoring Removal

Shoring systems typically are removed as part of the trench backfill process. Depending on the shoring system used, the removal process may create voids along the sides of the trench excavation. If these voids are left in place and are significantly large, backfill may shift laterally into the voids resulting in settlement of the backfill and overlying improvements. Therefore, care should be taken to remove the shoring system and backfill the trench in such a way as to not create these voids. If the shoring system requires removal after backfill is in place, resulting voids should be filled with cement slurry or grout.

Design Groundwater Conditions

Due to the hilly terrain and shallow claystone present within the northeast region of the project area, groundwater may be present perched above the interfaces of soil/bedrock, different weathering zones, or different fracture density zones in the rock mass. Perched groundwater is expected to be most prevalent during the winter and spring months, and declining throughout the summer and early fall.

Actual groundwater levels at any given location will vary with seasonal variations in rainfall and runoff, stream levels, irrigation practices, and other factors. A site-specific hydrogeologic evaluation of seasonal fluctuations is beyond the scope of this study.

SITE DRAINAGE

As discussed in this letter and the referenced geotechnical report, the shrink-swell characteristics resulting from wetting and drying of the onsite site soils can have detrimental effects on the proposed construction. Therefore, proper site drainage is highly emphasized around the proposed structural improvements for long-term performance of the planned building, retaining walls, and exterior concrete flatwork. Landscaping planters are considered a primary source of seepage and moisture intrusion into subsurface soils.

Further discussion on site drainage for the project is discussed in Section 6.8, SITE DRAINAGE, of the referenced geotechnical report.

CLOSURE

Unless specifically superseded in this addendum, the recommendations in the above-referenced geotechnical report remain applicable. This document is intended to provide specific recommendations for the subject project. Accordingly, it cannot be considered an independent document, as it does not contain adequate background information. This document is directed only to the personnel with detailed knowledge of the subject project. Please attach this addendum to the above-referenced geotechnical report. The conclusions and recommendations presented in this addendum were prepared under the conditions and limitations presented in our above-referenced October 2017 geotechnical investigation report.

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We trust this information meets your current needs. We appreciate the opportunity to be of professional service to you on this project. If you have any questions, please do not hesitate to contact us at (916) 366-1701.

Respectfully submitted,

KLEINFELDER, INC.

Edgar A. Santos, EIT
Staff Engineer

Rebecca L. Money, PE, GE
Senior Geotechnical Engineer

Reviewed by:

Kenneth G. Sorensen, PE, GE
Principal Geotechnical Engineer

KLEINFELDER
Bright People. Right Solutions.

August 16, 2018
File: 20181569.002A

Mr. Ron Johnson
Contra Costa Community College District
2600 Mission Bell Drive
San Pablo, CA 94806
Email: ronj@cslpm.com

Subject: **Addendum #3 - Bearing Capacity Factor of Safety Clarification**
C-4016 New Allied Science Building
Contra Costa College
2600 Mission Bell Drive
San Pablo, California

Dear Mr. Johnson:

This letter provides clarification on the factor of safety used in our calculations to obtain the allowable bearing capacity for the subject project. This letter was prepared in response to our e-mail and phone communications with the Structural Engineer, Mr. Jeff Smith, of Rutherford + Chekene. Factors of safety for the allowable bearing capacity with respect to allowable stress design and overstrength factors outlined in the 2016 CBC Section 1605A.I.1 are provided below.

Bearing Capacity Recommendations (Spread Footings)

The geotechnical report, Geotechnical Engineering Investigation Report, C-4016 New Allied Science Building, Contra Costa College, 2600 Mission Bell Drive, San Pablo, California," dated October 17, 2017 (File No. 20181569.001A/PLE17R67485) recommended a net allowable bearing capacity of 3,000 psf for spread footings (dead + live loads); the allowable bearing capacity included a factor of safety of at least 3. This recommendation applied to footings with a minimum width of 18 inches, founded at least 30 inches below adjacent finished grade, and with the earthwork recommendations provided in the report.

A one-third increase in the net allowable bearing capacity was recommended to consider short-term loading due to wind or seismic forces. Per the Structural Engineer, a bearing capacity of 4,000 psf has been used for design of the building's spread footings for the seismic condition (dead + live + seismic loads) and minimum footing widths for the project are 3 feet and founded 30 inches below adjacent finished grade. Based upon the proposed footing sizes and depth for this project, an ultimate bearing capacity of 10,000 psf can be utilized for seismic design of footings. A factor of safety of 2.5 is recommended for the seismic condition; therefore the net allowable bearing capacity for the seismic condition is 4,000 psf.

We understand a factor of safety of 2.5 for the seismic allowable bearing capacity is equal to the factor of safety used for the overstrength factor in design of the seismic force-resisting system.

LIMITATIONS

This letter is subject to the recommendations and provisions and requirements outlined in the limitations section of the 2017 geotechnical investigation report. No warranty, express or implied, is made.

CLOSURE

Thank you for this opportunity to be of service. If you have any questions or if we can be of further assistance, please contact the undersigned at (916) 366-1701.

Sincerely,

KLEINFELDER, INC.

Rebecca L. Money, PE, GE
Principal Geotechnical Engineer

Kenneth G. Sorensen, PE, GE
Senior Principal Engineer

December 12, 2018
File No: 20181569.002A

Mr. Ron Johnson
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2600 Mission Bell Drive
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Email: ronj@cslpm.com

**SUBJECT: Geotechnical Engineering Addendum Letter No. 4
Additional Geotechnical Investigation and Recommendations for Foundation Uplift Anchors
C-4016 New Allied Science Building
Contra Costa College
2600 Mission Bell Drive, San Pablo, California**

Dear Mr. Johnson:

This addendum letter provides supplemental recommendations for foundation uplift anchor design for the subject project and is based on the additional field explorations, laboratory testing, and engineering analysis performed; this addendum letter supersedes Addendum No. 1, dated March 2, 2018.

BACKGROUND

Kleinfelder prepared a geotechnical report for the project in October 2017. As the design of the project progressed, foundation uplift anchors were determined to be necessary to resist uplift forces caused by a seismic event. Kleinfelder prepared this letter based on the results of this investigation and our previous reports and addenda, which include the following:

- "Geotechnical Engineering Investigation Report," dated October 17, 2017
- "Geologic and Seismic Hazards Assessment Report," dated October 20, 2017
- "Response to Comments and Addendum Letter No. 1, Foundation Uplift Anchors," dated March 2, 2018.
- "Response to Comments and Addendum Letter No. 2, Temporary Shoring and use of Native Soil as Backfill," dated June 14, 2018
- "Addendum #3 – Geotechnical Recommendations, Bearing Capacity Factor of Safety," dated August 16, 2018
- "Response to DSA Geotechnical Comments on Increment 1 Submittal," dated October 12, 2018
- "On-Site Soil Analytical Testing," dated December 4, 2018

Seismic uplift anchors are currently planned to extend to depths up to about 50 feet below the new structure foundations. As a result, Kleinfelder performed supplemental explorations utilizing different drilling methods and greater depths to evaluate the subsurface conditions and provide updated uplift anchor design recommendations.

FIELD EXPLORATION AND LABORATORY TESTING

Supplemental Core Borings

Core Borings B-5 and B-6 were drilled by Pitcher Drilling Company of East Palo Alto, California on November 7 through 9, 2018. The borings were drilled using a Fraste Multidrill XL, track-mounted drill rig equipped with 5-inch-diameter, solid stem augers and an HQ-3, triple tube, wireline coring system. The site location is shown on Figure 1, Site Vicinity Map. The approximate locations of the borings drilled for this study and our previous investigation are shown on Figure 2, Site Plan. Borings were located in the field by measuring from existing landmarks and were not surveyed. Therefore, the locations of the borings shown on Figure 2 should be considered approximate.

A Kleinfelder representative observed and sampled the materials encountered in the borings. The engineer maintained a log of each boring, visually classifying the soil from samples and auger cuttings in general accordance with ASTM Method D2488. Sample classifications, hammer blow counts during sampling, and other related information were recorded on the boring logs. Rock was logged in accordance with Kleinfelder's standard rock classification system that is based on a combination of U.S. Army Corps of Engineers, U.S Bureau of Reclamation, and International Society of Rock Mechanics rock property criteria. The core samples were reviewed by our Certified Engineering Geologist prior to completing the boring logs. Keys to the soil and rock descriptions and symbols used on the boring logs are presented on Figures A-1 through A-3 in Appendix A. Boring logs from our previous explorations in 2017 (B-1 through B-4) are included on Figures A-4 through A-7 in Appendix A. Logs of borings from this supplemental investigation are presented on Figures A-8 and A-9. Photographs of rock core samples recovered from the borings are presented on Figures B-1 through B-16 in Appendix B.

Disturbed and relatively undisturbed samples were taken from the borings at selected intervals during drilling. Soils were sampled by driving either a 2.5-inch inner diameter (ID) split-barrel (California) sampler or a 1.4-inch ID Standard Penetration Test (SPT) sampler into the soil with a 140-pound automatic-trip hammer free-falling a distance of 30 inches. The California sampler was used with stainless steel liners and is in general conformance with ASTM D3550. The SPT sampler was used without liners and is in general conformance with ASTM D1586. Blow counts were recorded at 6-inch intervals for each sample attempt and are reported on the boring logs. Blow counts shown on the logs have not been corrected for the effects of overburden pressure, rod length, sampler size, or hammer efficiency. However, sampler size correction factors were applied to estimate the sample apparent density noted on the boring logs.

Soil samples obtained from the borings were packaged and sealed in the field to reduce moisture loss and disturbance. Rock core samples were recovered in about 2- to 5-foot-long core runs, measured, logged, and photographed before placing in plastic core boxes. The degree of weathering and fracturing, sample recovery and rock quality designation (RQD) for the rock was recorded per run length and is documented on the boring logs.

Following drilling, the soil and rock samples were returned to our Sacramento and Hayward laboratories for further examination and testing.

Borings B-5 and B-6 were backfilled with neat cement grout from the bottom upward through a tremie pipe under the supervision of a Contra Costa County Environmental Health inspector. The drill cuttings and fluids were placed into 55-gallon steel drums. Pitcher Drilling subsequently coordinated the testing, pick-up and off-site disposal of the drums of cuttings.

Subsurface Conditions

It should be noted that the exploratory borings drilled for this addendum extended to depths ranging from about 47 to 91 feet below the ground surface and encountered mainly sandy lean clay and clayey sand fill and native soils underlain by claystone bedrock with interbedded siltstone and sandstone (graywacke). The bedrock units encountered appeared to be decomposed to highly weathered and intensely fractured with RQD of zero. Photographs of the core samples recovered from Borings B-5 and B-6 are included on Figures B-1 through B-16 in Appendix B.

The bedrock depths and corresponding elevations, as encountered in the borings drilled for this addendum and our previous geotechnical investigation report, are presented in Table 1 below.

Table 1
Summary of Bedrock Depths and Elevations

Boring No.	Depth to Bedrock (ft)	Bedrock Elevation (ft, msl)
B-1	20	72
B-2	5	88
B-3	20	60
B-4	20	60
B-5	27	64
B-6	26	54

Soil and Rock Geotechnical Laboratory Testing

Geotechnical laboratory testing was directed toward assessing the strength characteristics of the subsurface bedrock units. Our laboratory testing included three unconsolidated undrained triaxial (TXUU) tests conducted in general accordance with ASTM test procedure D2850 by Kleinfelder's laboratory in Sacramento, California. Due to the high degree of fracturing in the sandstone (graywacke) rock unit, performance of rock unconfined compressive strength or point load index testing could not be performed. The results of laboratory tests performed for this addendum are presented in Appendix C.

FOUNDATION UPLIFT ANCHOR DESIGN RECOMMENDATIONS

General

We understand ground anchors will be used to resist seismic uplift forces on the braced frame foundations beneath the building.

Horizontal anchor spacing is presently proposed to be about 5 feet center to center. The anchors are proposed to consist of a steel anchor bar in a grouted hole with a minimum unbonded (free stressing) length of 10 feet. The ultimate anchor load for an individual anchor is anticipated to be about 235 kips.

Uplift Anchor Design and Construction

For preliminary anchor design, verification load testing, final design and construction, we recommend using the procedures outlined in the FHWA Geotechnical Engineering Circular No.

4, “Ground Anchors and Anchored Systems,” dated June 1999 (Publication No. FHWA-IF-99-015).

Minimum Diameter and Spacing

We recommend anchor bond zones be 8 to 10 inches in drilled diameter and sized to allow a minimum of 1 inch of grout cover around the anchor bar and its corrosion protection. Additionally, the hole diameter should be sized to allow for placement of tremie grout and post-grout tubes alongside the bar.

Uplift anchors should maintain a center to center spacing between bond zones of at least 5 feet. We are recommending this because deep, small diameter anchors can wander off a vertical in some cases. If that happens, the bond zones could end up being closer than anticipated. The minimum center to center spacing between installed bond zones must be greater than 3 anchor diameters. Staggering of the bond zone depths or varying the inclination of adjacent anchors should be adopted if closer spacing is necessary. Kleinfelder should evaluate that condition and its effect on anchor capacity on a case by case basis, if needed.

