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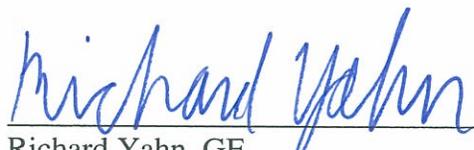
Lawrence Berkeley National Laboratory  
One Cyclotron Road  
Berkeley, California 94720

Attention: Mr. Kirk Haley  
Building 90H

**PRELIMINARY  
GEOTECHNICAL INVESTIGATION REPORT  
COMPUTATION RESEARCH AND  
THEORY BUILDING  
LAWRENCE BERKELEY NATIONAL  
LABORATORY  
BERKELEY, CALIFORNIA**

**Kleinfelder Job No: 74911**

Reviewed By:

  
\_\_\_\_\_  
Mark H. Stanley, G.E. 2397  
Geotechnical Engineer  
\_\_\_\_\_  
Richard Yahn, GE  
Geotechnical Engineer

**KLEINFELDER WEST, INC.**  
2240 Northpoint Parkway  
Santa Rosa, California 95407  
(707) 571-1883

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Seismic Refraction Survey, Advanced Geologic Services, September 19, 2006

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Technical Memorandum (Cut Slope Preliminary Design Summary), Kleinfelder – Bellevue, WA, September 21, 2006

**APPENDIX E**

Select Boring Logs – Previous Investigations by Others

**APPENDIX F**

Corrosion Test Report, Environmental Technical Services, September 29, 2006

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**1.0 INTRODUCTION**

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This report presents the results of our preliminary geotechnical investigation for the planned Computation Research and Theory Building (CRT) to be constructed within the Lawrence Berkeley National Laboratory (LBNL) facility in Berkeley, California (Site Location, Plate 1). The proposed structure will be constructed on the slope between Building 70A and the Blackberry Gate entrance kiosk located on Cyclotron Road (referenced herein as “site”). Work performed during this investigation was conducted in accordance with the tasks described in our proposal dated May 19, 2006. Kleinfelder also conducted a fault investigation for the project documented in a report titled “Fault Investigations, Computational Research and Theory Building, Lawrence Berkeley National Laboratory, Berkeley, California,” dated September 27, 2006.

**1.1 PROJECT DESCRIPTION**

The project consists of constructing a 50,000 square foot (footprint) computer facility building that may include up to 8 stories with associated parking and utilities. LBNL provided an architectural rendering (sketch) showing the proposed structure footprint on a conceptual site plan entitled “B70 West Site” (undated) and in subsequent untitled and undated sketches. The approximate footprint of the structure and associated improvements are shown on the Site Plan, Plate 2. At the time of this report, the actual limits and type of building construction have not been confirmed.

The building will be situated on the west-facing slope between building 70A and Cyclotron Road near the Blackberry Gate entrance kiosk. The sketches indicate that the building will be stepped up the hillside with the lowest level at approximately elevation 637 feet (LBNL Datum) and the upper entrance deck at elevation 760 feet. A parking area, approximately 1/3 acre in size, is planned north of the building site and will be entered from the drive to Buildings 50E and 50F off Cyclotron Road. A west-facing stairway to Cyclotron Road is planned between the building and parking area.

## 1.2 PURPOSE AND SCOPE OF SERVICES

The purpose of this investigation is to provide preliminary geotechnical recommendations for the design of the new building and retaining structures. The scope of this investigation included researching published data, evaluating site geologic conditions by drilling test borings, and analyzing field and laboratory data to address the following:

- Seismic design criteria in accordance with the 2006 International Building Code standards.
- Potential geologic hazards.
- Site preparation and excavation.
- Foundation type and design criteria for new structures.
- Slab-on-grade recommendations.
- Geotechnical drainage.

Two additional deep borings/rock cores were proposed within the slope near the lower edge of the building footprint. Drilling of these borings was postponed by LBNL. These and possibly additional borings should be conducted pending finalization of the actual building footprint and more specific determination of anticipated building loads. Once this information is better defined, the borings can be strategically located within the building envelope and a design level geotechnical report can be prepared.

### 1.3 AUTHORIZATION

This investigation was authorized by Ms. Laura Crosby in the LBNL subcontract agreement numbered 6805923, Modification Number 1.

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## 2.0 FIELD INVESTIGATION

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### 2.1 SUBSURFACE EXPLORATION

Our subsurface investigation comprised three phases; drilling geotechnical borings, excavating test pits, and using a seismic refraction survey. Our test pits were excavated on August 29, 2006, the borings were drilled on September 18, 2006, and a seismic refraction survey was conducted on August 31, 2006. Our subsurface investigation locations are shown on the Site Plan (Plate 2). The purpose of the investigation was to evaluate subsurface conditions for construction of the planned building, parking areas, and retaining walls; therefore, our explorations were advanced through surficial soils and into the underlying bedrock, where possible.