Anchor Grouting and Grout to Ground Bond Stress

The bond zones of the proposed uplift anchors should be situated within the bedrock units at depths greater than shown in Table 1 above.

Based on our explorations and laboratory testing, as well as the presumptive bond stress values provided in Table C6.2 in the Post-Tensioning Institute (PTI) “Recommendations for Prestressed Rock and Soil Anchors,” (Publication No. PTI DC35.1-14, dated 2014), a preliminary, ultimate bond stress value of 20 to 25 psi is recommended for the claystone and other bedrock units beneath the site provided the anchors are post-grouted following initial installation and grouting. To achieve these preliminary design bond stress values, multiple stages of post-grouting may be required. We recommend no more than 3 stages of post grouting be performed. Post-grout injection ports should be spaced no further than 5 feet apart along the grout tubes.

The provided bond stress value is based on the assumption that verification load testing will be performed on at least 3 sacrificial test anchors installed at locations selected by Kleinfelder and the project designer. Final design should be based on the results of pre-production verification testing performed by the Contractor prior to installation of the production anchors. Anchor load testing recommendations are provided in subsequent sections of this letter.

Minimum Unbonded/Bonded Length of Tendon

Anchors should be designed with a minimum unbonded/free length of 10 feet for bar tendons. The bonded length should be a minimum of 15 feet in bedrock below the elevations shown on Table 1. However, the minimum bonded length should be based on the required uplift capacity developed by skin friction of the grout to rock bond.

Geotechnical Considerations

Fractures and joints in the bedrock can cause excessive grout takes when using pressure-grouted anchors. The presence of fractures and joints is pervasive in the bedrock unit, as can be seen in the attached core photographs. Therefore, the contractor should consider measures to manage excessive grout losses into discontinuities in the anchor bond zones.

Anchor Axial Tension Stiffness

Anchor axial tension stiffness should be provided by the structural engineer.

Corrosion Protection

Based on the 2016 CBC Section 1811A, Class I corrosion protection is required at a minimum for permanent anchors. Analytical lab testing performed for the referenced geotechnical report resulted in the site soils being characterized as having an extreme to high corrosion potential when compared to American Water Works Association (AWWA) standards. Additionally, the environment is considered ‘aggressive’ by PTI due to a low soil electrical resistivity (less than 2,000 ohm-cm). Reference should be made to the above referenced guidelines for specific recommendations on corrosion protection. Additionally, a qualified corrosion engineer should be retained to provide corrosion protection measures for the anchors.

Verification Load Testing

Sacrificial load tests (often termed pre-production load tests or verification tests) should be performed to verify the design and installation procedure for the uplift anchors prior to final design and construction of production anchors. These load tests are also needed to evaluate the anchor grout to ground bond stress for final design. The tests should be performed at three (3) locations to be determined by the designer and Geotechnical engineer.

Each anchor should be load tested in tension to at least 150 percent of the design load, per ASTM D 3689. The central reinforcing bar should be designed such that the maximum tensile stress does not exceed 80 percent of the yield strength of the steel. The jack should be positioned at the beginning of the test such that unloading and repositioning of the jack during the test will not be required. Upon completion of the load testing, the geotechnical and structural engineers should evaluate the data obtained and provide final recommendations for the production anchors.

During production anchor construction, proof-load testing should be performed on all production anchors up to 133 percent of the design load, per FHWA guidelines.

Lock-off Loading

The magnitude of the lock-off load shall be specified by the designer and shall not exceed 70 percent of the steel yield strength.

Drilling Methods

The anchor drilling method should be selected by the Contractor and should be appropriate for the encountered soil and rock conditions and proposed grouting method. Caving conditions are not anticipated within the clayey on-site soils and claystone. However, some caving of sandy zones below groundwater could occur for uncased holes.

Additionally, groundwater was not encountered within the 2017 borings that extended to depths of approximately 40 feet. However, groundwater has been encountered in borings and trenches performed throughout the college campus at depths ranging between about 9 and 23 feet below the ground surface. Groundwater was not measured in the supplemental borings drilled for this addendum due to the use of mud rotary drilling methods.

Construction Observation and Monitoring

We recommend that all anchor construction and testing be monitored by a representative of Kleinfelder, including drilling, grout placement, and all verification and proof-load testing in accordance with Chapter 17 of the CBC and FHWA (1999) requirements. The purpose of these services would be to provide Kleinfelder the opportunity to observe the subsurface conditions encountered during construction, evaluate the applicability of the recommendations presented in this addendum letter to the subsurface conditions encountered, and prepare recommendations for final anchor design and construction.

LIMITATIONS

This letter is subject to the recommendations and provisions and requirements outlined in the limitations section of the referenced 2017 geotechnical investigation report. No warranty, express or implied, is made.

CLOSURE

Unless specifically superseded in this addendum, the recommendations presented in the above-referenced geotechnical report remain applicable. This document is intended to provide specific recommendations for design and construction of uplift anchors for the subject project. Accordingly, it cannot be considered an independent document, as it does not contain adequate background information. This document is directed only to the personnel with detailed knowledge of the subject project. The conclusions and recommendations presented in this addendum were prepared under the conditions and limitations presented in our above-referenced October 2017 geotechnical investigation report.

We trust this information meets your current needs. We appreciate the opportunity to be of professional service to you on this project. If you have any questions, please do not hesitate to contact us at (916) 366-1701.

Respectfully submitted,

KLEINFELDER, INC.

Rebecca L. Money, PE, GE
Senior Geotechnical Engineer

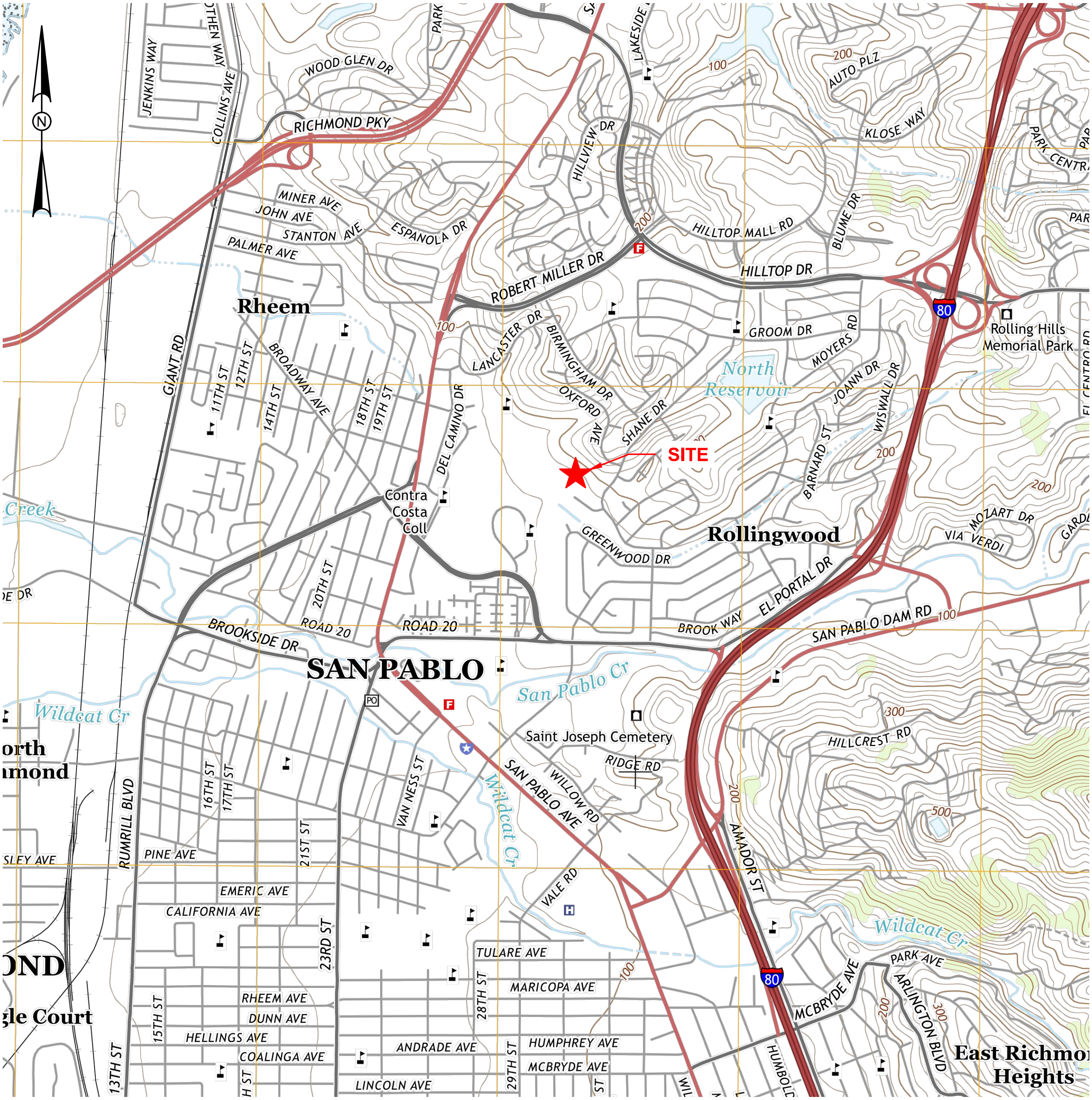
Kenneth G. Sorensen, PE, GE
Principal Geotechnical Engineer

Don Adams, PE
Project Manager

Attachments:
Figure 1 – Site Vicinity Map
Figure 2 – Site Plan
Appendix A – Logs of Borings B-1 through B-6
Appendix B – Core Photographs
Appendix C – Laboratory Test Results for this Addendum

KLEINFELDER 2882 Prospect Park Dr., Suite 200, Rancho Cordova, CA 95670 p | 916.366-1701 f | 916.366-7013

FIGURES



REFERENCE:

VICINITY MAP CREATED FROM DATA COMPILED BY USGS US TOPO RICHMOND, CA. QUADRANGLE 7.5-MINUTE, 2015

0 2000 4000

SCALE: 1" = 2000' SCALE IN FEET

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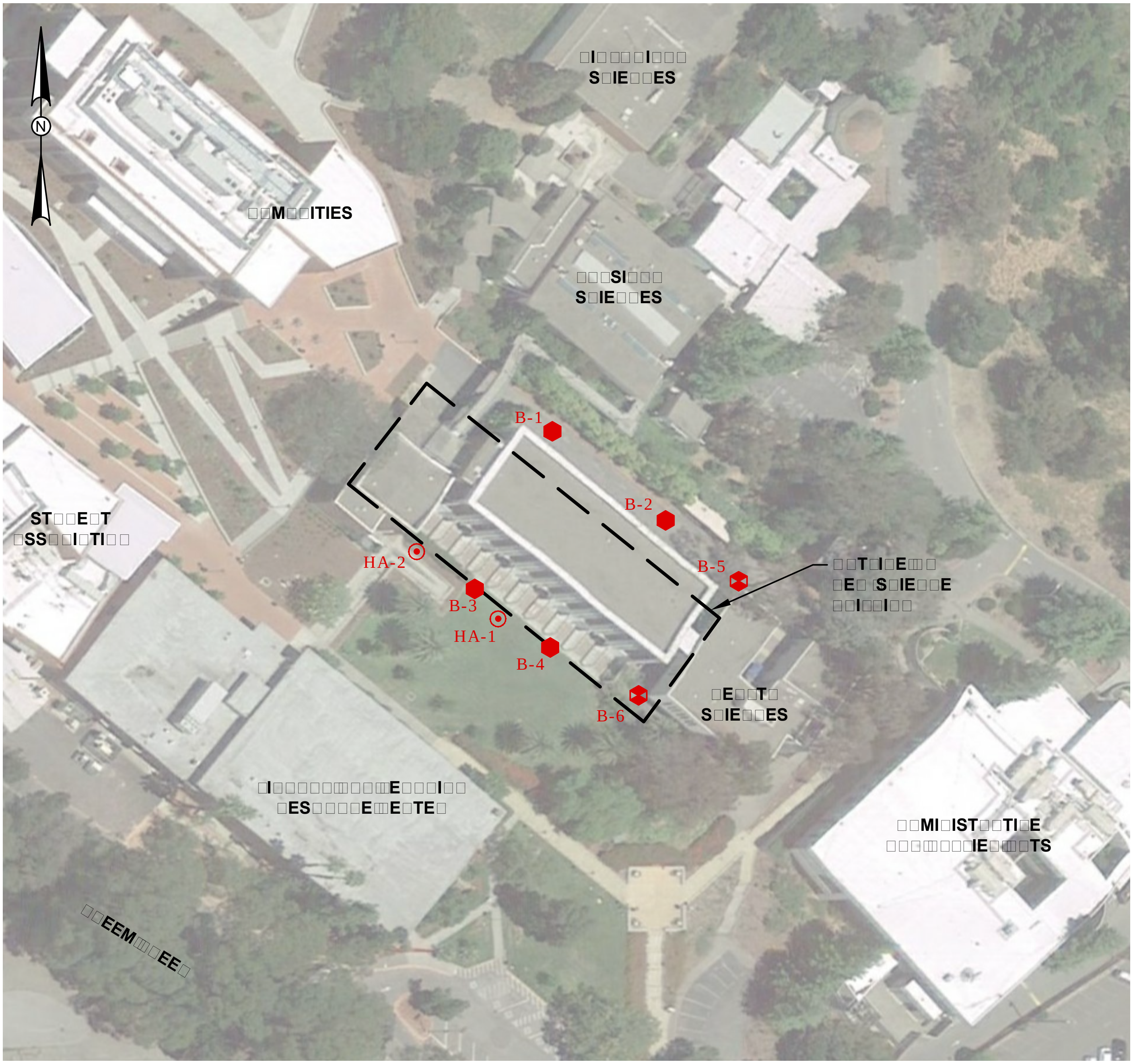
PROJECT NO.	20181569
DRAWN BY:	JDS
CHECKED BY:	JA
DATE:	08/30/2017
REVISED:	

SITE VICINITY MAP

CONTRA COSTA COMMUNITY COLLEGE
NEW SCIENCE BUILDING
2600 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA

FIGURE

1



0 100 200

SCALE: 1" = 100' SCALE IN FEET

REFERENCE:

IMAGE CREATED FROM DATA COMPILED BY GOOGLE EARTH PRO., DATED 5-20-2017

LEGEND

- SOIL BORING (By Kleinfelder, 2018)
- HAND AUGER (By Kleinfelder, 2018)
- SOIL BORING (By Kleinfelder, 2017)

NOTE: All locations are approximate.

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DATE:	12/12/2018
REVISED:	

SITE PLAN

CONTRA COSTA COMMUNITY COLLEGE
NEW SCIENCE BUILDING
2600 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA

FIGURE

2

PLOTTED: 12/12/2018 11:45 AM BY: jeff sala

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APPENDIX A
LOGS OF BORINGS B-1 THROUGH B-6

SAMPLER AND DRILLING METHOD GRAPHICS

- BULK / GRAB / BAG SAMPLE
- MODIFIED CALIFORNIA SAMPLER (2 or 2-1/2 in. (50.8 or 63.5 mm.) outer diameter)
- CALIFORNIA SAMPLER (3 in. (76.2 mm.) outer diameter)
- STANDARD PENETRATION SPLIT SPOON SAMPLER (2 in. (50.8 mm.) outer diameter and 1-3/8 in. (34.9 mm.) inner diameter)
- SHELBY TUBE SAMPLER
- HOLLOW STEM AUGER
- SOLID STEM AUGER
- WASH BORING

GROUND WATER GRAPHICS

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

NOTES

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

ABBREVIATIONS
WOH - Weight of Hammer
WOR - Weight of Rod

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487)

Major Division	Group	Fines	Criteria	Symbol	Description
COARSE GRAINED SOILS (More than half of material is larger than the #200 sieve)	**GRAVELS** (More than half of coarse fraction is larger than the #4 sieve)	CLEAN GRAVEL WITH <5% FINES	Cu ≥4 and 1≤ Cc≤3	**GW**	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
			Cu <4 and/ or 1>Cc >3	**GP**	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
		GRAVELS WITH 5% TO 12% FINES	Cu ≥4 and 1≤ Cc≤3	**GW-GM**	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GW-GC	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
			Cu <4 and/ or 1>Cc >3	**GP-GM**	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE FINES
				GP-GC	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE CLAY FINES
		GRAVELS WITH > 12% FINES		**GM**	SILTY GRAVELS, GRAVEL-SILT-SAND MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
				GC-GM	CLAYEY GRAVELS, GRAVEL-SAND-CLAY-SILT MIXTURES
	SANDS (More than half of coarse fraction is smaller than the #4 sieve)	CLEAN SANDS WITH <5% FINES	Cu ≥6 and 1≤ Cc≤3	**SW**	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
			Cu <6 and/ or 1>Cc >3	**SP**	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
		SANDS WITH 5% TO 12% FINES	Cu ≥6 and 1≤ Cc≤3	**SW-SM**	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SW-SC	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
			Cu <6 and/ or 1>Cc >3	**SP-SM**	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE FINES
				SP-SC	POORLY GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE CLAY FINES
		SANDS WITH > 12% FINES		**SM**	SILTY SANDS, SAND-GRAVEL-SILT MIXTURES
				SC	CLAYEY SANDS, SAND-GRAVEL-CLAY MIXTURES
				SC-SM	CLAYEY SANDS, SAND-SILT-CLAY MIXTURES

Major Division	Group	Symbol	Description
FINE GRAINED SOILS (More than half of material is smaller than the #200 sieve)	SILTS AND CLAYS (Liquid Limit less than 50)	**ML**	INORGANIC SILTS AND VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS, SILTS WITH SLIGHT PLASTICITY
		CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		CL-ML	INORGANIC CLAYS-SILTS OF LOW PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS (Liquid Limit greater than 50)	**MH**	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILT
		CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
		OH	ORGANIC CLAYS & ORGANIC SILTS OF MEDIUM-TO-HIGH PLASTICITY

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PROJECT NO.:	20181569
DRAWN BY:	MAP/JDS
CHECKED BY:	EBM
DATE:	11/26/2018
REVISED:	-

GRAPHICS KEY

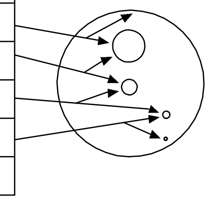
CONTRA COSTA COMMUNITY COLLEGE
NEW SCIENCE BUILDING
2600 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA

FIGURE
A-1

PLOTTED: 11/26/2018 10:43 AM BY: JSala
OFFICE FILTER: PLEASANTON
PROJECT NUMBER: 20181569.001A
gINT FILE: Klf_gint_master_2017
gINT TEMPLATE: E:KLF_STANDARD_GINT_LIBRARY_2017.GLB [LEGEND 1 (GRAPHICS KEY) USCS_STANDARD]

GRAIN SIZE

DESCRIPTION		SIEVE SIZE	GRAIN SIZE	APPROXIMATE SIZE
Boulders		>12 in. (304.8 mm.)	>12 in. (304.8 mm.)	Larger than basketball-sized
Cobbles		3 - 12 in. (76.2 - 304.8 mm.)	3 - 12 in. (76.2 - 304.8 mm.)	Fist-sized to basketball-sized
Gravel	coarse	3/4 -3 in. (19 - 76.2 mm.)	3/4 -3 in. (19 - 76.2 mm.)	Thumb-sized to fist-sized
	fine	#4 - 3/4 in. (#4 - 19 mm.)	0.19 - 0.75 in. (4.8 - 19 mm.)	Pea-sized to thumb-sized
Sand	coarse	#10 - #4	0.079 - 0.19 in. (2 - 4.9 mm.)	Rock salt-sized to pea-sized
	medium	#40 - #10	0.017 - 0.079 in. (0.43 - 2 mm.)	Sugar-sized to rock salt-sized
	fine	#200 - #40	0.0029 - 0.017 in. (0.07 - 0.43 mm.)	Flour-sized to sugar-sized
Fines		Passing #200	<0.0029 in. (<0.07 mm.)	Flour-sized and smaller



SECONDARY CONSTITUENT

	AMOUNT	
Term of Use	Secondary Constituent is Fine Grained	Secondary Constituent is Coarse Grained
Trace	<5%	<15%
With	≥5 to <15%	≥15 to <30%
Modifier	≥15%	≥30%

MOISTURE CONTENT

DESCRIPTION	FIELD TEST
Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, usually soil is below water table

CEMENTATION

DESCRIPTION	FIELD TEST
Weakly	Crumbles or breaks with handling or slight finger pressure
Moderately	Crumbles or breaks with considerable finger pressure
Strongly	Will not crumble or break with finger pressure

CONSISTENCY - FINE-GRAINED SOIL

CONSISTENCY	SPT - N_{60} (# blows / ft)	Pocket Pen (tsf)	UNCONFINED COMPRESSIVE STRENGTH (Q_u)(psf)	VISUAL / MANUAL CRITERIA
Very Soft	<2	PP < 0.25	<500	Thumb will penetrate more than 1 inch (25 mm). Extrudes between fingers when squeezed.
Soft	2 - 4	0.25 ≤ PP <0.5	500 - 1000	Thumb will penetrate soil about 1 inch (25 mm). Remolded by light finger pressure.
Medium Stiff	4 - 8	0.5 ≤ PP <1	1000 - 2000	Thumb will penetrate soil about 1/4 inch (6 mm). Remolded by strong finger pressure.
Stiff	8 - 15	1 ≤ PP <2	2000 - 4000	Can be imprinted with considerable pressure from thumb.
Very Stiff	15 - 30	2 ≤ PP <4	4000 - 8000	Thumb will not indent soil but readily indented with thumbnail.
Hard	>30	4 ≤ PP	>8000	Thumbnail will not indent soil.

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

REACTION WITH HYDROCHLORIC ACID

DESCRIPTION	FIELD TEST
None	No visible reaction
Weak	Some reaction, with bubbles forming slowly
Strong	Violent reaction, with bubbles forming immediately

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	SPT-N_{60} (# blows/ft)	MODIFIED CA SAMPLER (# blows/ft)	CALIFORNIA SAMPLER (# blows/ft)	RELATIVE DENSITY (%)
Very Loose	<4	<4	<5	0 - 15
Loose	4 - 10	5 - 12	5 - 15	15 - 35
Medium Dense	10 - 30	12 - 35	15 - 40	35 - 65
Dense	30 - 50	35 - 60	40 - 70	65 - 85
Very Dense	>50	>60	>70	85 - 100

FROM TERZAGHI AND PECK, 1948

PLASTICITY

DESCRIPTION	LL	FIELD TEST
Non-plastic	NP	A 1/8-in. (3 mm.) thread cannot be rolled at any water content.
Low (L)	< 30	The thread can barely be rolled and the lump or thread cannot be formed when drier than the plastic limit.
Medium (M)	30 - 50	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.
High (H)	> 50	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.

STRUCTURE

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

ANGULARITY

DESCRIPTION	CRITERIA
Angular	Particles have sharp edges and relatively plane sides with unpolished surfaces.
Subangular	Particles are similar to angular description but have rounded edges.
Subrounded	Particles have nearly plane sides but have well-rounded corners and edges.
Rounded	Particles have smoothly curved sides and no edges.

KLEINFELDER Bright People. Right Solutions.	PROJECT NO.: 20181569 DRAWN BY: MAP/JDS CHECKED BY: OK DATE: 9/19/2017 REVISED: -	SOIL DESCRIPTION KEY CONTRA COSTA COMMUNITY COLLEGE NEW SCIENCE BUILDING 2600 MISSION BELL DRIVE SAN PABLO, CALIFORNIA	FIGURE A-2

INFILLING TYPE

NAME	ABBR	NAME	ABBR
Albite	Al	Muscovite	Mus
Apatite	Ap	None	No
Biotite	Bi	Partially filled	Pa
Clay	Cl	Quartz	Qz
Calcite	Ca	Sand	Sd
Chlorite	Ch	Sericite	Ser
Epidote	Ep	Silt	Si
Iron Oxide	Fe	Talc	Ta
Manganese	Mn	Unknown	Uk

DENSITY/SPACING OF DISCONTINUITIES

DESCRIPTION	SPACING CRITERIA
Unfractured	>6 ft. (>1.83 meters)
Slightly Fractured	2 - 6 ft. (0.061 - 1.83 meters)
Moderately Fractured	8 in - 2 ft. (203.20 - 609.60 mm)
Highly Fractured	2 - 8 in (50.80 - 203.30 mm)
Intensely Fractured	<2 in (<50.80 mm)

ADDITIONAL TEXTURAL ADJECTIVES

DESCRIPTION	RECOGNITION
Pit (Pitted)	Pinhole to 0.03 ft. (3/8 in.) (>1 to 10 mm.) openings
Vug (Vuggy)	Small openings (usually lined with crystals) ranging in diameter from 0.03 ft. (3/8 in.) to 0.33 ft. (4 in.) (10 to 100 mm.)
Cavity	An opening larger than 0.33 ft. (4 in.) (100 mm.), size descriptions are required, and adjectives such as small, large, etc., may be used
Honeycombed	If numerous enough that only thin walls separate individual pits or vugs, this term further describes the preceding nomenclature to indicate cell-like form.
Vesicle (Vesicular)	Small openings in volcanic rocks of variable shape and size formed by entrapped gas bubbles during solidification.

ADDITIONAL TEXTURAL ADJECTIVES

DESCRIPTION	CRITERIA
Unweathered	No evidence of chemical / mechanical alternation; rings with hammer blow.
Slightly Weathered	Slight discoloration on surface; slight alteration along discontinuities; <10% rock volume altered.
Moderately Weathered	Discoloring evident; surface pitted and alteration penetration well below surface; Weathering "halos" evident; 10-50% rock altered.
Highly Weathered	Entire mass discolored; Alteration pervading most rock, some slight weathering pockets; some minerals may be leached out.
Decomposed	Rock reduced to soil with relic rock texture/structure; Generally molded and crumbled by hand.