The test pits were excavated using a track-mounted excavator (Takeuchi - TB 145) equipped with a 30-inch-wide bucket. The test pits were excavated to depths ranging from 6 to 10½ feet below existing ground surface and were shored for safety. Our project geologist observed the excavation and test pits, and recorded the soil and bedrock conditions encountered.

Borings K-1, K-2, and K-3 were drilled with a truck-mounted Mobile B-53 power-auger drill utilizing 6-inch-diameter solid flight augers to depths ranging from 5½ feet to 30 feet. Our project geologist observed the drilling, logged the conditions encountered, and obtained samples for visual classification and laboratory testing. Samples of the soil were obtained using a 2.43-inch (inside diameter) sampler and a 1.42-inch (inside diameter) sampler driven with a 140-pound hammer dropped 30 inches. The blows required to drive the sampler were recorded and converted to equivalent Standard Penetration blow counts for correlation with empirical data.

The soils encountered are described in accordance with the Unified Soil Classification System (USCS) as presented on the Boring Log Legend, Plate A-1 in Appendix A. Bedrock has been classified in accordance with the Rock Description Criteria in Appendix A. Logs of the borings are presented on Plates A-3 through A-5 and the logs of the test pits are presented on Plate A-6 in Appendix A.

The seismic refraction survey was conducted by Advanced Geological Services (AGS). Their report is included as Appendix C. The location of the seismic survey lines are shown on Plate 2.

## 2.2 LABORATORY TESTING

Selected samples were tested to evaluate pertinent engineering and physical properties of the soils/rock encountered. The laboratory testing program evaluated the plasticity, strength, and particle size of the soil, and the soil derived from processing the fractured rock. Classifications made in the field were modified, as appropriate, based on the laboratory test results; classifications presented on the boring logs reflect modifications made as a result of laboratory tests. The results of the laboratory testing are presented in Appendix B and on the boring logs.

Routine corrosion testing was performed by Environmental Technical Services (ETS) of Petaluma, California on a sample of expected foundation bearing material collected from boring K-3, and their report is included in Appendix F. It should be noted that Kleinfelder does not practice corrosion engineering. As such, the corrosivity test results are presented for the preliminary use of our Client and their project team. Our review of comments contained in the corrosion report indicates that the soil tested has fair resistivity, low chloride and sulfate concentrations, is mildly acidic, and has mild redox so standard concrete mixes appear to be acceptable in soil. Type II cement is recommended by the testing laboratory due to the soil Redox levels. A corrosion specialist should be contacted to review these results and determine if additional testing is warranted.

## 2.3 REPORTS BY OTHERS

As part of our geotechnical investigation, we reviewed the geotechnical files retained at the LBNL campus for the CRT site and adjacent building sites. Information provided in these reports was utilized in our evaluation of surface and subsurface conditions. The reports reviewed include:

1. Dames & Moore, "Foundation Investigation, Proposed Building 70 Addition," dated April 2, 1959.

2. Harding Lawson Associates, "Geology of Lawrence Berkeley Laboratory," dated September 28, 1982.
3. Harding Lawson Associates, "Geotechnical Investigation, Building 50F, Office Addition," dated March 1, 1983.
4. Geo/ Resource Consultants, Inc., "Proposed Air Handling Units (AHU), South of Building 70A," dated May 22, 1990.
5. Subsurface Consultants, Inc., "Geotechnical Investigation, Acid Neutralization Tank Enclosure, Building 70A," dated March 30, 1992, LBNL File Number 286.
6. Subsurface Consultants, Inc., "Geotechnical Investigation, Metal Tank Enclosure & Foundation Pad, Building 70A, 76 and 77," dated July 31, 1992.
7. Fugro West, Inc., "Geotechnical Investigation, Proposed Building 50X," dated August 5, 2002.
8. Fugro West, Inc., "Fault Rupture Hazard Investigation, Proposed Building 50X," dated August 21, 2002.
9. Fugro West, Inc., "Preliminary Geotechnical Evaluation, Building 70A Parking Lot," dated September 30, 2002.
10. Kleinfelder, Inc., "Geotechnical Investigation Report, Blackberry Gate Improvements," dated May 1, 2006.
11. Kleinfelder, Inc., "Fault Investigation, Computation Research and Theory Building," dated September 27, 2006.

In addition to geotechnical investigation reports, we reviewed the "as-built" drawings for Building 50F, Building 70 and Building 70A.