RELATIVE HARDNESS / STRENGTH DESCRIPTIONS

GRADE		UCS (Mpa)	FIELD TEST
R0	Extremely Weak	0.25 - 1.0	Indented by thumbnail
R1	Very Weak	1.0 - 5.0	Crumbles under firm blows of geological hammer, can be peeled by a pocket knife.
R2	Weak	5.0 - 25	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer.
R3	Medium Strong	25 - 50	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single firm blow of a geological hammer.
R4	Strong	50 - 100	Specimen requires more than one blow of geological hammer to fracture it.
R5	Very Strong	100 - 250	Specimen requires many blows of geological hammer to fracture it.
R6	Extremely Strong	> 250	Specimen can only be chipped with a geological hammer.

ROCK QUALITY DESIGNATION (RQD)

DESCRIPTION	RQD (%)
Very Poor	0 - 25
Poor	25 - 50
Fair	50 - 75
Good	75 - 90
Excellent	90 - 100

APERTURE

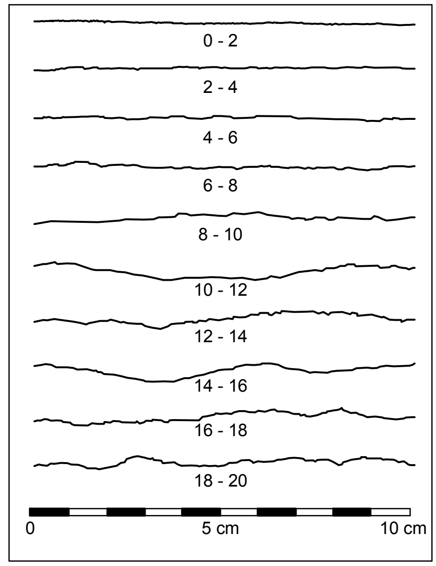
DESCRIPTION	CRITERIA [in (mm)]
Tight	<0.04 (<1)
Open	0.04 - 0.20 (1 - 5)
Wide	>0.20 (>5)

BEDDING CHARACTERISTICS

DESCRIPTION	Thickness [in (mm)]
Very Thick Bedded	>36 (>915)
Thick Bedded	12 - 36 (305 - 915)
Moderately Bedded	4 - 12 (102 - 305)
Thin Bedded	1 - 4 (25 - 102)
Very Thin Bedded	0.4 - 1 (10 - 25)
Laminated	0.1 - 0.4 (2.5 - 10)
Thinly Laminated	<0.1 (<2.5)

Bedding Planes — Planes dividing the individual layers, beds, or stratigraphy of rocks.
Joint — Fracture in rock, generally more or less vertical or traverse to bedding.
Seam — Applies to bedding plane with unspecified degree of weather.

JOINT ROUGHNESS COEFFICIENT (JRC)



From Barton and Choubey, 1977

RQD — Rock-quality designation (RQD) Rough measure of the degree of jointing or fracture in a rock mass, measured as a percentage of the drill core in lengths of 10 cm. or more.

CORE SAMPLER TYPE GRAPHICS

- CORE SAMPLER
- AQ CORE BARREL (1.067 in. (27.1 mm.) core diameter)
- AX CORE BARREL (1.185 in. (30.1 mm.) core diameter)
- BQ CORE BARREL (1.433 in. (36.4 mm.) core diameter)
- CONTINUOUS CORE SAMPLE (2.000 in. (50.8 mm.) core diameter)
- EX CORE BARREL (0.846 in. (21.5 mm.) core diameter)
- HQ CORE SAMPLE (2.500 in. (63.5 mm.) core diameter)
- NQ CORE SAMPLE (1.874 in. (47.6 mm.) core diameter)
- NO RECOVERY CORE SAMPLE
- NX CORE SAMPLE (2.154 in. (54.7 mm.) core diameter)

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PROJECT NO.: 20181569	ROCK DESCRIPTION KEY	FIGURE	
DRAWN BY: MAP/JDS			
CHECKED BY: OK	CONTRA COSTA COMMUNITY COLLEGE NEW SCIENCE BUILDING 2600 MISSION BELL DRIVE SAN PABLO, CALIFORNIA	A-3	
DATE: 9/19/2017			
REVISED: -			

BORING LOG B-1

Date Begin - End:	8/11/2017	**Drilling Co.-Lic.#:**	Gregg - #CA107979		
Logged By:	J. Anderson	**Drill Crew:**	Jeremy		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Truck Mounted M11	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Cloudy	**Exploration Diameter:**	Approx. 6 in.		

Latitude: 37.96986° N
Longitude: -122.33678° E
Approximate Ground Surface Elevation (ft.): 92.00
Surface Condition: Asphalt

FIELD EXPLORATION							LABORATORY RESULTS							
Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
	0		approximately 2-inches of asphalt											
			Sandy Lean CLAY with Gravel (CL): low plasticity, yellowish brown, moist, stiff to very stiff, subrounded to subangular gravel		BC=5 7 9	12"		18.9	109.7					
90														
	5		olive brown, stiff to very stiff		BC=5 6 8	12"		19.1	108.8					TXUU: c = 2.12 ksf
85														
	10		**Sandy Lean CLAY (CL)**: fine-grained sand, some gravel, medium plasticity, reddish yellow mottled, moist, very stiff		BC=6 10 14	12"		14.0	115.8					TXUU: c = 2.55 ksf
80														
			some angular claystone fragments, yellowish brown, hard											
	15				BC=12 18 22	12"								
75														
	20		**CLAYSTONE**: fine-grained, medium plasticity, yellowish brown, moderately weathered, weak to medium strong		BC=22 36 50/5"	11"								
70														
	25				BC=11 29 50	12"								
65														
	30		moderately weathered, weak to medium strong, interbedded with siltstone		BC=29 50/3"	8"								
60														

Date Begin - End: 8/11/2017	**Drilling Co.-Lic.#:** Gregg - #CA107979		**BORING LOG B-1**
Logged By: J. Anderson	**Drill Crew:** Jeremy		
Hor.-Vert. Datum: Not Available	**Drilling Equipment:** Truck Mounted M11	**Hammer Type - Drop:** 140 lb. Auto - 30 in.	
Plunge: -90 degrees	**Drilling Method:** Hollow Stem Auger		
Weather: Cloudy	**Exploration Diameter:** Approx. 6 in.		

FIELD EXPLORATION | LABORATORY RESULTS

Latitude: 37.96986° N
Longitude: -122.33678° E
Approximate Ground Surface Elevation (ft.): 92.00
Surface Condition: Asphalt

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
			CLAYSTONE: fine-grained, yellowish brown, moderately weathered, medium strong		BC=26 50	2"								
55														
	40		- light brownish gray, slightly weathered, medium strong to strong		BC=44 50/2"	8"								
50														
	45													
45														
	50													
40														
	55													
35														
	60													
30														
	65													
25														

The boring was terminated at approximately 40.5 ft. below ground surface. The boring was backfilled with cement grout on August 11, 2017.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.
GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder.

Date Begin - End:	8/11/2017	Drilling Co.-Lic.#:	Gregg - #CA107979		BORING LOG B-2
Logged By:	J. Anderson	Drill Crew:	Jeremy		
Hor.-Vert. Datum:	Not Available	Drilling Equipment:	Truck Mounted M11	Hammer Type - Drop:	140 lb. Auto - 30 in.
Plunge:	-90 degrees	Drilling Method:	Hollow Stem Auger		
Weather:	Cloudy	Exploration Diameter:	Approx. 6 in.		

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 37.96973° N
Longitude: -122.33647° E
Approximate Ground Surface Elevation (ft.): 93.00
Surface Condition: Asphalt

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
			approximately 2-inches of asphalt											
			Clayey SAND (SC): fine to medium-grained sand, low plasticity, mottled yellowish brown, dry, medium dense		BC=10 12 14	12"								
90			**Lean CLAY (CL)**: medium plasticity, yellowish brown, moist, very stiff					11.3	110.8					
	5		**CLAYSTONE**: fine-grained, yellowish brown, moderately weathered to highly weathered, weak to medium strong		BC=17 18 26	6"								
85														
			reddish yellow, fragmented moderately weathered, weak to medium strong											
	10				BC=16 14 50/4"	10"		9.5	118.9					
80														
	15		olive brown, weak to medium strong		BC=14 36 50/5"	2"								Very hard drilling
75														
	20		- yellowish brown with reddish brown stains, moderately weathered, intensely fractured medium strong		BC=23 50	4"								
70														
	25		weak		BC=13 14 20	2"								
65														
	30		medium-grained, yellow, moderately weathered, weak, highly fractured, interbedded with subrounded gravel		BC=11 18 34	10"								
60														

Date Begin - End:	8/11/2017	**Drilling Co.-Lic.#:**	Gregg - #CA107979		**BORING LOG B-2**
Logged By:	J. Anderson	**Drill Crew:**	Jeremy		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Truck Mounted M11	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Cloudy	**Exploration Diameter:**	Approx. 6 in.		

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 37.96973° N
Longitude: -122.33647° E
Approximate Ground Surface Elevation (ft.): 93.00
Surface Condition: Asphalt

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
55			**CLAYSTONE**: fine-grained, yellowish brown, moderately weathered to highly weathered, weak to medium strong fine-grained, light brownish gray, weak to medium strong, highly fractured		BC=9 29 50/5"	3"								
	40				BC=21 50									

The boring was terminated at approximately 41 ft. below ground surface. The boring was backfilled with cement grout on August 11, 2017.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.
GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder.

Elevation scale: 50, 45, 40, 35, 30, 25
Depth scale: 45, 50, 55, 60, 65

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PROJECT NO.: 20181569	BORING LOG B-2	FIGURE
DRAWN BY: MAP/JDS		A-5
CHECKED BY: OK	CONTRA COSTA COMMUNITY COLLEGE NEW SCIENCE BUILDING 2600 MISSION BELL DRIVE SAN PABLO, CALIFORNIA	
DATE: 9/19/2017		
REVISED: -		PAGE: 2 of 2

Date Begin - End: 8/18/2017
Drilling Co.-Lic.#: Gregg - #CA107979
BORING LOG B-3

Logged By: J. Anderson
Drill Crew: Jeremy and Leo

Hor.-Vert. Datum: Not Available
Drilling Equipment: D42
Hammer Type - Drop: 140 lb. Auto - 30 in.

Plunge: -90 degrees
Drilling Method: Hollow Stem Auger

Weather: Overcast
Exploration Diameter: Approx. 6 in.

FIELD EXPLORATION / LABORATORY RESULTS

Latitude: 37.96965° N
Longitude: -122.33695° E
Approximate Ground Surface Elevation (ft.): 80.00
Surface Condition: Grass

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
			Sandy Lean CLAY (CL): medium plasticity, olive brown, moist, very stiff, (FILL)		BC=3 8 13	12"						27	12	
75	5		**Lean CLAY with Sand (CL)**: medium plasticity, olive brown, moist, very stiff, (FILL)		BC=4 8 12	11"								
			Sandy Lean CLAY (CL): medium plasticity, yellowish brown, moist, stiff											
70	10				BC=2 4 7	12"		26.8	94.7					TXUU: c = 1.25 ksf
65	15		**Clayey SAND (SC)**: non-plastic to low plasticity, yellowish brown, moist, loose		BC=4 4 5	12"	SC				49	33	18	
60	20		**CLAYSTONE**: fine-grained, olive brown, weak to medium strong, interbedded with siltstone		BC=20 42 50/5"	11"								
55	25		light gray, medium strong to strong		BC=40 50/5"	11"								
50	30		moderately to slightly weathered, weak, highly fractured		BC=20 25 26	12"								

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PROJECT NO.: 20181569
DRAWN BY: MAP/JDS
CHECKED BY: OK
DATE: 9/19/2017
REVISED: -

BORING LOG B-3

CONTRA COSTA COMMUNITY COLLEGE
NEW SCIENCE BUILDING
2600 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA

FIGURE
A-6

Date Begin - End: 8/18/2017	**Drilling Co.-Lic.#:** Gregg - #CA107979		**BORING LOG B-3**
Logged By: J. Anderson	**Drill Crew:** Jeremy and Leo		
Hor.-Vert. Datum: Not Available	**Drilling Equipment:** D42	**Hammer Type - Drop:** 140 lb. Auto - 30 in.	
Plunge: -90 degrees	**Drilling Method:** Hollow Stem Auger		
Weather: Overcast	**Exploration Diameter:** Approx. 6 in.		

Latitude: 37.96965° N
Longitude: -122.33695° E
Approximate Ground Surface Elevation (ft.): 80.00
Surface Condition: Grass

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
			CLAYSTONE: fine-grained, olive gray, weak		BC=18 27 30	12"								
40	40		olive, medium strong		BC=17 36 50/5"									
35	45													
30	50													
25	55													
20	60													
15	65													

The boring was terminated at approximately 41.5 ft. below ground surface. The boring was backfilled with cement grout on August 18, 2017.

GROUNDWATER LEVEL INFORMATION:
Groundwater was not observed during drilling or after completion.
GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder.

Date Begin - End:	8/18/2017	**Drilling Co.-Lic.#:**	Gregg - #CA107979		**BORING LOG B-4**
Logged By:	J. Anderson	**Drill Crew:**	Jeremy and Leo		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	D42	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Hollow Stem Auger		
Weather:	Overcast	**Exploration Diameter:**	Approx. 6 in.		

Latitude: 37.96953° N
Longitude: -122.33673° E
Approximate Ground Surface Elevation (ft.): 80.00
Surface Condition: Grass

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description (FIELD EXPLORATION)	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol (LABORATORY RESULTS)	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
			Lean Fat CLAY with Sand (CL): medium to high plasticity, olive brown, moist, hard, (FILL)		BC=11 13 16 PP=4-4.5+	11"								
75	5		**Lean CLAY with Sand (CL)**: medium plasticity, olive brown, moist, hard, (FILL)		BC=9 12 23 PP=4.5	12"						43	28	
70	10		increase in sand content, very stiff, organics, brick fragments with gravel and brick at 11.5 feet		BC=9 11 12 PP=1.5-1.75									
			Clayey GRAVEL with Sand (GC): dark brown, moist, medium dense, fine to coarse gravel											
65	15		**Clayey SAND with Gravel (SC)**: medium to coarse-grained, olive brown, moist, medium dense		BC=17 18 12						16			
60	20		**Sandy CLAYSTONE**: fine-grained, olive, weak to medium strong, moderately weathered, interbedded with siltstone		BC=20 27 25	12"								
55	25		medium strong		BC=18 33 48	12"								
50	30		medium strong to strong		BC=27 50/5"									

The boring was terminated at approximately 31 ft. below ground surface. The boring was backfilled with cement grout on August 18, 2017.

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

GENERAL NOTES:
The exploration location and elevation are approximate and were estimated by Kleinfelder.

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PROJECT NO.:	20181569
DRAWN BY:	MAP/JDS
CHECKED BY:	OK
DATE:	9/19/2017
REVISED:	-

BORING LOG B-4

CONTRA COSTA COMMUNITY COLLEGE
NEW SCIENCE BUILDING
2600 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA

FIGURE
A-7

BORING LOG B-5

Date Begin - End:	11/09/2018	**Drilling Company:**	Pitcher
Logged By:	M. Beckman	**Drill Crew:**	Willy/ Elvis
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Track Fraste Multidrill XL
Plunge:	-90 degrees	**Drilling Method:**	Mud Rotary
Weather:	Sunny	**Exploration Diameter:**	5 in. O.D.

Hammer Type - Drop: 140 lb. Auto - 30 in.

FIELD EXPLORATION | LABORATORY RESULTS

Latitude: °
Longitude: °
Approximate Ground Surface Elevation (ft.): 91.00
Surface Condition: Fill

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
90			**Sandy CLAY (CL)**: fine to coarse-grained sand, low plasticity, yellow brown, moist, firm, trace fine to coarse gravel (FILL)											hand augered to 5 feet set casing to 3.5 feet
85	5													
					BC=4 11 12									
80	10													
75	15													
70	20													
65	25		**Lean CLAY (CL)**: olive gray and reddish brown mottled, trace sand, decomposed claystone, relict rock structure											
60	30				BC=10 16 20									

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PROJECT NO.:	20181569
DRAWN BY:	JDS
CHECKED BY:	EMB
DATE:	11/26/2018
REVISED:	-

BORING LOG B-5

CONTRA COSTA COMMUNITY COLLEGE
NEW SCIENCE BUILDING
2600 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA

FIGURE
A-8

Date Begin - End:	11/09/2018	**Drilling Company:**	Pitcher		**BORING LOG B-5**
Logged By:	M. Beckman	**Drill Crew:**	Willy/ Elvis		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Track Fraste Multidrill XL	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Mud Rotary		
Weather:	Sunny	**Exploration Diameter:**	5 in. O.D.		

Latitude: °
Longitude: °
Approximate Ground Surface Elevation (ft.): 91.00
Surface Condition: Fill

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description (FIELD EXPLORATION)	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol (LABORATORY RESULTS)	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
55			**Lean CLAY (CL)**: olive gray and reddish brown mottled, trace sand, decomposed claystone, relict rock structure yellowish brown, some white staining		BC=26 37 35									fluid loss
50	40		**SANDSTONE**: olive yellow, fine-grained sand with clay, R0, decomposed to residual soil		BC=17 25 22									
45	45		**See next page for continuation of boring log - rock sampling/coring data begins next page.**											
40	50													
35	55													
30	60													
25	65													

BORING LOG B-5

Date Begin - End:	11/09/2018	**Drilling Company:**	Pitcher
Logged By:	M. Beckman	**Drill Crew:**	Willy/ Elvis
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Track Fraste Multidrill XL
Plunge:	-90 degrees	**Coring Method:**	Coring
Weather:	Sunny	**Core Bit Type:**	

ROCK CORING INFORMATION

Latitude: °
Longitude: °
Approximate Ground Surface Elevation (ft.): 91.00

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Formation and Rock Type, Color, Grain/Particle Size, Weathering, Bedding, Density or Spacing	Box Number	Run Number	Sample Type	Recovery (NR=No Recovery)	Drill Rate (min/ft)	RQD (%)	Relative Strength	Discontinuity Description Fracture#: (Depth), Type, Relative Dip, Density or Spacing. Degree of Infilling, Infilling Type, Aperture, Surface Weathering, JRC
			SANDSTONE: olive yellow, fine-grained sand with clay, decomposed to residual soil Lense of decomposed claystone at 43.7' to 44.7'	1	1		44"	0.00	0	R0	
45	45		**GREYWACKE**: olive brown, fine to medium grained sand (rounded), decomposed, intensely fractured Gray, medium to coarse sand at 46.5							R0	(45'), shear, 20°, pa, Cl/Sd, tight, JRC=4-6 (45.1'), shear, 20°, pa, Cl/Sd, tight, JRC=4-6 (45.3'), shear, 40°, filled, Cl, tight, JRC=4-6 (45.4'), shear, 40°, healed, Cl, tight, JRC=6-8 (46.5'), shear, 30°, pa, Cl/Sd, open, JRC=6-8 (46.8'), shear, 30°, pa, Cl/Sd, open, JRC=4-6
40	50										
35	55										
30	60										
25	65										
20	70										
15	75										

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PROJECT NO.: 20181569	BORING LOG B-5	FIGURE
DRAWN BY: JDS		
CHECKED BY: EMB	CONTRA COSTA COMMUNITY COLLEGE NEW SCIENCE BUILDING 2600 MISSION BELL DRIVE SAN PABLO, CALIFORNIA	A-8
DATE: 11/26/2018		
REVISED: -		PAGE: 1 of 1

Date Begin - End:	11/07/2018 - 11/08/2018	**Drilling Company:**	Pitcher		**BORING LOG B-6**
Logged By:	M. Beckman	**Drill Crew:**	Will/ Willy		
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Track Fraste Multidrill XL	**Hammer Type - Drop:**	140 lb. Auto - 30 in.
Plunge:	-90 degrees	**Drilling Method:**	Mud Rotary/ HQ		
Weather:	Sunny	**Exploration Diameter:**	6 in. O.D.		

FIELD EXPLORATION — LABORATORY RESULTS

Latitude: °
Longitude: °
Approximate Ground Surface Elevation (ft.): 80.00
Surface Condition: Fill

Approximate Elevation (feet)	Depth (feet)	Graphical Log	Lithologic Description	Sample Type	Blow Counts(BC)= Uncorr. Blows/6 in. Pocket Pen(PP)= tsf	Recovery (NR=No Recovery)	USCS Symbol	Water Content (%)	Dry Unit Wt. (pcf)	Passing #4 (%)	Passing #200 (%)	Liquid Limit	Plasticity Index (NP=NonPlastic)	Additional Tests/ Remarks
			Lean CLAY with Sand (CL): medium plasticity, brown, moist, firm, (FILL)											cased hole to 3.5 feet
					BC=7 13 11									
					BC=6 9 13									
75	5													
			olive brown		BC=6 12 18									
70	10													
65	15													
60	20		**Clayey SAND with Gravel (SC)**: low plasticity, yellowish brown, moist, loose, (FILL)		BC=6 6 6									
55	25				BC=10 17 26									
			See next page for continuation of boring log - rock sampling/coring data begins next page.											
50	30													

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PROJECT NO.: 20181569	BORING LOG B-6		FIGURE
DRAWN BY: JDS			A-9
CHECKED BY: EMB	CONTRA COSTA COMMUNITY COLLEGE NEW SCIENCE BUILDING 2600 MISSION BELL DRIVE SAN PABLO, CALIFORNIA		
DATE: 11/26/2018			
REVISED: -			PAGE: 1 of 1

BORING LOG B-6

Date Begin - End:	11/07/2018 - 11/08/2018	**Drilling Company:**	Pitcher
Logged By:	M. Beckman	**Drill Crew:**	Will/ Willy
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Track Fraste Multidrill XL
Plunge:	-90 degrees	**Coring Method:**	Coring
Weather:	Sunny	**Core Bit Type:**	

ROCK CORING INFORMATION

Latitude: °
Longitude: °
Approximate Ground Surface Elevation (ft.): 80.00

Approximate Elevation (feet)	Depth (feet)	Formation and Rock Type, Color, Grain/Particle Size, Weathering, Bedding, Density or Spacing	Box Number	Run Number	Recovery (NR=No Recovery)	Drill Rate (min/ft)	RQD (%)	Relative Strength	Discontinuity Description: Fracture#: (Depth), Type, Relative Dip, Density or Spacing. Degree of Infilling, Infilling Type, Aperture, Surface Weathering, JRC
		CLAYSTONE: medium plasticity, moist, hard, completely weathered, residual soil							
		CLAYSTONE: orangish brown and gray, decomposed, laminated, intensely fractured	1	1	20"	0.00 0.00	0	R0	
50	30	Reddish brown and olive gray mottled, gray siltstone cobbles 30' to 30.7'		2	32"	0.00	0	R1-R2 R0-R1	
		Highly weathered to decomposed		3	25"	0.00	0		
45	35	Grades silty							
		SILTSTONE: olive gray to gray with reddish brown mottling, highly weathered to decomposed, intensely fractured, pervasive 25°-55° shears/fracturing, pronounced 30°-40° shears at 37', 37.1', 37.2', 37.4', 37.5', 38.4', 39.1', 39.6', 40.1'		4	50"	0.00	0	R0	
40	40		2			0.00			
				5	60"	0.00	0		(41.6'), shear, 10°, filled, Sd, wide, JRC=12-14
		GREYWACKE: olive gray and gray, highly weathered to decomposed, intensely fractured Silty inclusion at 43.6' to 43.8' Sandy zone at 44.6' to 45'							(41.8'), shear, 50°, filled, Cl, tight, JRC=6-8 (42.1'), shear, 10°, filled, Cl, open, JRC=6-8 (42.3'), shear, 50°, pa, Cl, tight, JRC=6-8 (42.5'), shear, 60°, pa, Cl, tight, JRC=6-8 (42.6'), shear, 80°, filled, Cl, tight, JRC=4-6 (42.7'), shear, 60°, pa, Cl, tight, JRC=4-6 (43'), shear, 60°, pa, Cl, tight, JRC=6-8 (43.2'), shear, 20°, pa, Sd/Cl, tight, JRC=6-8 (43.6'), shear, 50°, pa, Cl/Sd, open, JRC=6-8, slickensided (45'), shear, 50°, pa, Sd, tight, JRC=6-8
35	45	Greenish gray, 2-3" siltstone inclusions at 47.6', 47.9' Edges inclined at 50°-60°, signs of healed shears		6	54"	0.00	0		(48.8'), shear, 60°, filled/healed, Cl, open, JRC=4-6
			3			0.00			
30	50	**SILTSTONE**: olive gray and gray, highly weathered to decomposed Intensely fractured Biotite infill in shears, siltstone is pervasively sheared - spacing ~1/2"- 2", clay infill. Cross-cutting shears inclined 70°-80°.						R0	(50.8'), shear, 50°, JRC=4-6, bottom piece in next run
				7	61"	0.00	0		(52.6'), shear, 50°, filled/healed, Cl, open, JRC=4-6
25	55	Less sheared below 54.5'							(54.5'), shear, 50°, pa, Cl, open, JRC=4-6
		SILTSTONE/CLAYSTONE: gray, highly weathered to decomposed, intensely fractured, [shear zone]		8	11"	0.00	0	R0	
		SILTSTONE: gray, highly weathered to decomposed, intensely fractured, with trace sand		9	22"	0.00	0	R0	
20	60		4			0.00			

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CHECKED BY:	EMB
DATE:	11/26/2018
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BORING LOG B-6

CONTRA COSTA COMMUNITY COLLEGE
NEW SCIENCE BUILDING
2600 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA

FIGURE
A-9

BORING LOG B-6

Date Begin - End:	11/07/2018 - 11/08/2018	**Drilling Company:**	Pitcher
Logged By:	M. Beckman	**Drill Crew:**	Will/ Willy
Hor.-Vert. Datum:	Not Available	**Drilling Equipment:**	Track Fraste Multidrill XL
Plunge:	-90 degrees	**Coring Method:**	Coring
Weather:	Sunny	**Core Bit Type:**	