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### 3.0 SITE CONDITIONS

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#### 3.1 REGIONAL GEOLOGY

The site is located within the Coast Ranges Geomorphic Province of Northern California. The Coast Ranges Province is a geologically complex and seismically active region characterized by sub-parallel northwest-trending faults, mountain ranges, and valleys which are a reflection of the dominant northwest structural trend of the bedrock in this region. The oldest mapped bedrock unit within the Coast Ranges Province is the Franciscan Complex, a diverse group of igneous, sedimentary, and metamorphic rocks of Upper Jurassic to Cretaceous age (140 to 65 million years old). Since deposition, the bedrock materials have been subjected to faulting and folding. These rocks are part of a northwest-trending belt of material that lies along the east side of the San Andreas fault system. Locally, these older bedrock deposits are overlain by younger, Quaternary age (less than 2 million years old) marsh, alluvial, and colluvial deposits.

#### 3.2 LOCAL GEOLOGY

The site and vicinity have been mapped by Graymer (2000, United States Geologic Survey – Miscellaneous Field Studies MF 2342) and Korbay (1982, Harding Lawson Associates, Geology of the Lawrence Berkeley Laboratory – HLA Job No. 2000, 135.01). The geologic maps prepared by these authors generally agree that the site is underlain by late Cretaceous sedimentary rocks. Graymer described the geology of the site area as “unnamed sedimentary rocks of the Great Valley Complex” that are characterized as “massive to distinctly bedded, biotite-bearing, brown-weathering, coarse- to fine-grained greywacke and lithic wacke, siltstone, and mudstone.”

The geologic map prepared by Korbay (1982) indicates the presence of a “Minor inactive fault” crossing the east corner of Building 70A. This fault is also mapped approximately 30 feet from the southwest corner of Building 70. Conversations with Mr. Steve Korbay revealed information regarding the nature of the fault zone. Mr. Korbay described the fault as a highly sheared zone

of gray shale that was exposed in the excavation for Building 50F. We have indicated zones of sheared shale on the cross sectional profiles (Plates 3, 4, 5 and 6). We have not identified a common trend or trace allowing us to connect the sheared shale to the fault mapped by Korbay. It is reasonable to assume that shear zones are present with many orientations and may not be laterally continuous. Depending on the final configuration of the CRT Building, the easternmost excavation for the building may encounter this fault zone.

### 3.3 FAULTING AND SEISMICITY

Evidence of active faulting was not observed in the trenches KT-1 or KT-2 (Plate 2). Results of our fault investigation are addressed in our September 27, 2006, Fault Investigation Report. The site, as well as the entire Northern California Coastal Region, is located within a seismically active portion of the state, dominated by the presence of the San Andreas fault system, which forms the boundary between two tectonic plates of the earth's crust. At this boundary, the Pacific Plate (west of the fault) is moving north relative to the North American Plate (east of the fault). In the San Francisco Bay Area, this movement is distributed across a complex system of strike-slip, right-lateral, parallel, and sub-parallel faults which include the San Andreas, West Napa, Healdsburg/Rodgers Creek, Maacama, Concord-Green Valley, and Hayward among others.

The site is located within an Earthquake Fault Zone as defined by the California Geologic Survey (formerly California Division of Mines and Geology) in accordance with the Alquist-Priolo Earthquake Fault Zone Act of 1972. The Richmond, Oakland East, and Oakland West quadrangles (California Geological Survey (CGS), 1982, Earthquake Fault Zones, Richmond Oakland East and Oakland West Quadrangle) indicate the Hayward fault is located less than 1 kilometer to the west of the planned CRT site. Moderate to major earthquakes generated on the Hayward fault can be expected to cause strong ground shaking at the site. In addition, strong ground shaking can be expected from moderate to major earthquakes generated on other faults in the region such as the Concord-Green Valley fault (located 20 kilometers east of the site), the Calaveras fault (located 19.5 kilometers east of the site), the San Andreas fault (located 30 kilometers west of the site), and the Healdsburg-Rodgers Creek fault (located 26 kilometers

north of the site). A number of large earthquakes have occurred within this region in the historic past. Some of the significant nearby events are the 1989 Loma Prieta earthquake (M6.9), the 1906 San Francisco earthquake (M8+), the 1868 Hayward fault earthquake (M7), the 1838 San Francisco earthquake (M7+) and the 1836 Oakland earthquake (M6+). Future seismic events in this region can be expected to produce strong seismic ground shaking at this site. The intensity of future shaking will depend on the distance from the site to the earthquake focus, the magnitude of the earthquake, and the response of the underlying soil and bedrock.