ROCK CORING INFORMATION

Latitude: °
Longitude: °
Approximate Ground Surface Elevation (ft.): 80.00

Approximate Elevation (feet)	Depth (feet)	Formation and Rock Type, Color, Grain/Particle Size, Weathering, Bedding, Density or Spacing	Box Number	Run Number	Sample Type	Recovery (NR=No Recovery)	Drill Rate (min/ft)	RQD (%)	Relative Strength	Discontinuity Description: Fracture#: (Depth), Type, Relative Dip, Density or Spacing. Degree of Infilling, Infilling Type, Aperture, Surface Weathering, JRC
			4	10		61"	0.00	0	R0	
		GREYWACKE: highly weathered to decomposed, intensely fractured to moderately fractured, with small gravel, brown staining to 62.4', grades finer 62.4 to 64							R0	
15	65	**SILTSTONE**: gray to olive gray, highly weathered to decomposed, moderately fractured							R0	(64.3'), shear, 40°, filled, Cl, open, JRC=4-6 (65'), shear, 40°, pa, Cl, open, JRC=4-6 (65.7'), shear, 40°, pa, Cl, tight, JRC=4-6
		Olive gray, yellowish brown staining		11		61"	0.00	0		(67.3'), shear, 30°, none, open, JRC=4-6 (68'), shear, 40°, filled, Cl, tight, JRC=2-4
		Clay inclusions at 68.5'. Cross cutting inclined shears at 68.8'.	5				0.00			
10	70	Grades clayey at 70'-71'								(70.5'), shear, 40°, filled, Sd, tight, JRC=2-4
		CLAYSTONE: olive gray, decomposed, moderately fractured, mottled		12		61"	0.00	0	R0	(72.3'), shear, 30°, filled, Cl, tight, JRC=6-8 (72.9'), shear, 60°, filled/healed, Cl, tight, JRC=4-6 (73.3'), joint, 40°, pa, Sd, open, JRC=8-10 (74'), shear, 50°, pa, Cl, tight, JRC=4-6
5	75	Pervasively weathered and shows relict sheared bedding structure Inclined clayey zone at 74.2' and 75.3'								
		Zone of gravel inclusion at 76.8' to 77.5' (sheared zone)		13		59"	0.00	0		
			6				0.00			(78'), shear, 60°, filled/healed, Cl, tight, JRC=2-4 (78.4'), shear, 20°, filled, Cl, tight, JRC=2-4 (78.9'), joint, 10°, pa, Sd, open, JRC=4-6
0	80									
		Highly weathered, moderately to highly fractured		14		59"	0.00	0		
		Olive, sandy shear zone at 82.1' inclined at 50°, greywacke inclusion at 81.5' Shear zone at 83.4' to 83.9' with calcite mineralization								(82.3'), shear, 50°, filled, Cl, open, JRC=4-6 (82.5'), shear, 20°, filled, Cl, open, JRC=4-6 (82.8'), shear, 50°, filled, Sd/Cl, open, JRC=4-6 (84'), shear, 20°, none, tight, JRC=4-6 (84.3'), shear, 30°, none, tight, JRC=4-6 (85'), shear, 70°, filled, Cl, open, JRC=4-6
-5	85									
			7	15		60"	0.00	0		(86.6'), shear, 70°, pa, Cl, open, JRC=6-8 (86.7'), shear, -60°, none, tight, JRC=8-10 (87'), shear, 70°, pa, Cl, tight, JRC=8-10 (87.1'), shear, 30°, pa, Cl, open, JRC=6-8 (87.3'), shear, 30°, pa, Cl, tight, JRC=2-4 (87.4'), shear, 50°, pa, Cl, tight, JRC=6-8 (87.5'), shear, 40°, pa, Cl, tight, JRC=2-4 (87.6'), shear, 40°, pa, Cl, tight, JRC=2-4 (88.2'), shear, 20°, pa, Cl, wide, JRC=2-4 (88.4'), shear, 20°, filled, Cl, wide, JRC=4-6 (88.9'), shear, 70°, none, tight, JRC=6-8 (89.1'), shear, 0°, pa, Cl, tight, JRC=2-4 (89.3'), contact, 0°
-10	90	**GREYWACKE**: olive, fine to coarse sand, fine gravel, from 89.3' to 91' grades to coarser material with less staining							R0	
-15	95									

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DRAWN BY:	JDS
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DATE:	11/26/2018
REVISED:	-

BORING LOG B-6

CONTRA COSTA COMMUNITY COLLEGE
NEW SCIENCE BUILDING
2600 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA

FIGURE
A-9

APPENDIX B
CORE PHOTOGRAPHS FOR BORINGS B-5 AND B-6

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FILE NAME:	ROCK CORE PHOTOS

BORING B-5 ROCK CORE

RUN 1 — 42.0—47.0 FT

C-4016 NEW SCIENCE BUILDING
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FIGURE

B-1

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FILE NAME:	ROCK CORE PHOTOS

BORING B-6 ROCK CORE

RUN 1 — 28.0—30.0 FT

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FIGURE

B-2

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FILE NAME:	ROCK CORE PHOTOS

BORING B-6 ROCK CORE

RUN 2 — 30.0—33.0 FT

C-4016 NEW SCIENCE BUILDING
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FIGURE

B-3

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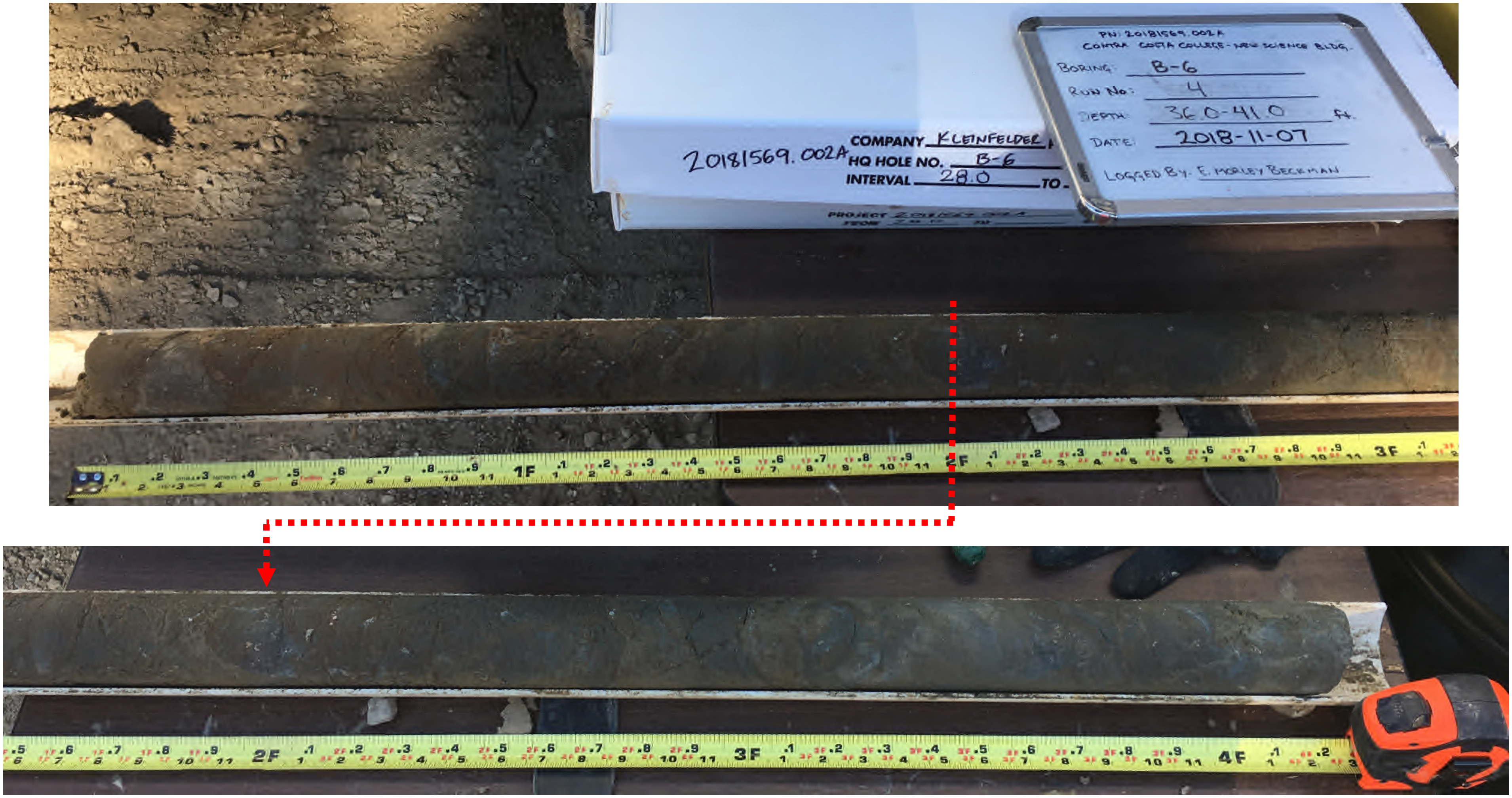
PROJECT NO. 20181569	**BORING B-6 ROCK CORE**	FIGURE
DRAWN: 12-03-18	**RUN 3 — 33.0—36.0 FT**	**B-4**
DRAWN BY: EMB	C-4016 NEW SCIENCE BUILDING	
CHECKED BY: RLM	CONTRA COSTA COLLEGE	
FILE NAME: ROCK CORE PHOTOS	SAN PABLO, CALIFORNIA	



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FILE NAME:	ROCK CORE PHOTOS

BORING B-6 ROCK CORE

RUN 4 — 36.0—41.0 FT

C-4016 NEW SCIENCE BUILDING
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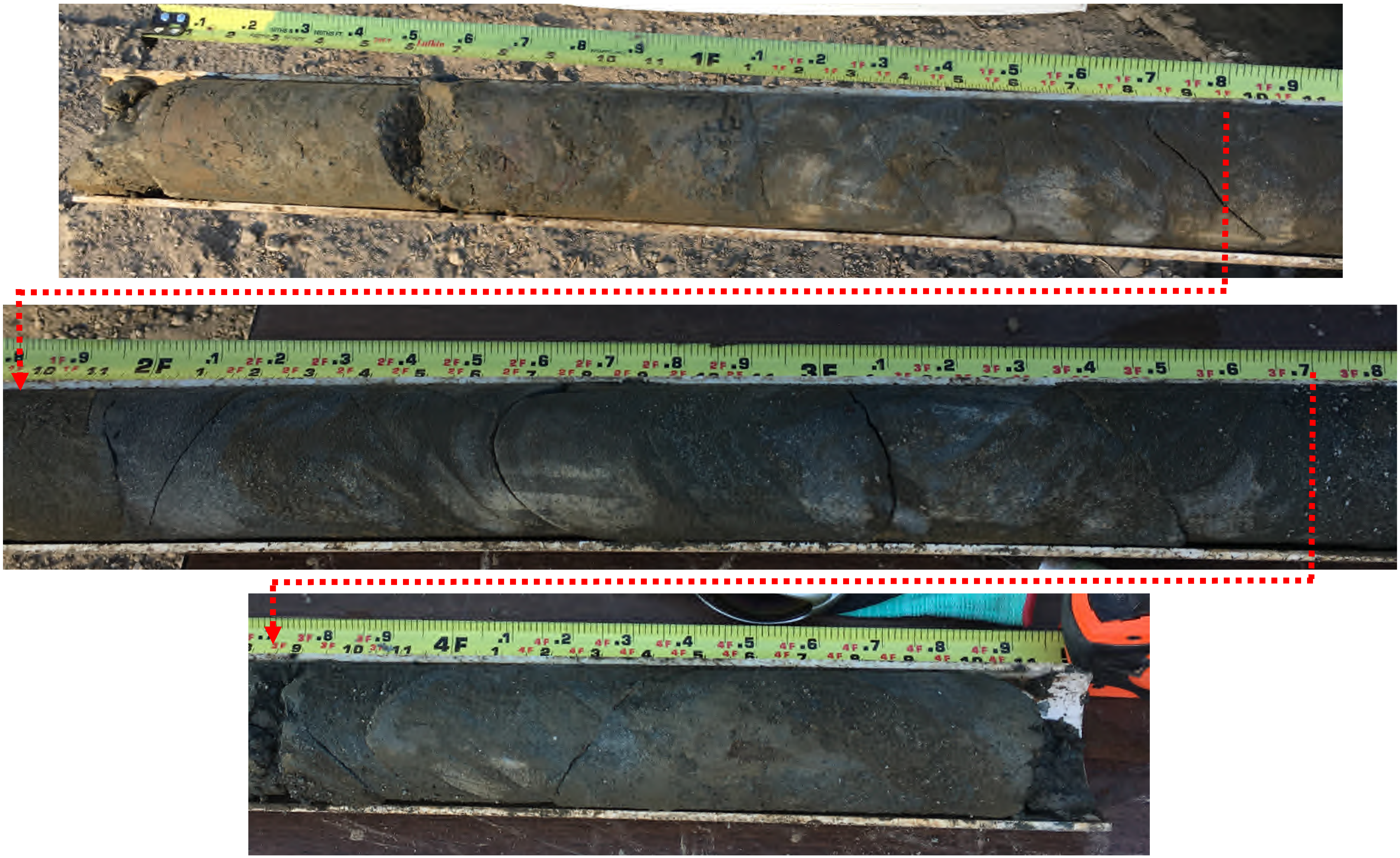
SAN PABLO, CALIFORNIA

FIGURE

B-5



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BORING B-6 ROCK CORE

RUN 5 — 41.0—46.0 FT

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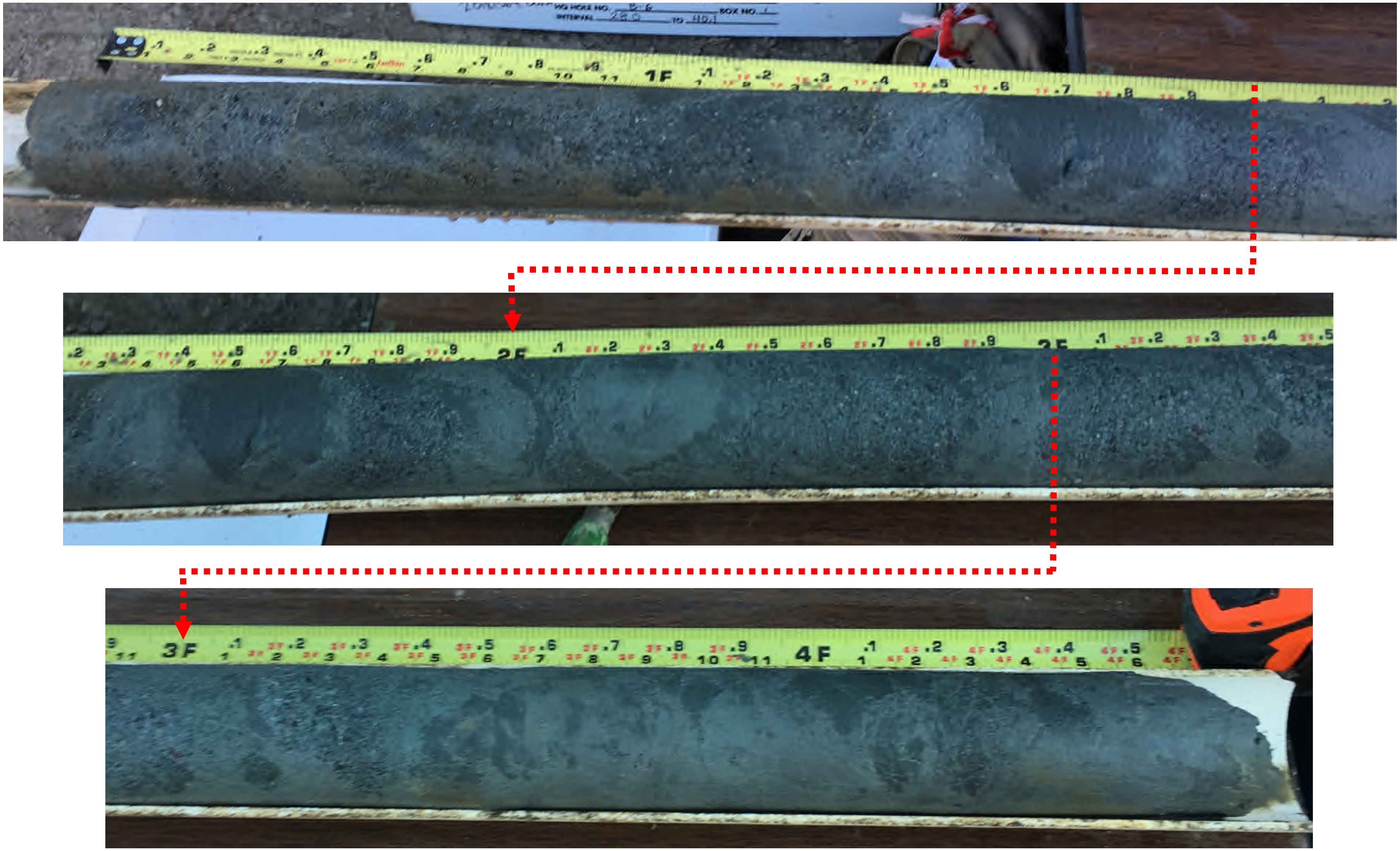
CONTRA COSTA COLLEGE
SAN PABLO, CALIFORNIA

FIGURE

B-6



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BORING B-6 ROCK CORE

RUN 6 — 46.0—51.0 FT

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CONTRA COSTA COLLEGE
SAN PABLO, CALIFORNIA

FIGURE

B-7

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PROJECT NO. 20181569	DRAWN: 12-03-18	**BORING B-6 ROCK CORE**	FIGURE
DRAWN BY: EMB	CHECKED BY: RLM	**RUN 7 — 51.0—56.0 FT**	**B-8**
FILE NAME: ROCK CORE PHOTOS		C-4016 NEW SCIENCE BUILDING CONTRA COSTA COLLEGE SAN PABLO, CALIFORNIA	

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BORING B-6 ROCK CORE

RUN 8 — 56.0—58.0 FT

C-4016 NEW SCIENCE BUILDING
CONTRA COSTA COLLEGE
SAN PABLO, CALIFORNIA

FIGURE

B-9

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BORING B-6 ROCK CORE

RUN 9 — 58.0—61.0 FT

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FIGURE

B-10

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BORING B-6 ROCK CORE

RUN 10 — 61.0—66.0 FT

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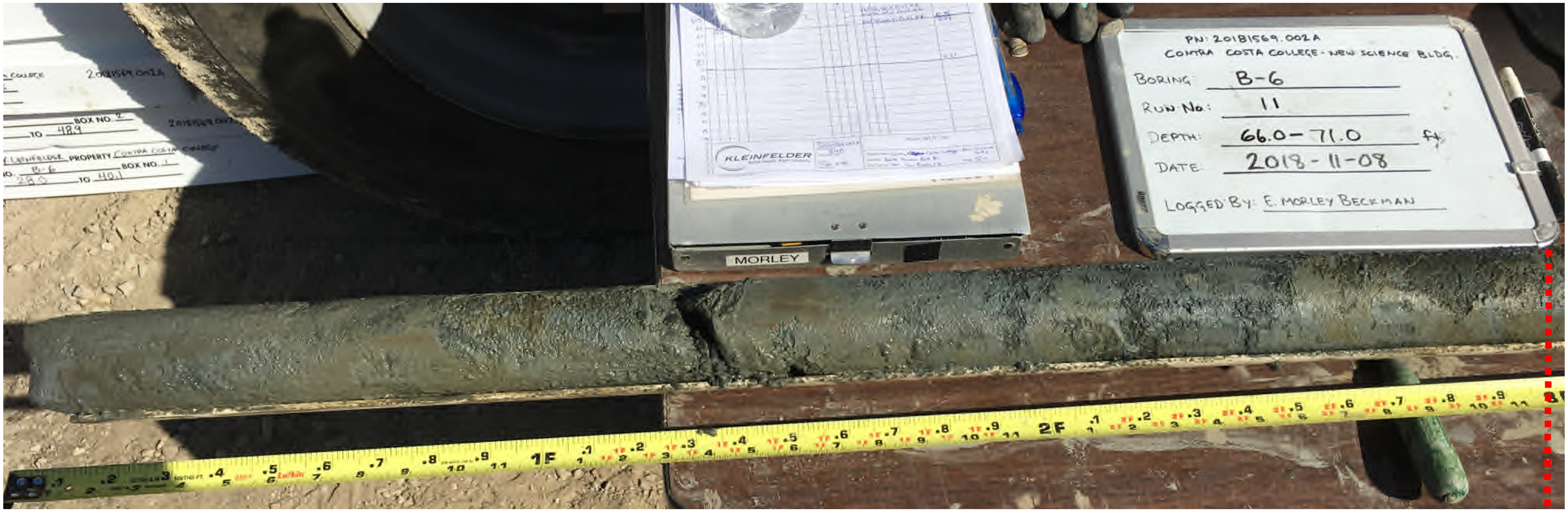
FIGURE

B-11



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BORING B-6 ROCK CORE

RUN 11 — 66.0—71.0 FT

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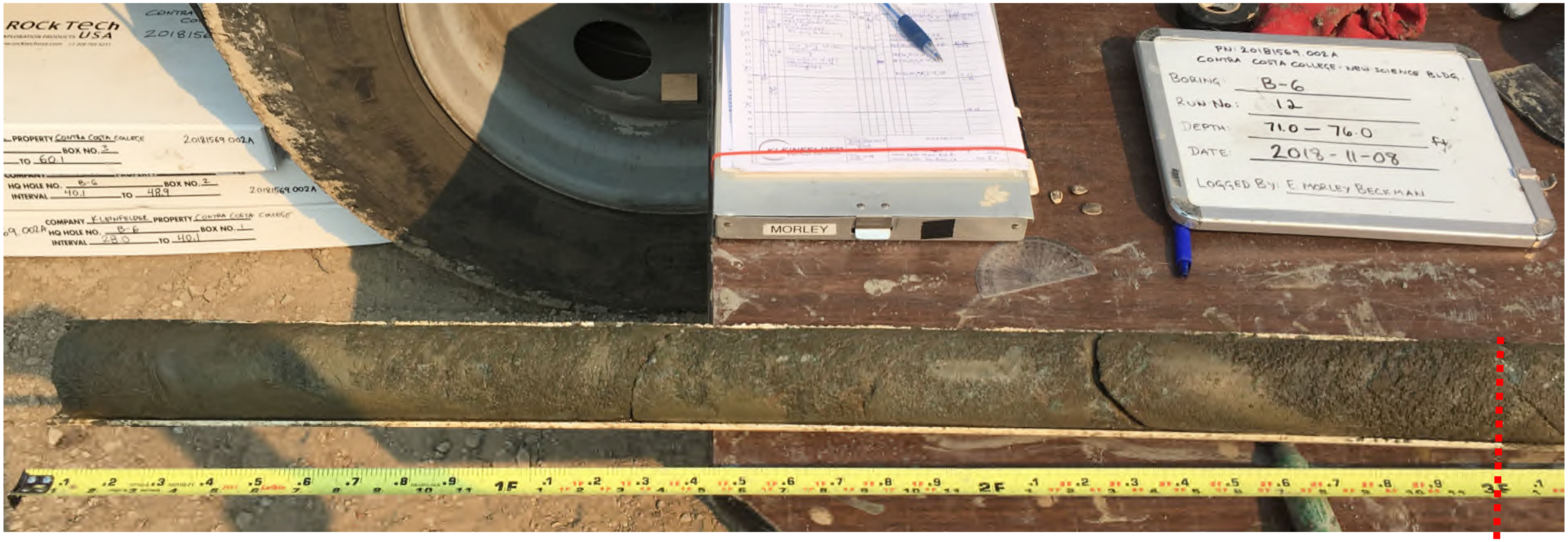
SAN PABLO, CALIFORNIA

FIGURE

B-12



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BORING B-6 ROCK CORE

RUN 12 — 71.0—76.0 FT

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SAN PABLO, CALIFORNIA

FIGURE

B-13

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BORING B-6 ROCK CORE

RUN 13 — 76.0—81.0 FT

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FIGURE

B-14

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BORING B-6 ROCK CORE

RUN 14 — 81.0—86.0 FT

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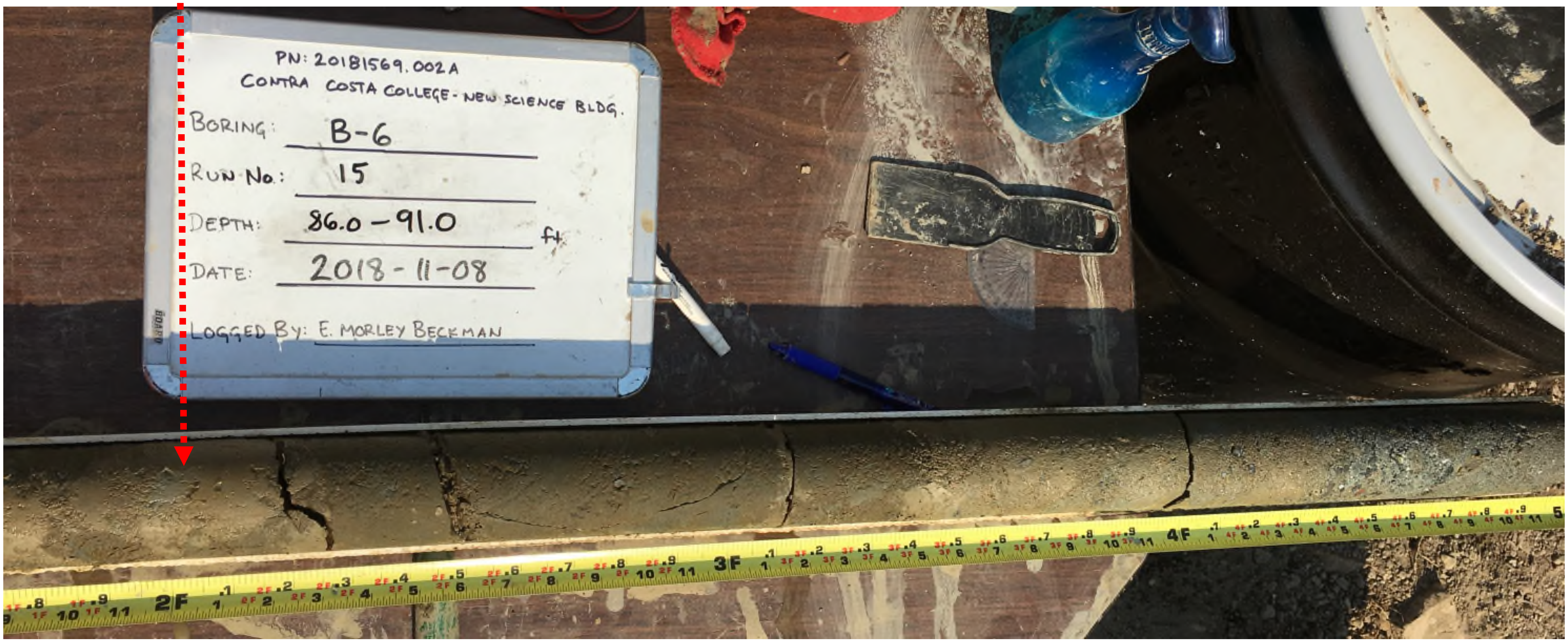
FIGURE

B-15



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FILE NAME:	ROCK CORE PHOTOS

BORING B-6 ROCK CORE

RUN 15 — 86.0—91.0 FT

C-4016 NEW SCIENCE BUILDING
CONTRA COSTA COLLEGE
SAN PABLO, CALIFORNIA

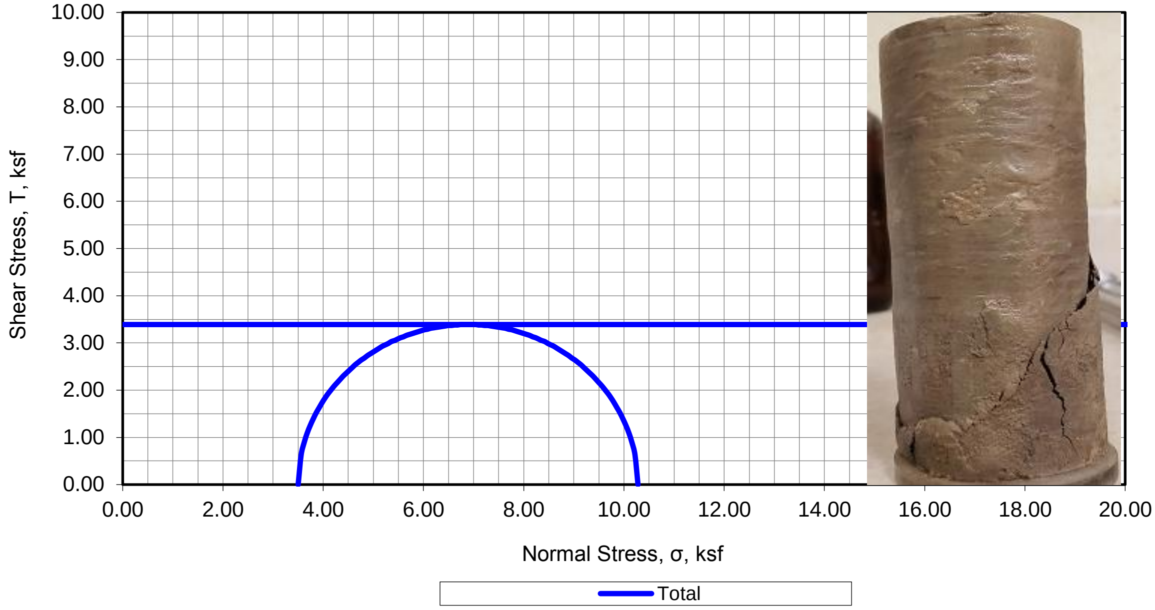
FIGURE

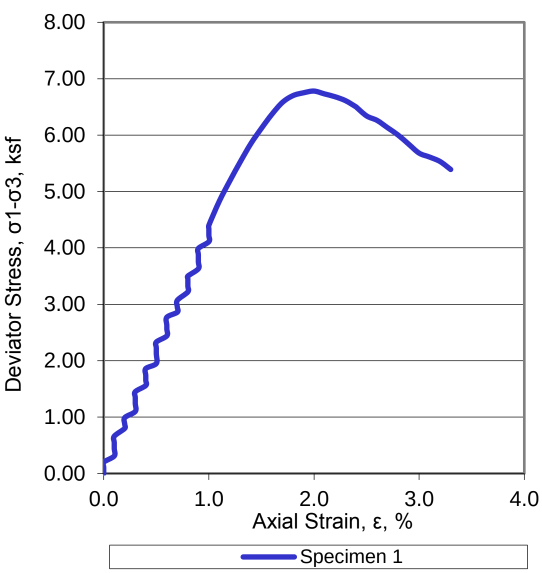
B-16

APPENDIX C
LABORATORY TEST RESULTS FOR THIS ADDENDUM

Total	
c = 3.39	ksf

Specimen Shear Picture





Specimen No.			1
Initial	Diameter, in	D_O	2.29
	Height, in	H_O	5.00
	Water Content, %	ω_O	10.9
	Dry Density, lbs/ft^3	γ_{d_o}	125.7
	Saturation, %	S_O	91
	Void Ratio	e_O	0.315
Minor Principal Stress, ksf		σ_3	3.50
Maximum Deviator Stress, ksf		$(\sigma1-\sigma3)_{max}$	6.78
Time to $(\sigma_1\text{-}\sigma_3)_{max}$, min		t_f	2.03
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	5.39
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		$'\varepsilon$	1.00
Axial Strain at Failure, %		ε_f	2.03

Description of Specimen: Brown Lean Clay with Sand (CL)

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

Boring:	B-6
Sample:	--
Depth, ft:	29.5
Test Date:	11/14/18

Remarks: nm= not measured, na = not applicable

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Project No.:	20181569.002A
Date:	11/15/18
Entry By:	CP
Checked By:	CP
File Name:	HL11806

TRIAXIAL COMPRESSION TEST (UU)

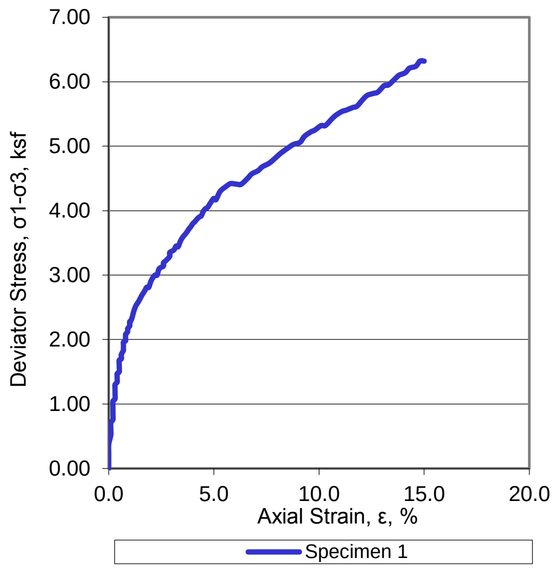
CONTRA COSTA COMMUNITY COLLEGE
NEW SCIENCE BUILDING
2600 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA

FIGURE

C-1

Total	
c = 3.16	ksf

Specimen Shear Picture



Specimen No.			1
Initial	Diameter, in	D_O	2.37
	Height, in	H_O	5.08
	Water Content, %	ω_O	10.6
	Dry Density, lbs/ft^3	γd_o	127.9
	Saturation, %	S_O	96
	Void Ratio	e_O	0.293
Minor Principal Stress, ksf		σ_3	4.51
Maximum Deviator Stress, ksf		$(\sigma 1-\sigma 3)_{max}$	6.32
Time to $(\sigma_1-\sigma_3)_{max}$, min		t_f	14.83
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	6.32
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		'ε	1.00
Axial Strain at Failure, %		ε_f	14.83

Description of Specimen: Olive Brown Sandy Lean Clay (CL)

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

Boring:	B-6
Sample:	--
Depth, ft:	37.5
Test Date:	11/14/18

Remarks: nm= not measured, na = not applicable

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Project No.:	20181569.002A
Date:	11/15/18
Entry By:	CP
Checked By:	CP
File Name:	HL11806

TRIAXIAL COMPRESSION TEST (UU)

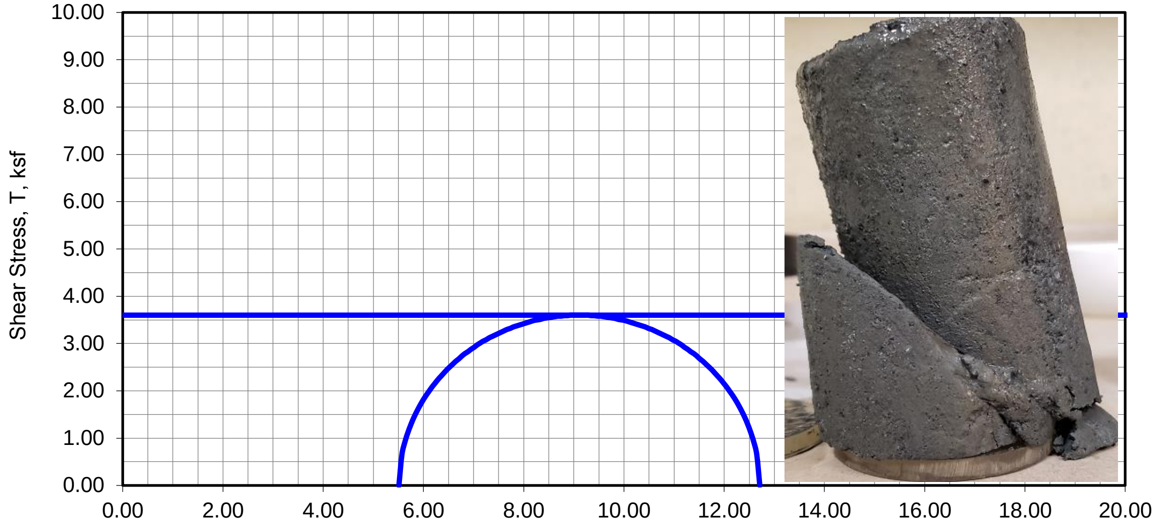
CONTRA COSTA COMMUNITY COLLEGE
NEW SCIENCE BUILDING
2600 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA

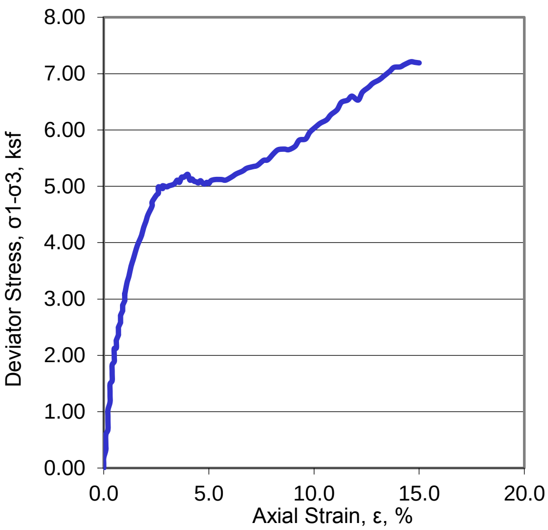
FIGURE

C-2

Total	
c = 3.60	ksf

Specimen Shear Picture





Specimen No.			1
Initial	Diameter, in	D_O	2.41
	Height, in	H_O	5.04
	Water Content, %	ω_O	9.6
	Dry Density, lbs/ft^3	γd_o	131.4
	Saturation, %	S_O	99
	Void Ratio	e_O	0.259
Minor Principal Stress, ksf		σ_3	5.50
Maximum Deviator Stress, ksf		$(\sigma1-\sigma3)_{max}$	7.21
Time to $(\sigma_1-\sigma_3)_{max}$, min		t_f	14.58
Deviator Stress @ 15% Axial Strain, ksf		$(\sigma_1-\sigma_3)_{15\%}$	7.19
Ultimate Deviator Stress, ksf		$(\sigma_1-\sigma_3)_{ult}$	na
Rate of strain, %/min		'ε	1.00
Axial Strain at Failure, %		ε_f	14.58

Description of Specimen: Dark Greenish Gray Clayey Sand (SC)

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

Boring:	B-6
Sample:	--
Depth, ft:	48.5
Test Date:	11/14/18

Remarks: nm= not measured, na = not applicable

* Membrane tore marginally during test.

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TRIAXIAL COMPRESSION TEST (UU)

CONTRA COSTA COMMUNITY COLLEGE
NEW SCIENCE BUILDING
2600 MISSION BELL DRIVE
SAN PABLO, CALIFORNIA

FIGURE

C-3