### 3.4 SURFACE CONDITIONS

The site is bounded by Cyclotron Road on the west and Buildings 70 and 70A on the east. Building 50X was proposed to the north but has not been constructed. A significant drainage crosses the slope approximately 150 feet to the south of the site. Minor grading appears to have been performed in the area; however, there is no record of previous building structures on the site. A wooden staircase crosses the site and previous staircase alignments have utilized the graded flat areas as picnic areas. A service road supported by short retaining walls has been constructed below the elevated parking area west of 70A. The project area consists of west-facing slopes inclined at approximately 2H:1V (horizontal: vertical) to 4H:1V. Cutslopes as steep as 1.25H:1V are located along the eastside of Cyclotron Road below and west of the site. The cutslopes were performed during recent grading in late July 2006 as part of the Blackberry Gate Improvements project that included new cutslopes for additional pavement areas and a new gate kiosk. Evidence of slope instability within the cut slope or adjacent slopes was not observed on the surface.

### 3.5 SUBSURFACE CONDITIONS

Five test pits were excavated on the slope of the planned building area (see Plate 2). The test pits encountered thin topsoil over colluvial deposits. In TP-1 through TP-4 underlying the colluvial deposits, completely weathered bedrock over less weathered bedrock was encountered. TP-5 did not encounter bedrock by a depth of 10 feet, which was the depth limit based on the position of the excavator on the slope. The topsoil was described as sandy silt with gravel that, at the time of our investigation was dry, very stiff to hard, and slightly porous. The colluvium was gravelly,

sandy clay that was moist and hard. The bedrock was described as a siltstone that is completely to moderately weathered, extremely weak to weak with very close fractures. The materials observed in these test pits is very similar to the subsurface conditions encountered during our fault trench investigation for the same project. However, moisture contents were dramatically different. In fact, our fault trenches readily collected water while the soil in our test pits appeared mostly dry.

In test pits TP-3 and TP-4, and in our fault trenches, we encountered a thin gray clay seam at the base of the completely weathered bedrock.

Borings K-1 and K-2 were drilled in the service road area below the elevated parking west of Building 70A (see Plate 2). Boring K-1 encountered gravelly sandy clay (colluvium) that was moist and very stiff during our drilling below the pavement section. The gravelly sandy clay was observed in the boring to a depth of approximately 7 feet, where we encountered siltstone. The siltstone was described as completely weathered, very closely fractured, and very weak. At approximately 18 feet, we encountered shale that was highly weathered, very closely fractured, weak, and thinly bedded. Our boring encountered sandstone at approximately 24½ feet. The sandstone was highly weathered, very closely fractured, and weak. Drilling became very difficult at approximately 26 feet and the boring was terminated at approximately 27 feet.

Boring K-2 encountered clayey, sandy gravel below the pavement section (fill) to a depth of approximately 5½ feet. At approximately 5½ feet our boring encountered concrete, at which time we contacted Mr. Jim Mankini of LBNL regarding utilities. After review of the utility plans, it was suspected that the concrete is the top of a duct bank for communication lines and the boring was abandoned.

K-3 was drilled in the parking area west of Building 70 as seen on Plate 2. Boring K-3 encountered interbedded siltstone and sandstone below the pavement section to a depth of approximately 27 feet. The siltstone and sandstone were described as completely to moderately weathered, very closely fractured, and very weak. At approximately 27 feet, our boring encountered shale that was very closely fractured and very weak.

A dormant landslide was encountered in trenches KT-1 and KT-2; these trenches were excavated as part of our 2006 fault investigation (located in the vicinity of the southwest corner of the proposed building footprint [Plate 2]). The landslide is located approximately 10 feet (vertically) below the ground surface and was a source of seepage.

### 3.6 GROUNDWATER

Groundwater was encountered in boring K-3 at a depth of approximately 27 feet at the time of drilling. The groundwater level was not allowed to stabilize and groundwater levels should be expected to rise significantly. Groundwater was encountered at similar depths in borings drilled for Building 50A (HLA 1965) and Building 70 (Dames and Moore 1959). The fault zone mapped by Korbay (1982) discussed above, is a possible aquatard damming groundwater at various depths east of the fault. It should be understood that seepage and groundwater levels can vary seasonally and could rise and fall several feet annually; however, we do not expect that groundwater will enter typical shallow excavations.

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## 4.0 DISCUSSION AND CONCLUSIONS

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### 4.1 GENERAL

Based on the results of our field exploration, laboratory testing, review of reports by others, and engineering analyses; we conclude that, from a geotechnical engineering standpoint, the construction of the building and related improvements are feasible. The most significant geotechnical engineering factors that must be considered in design and construction of the project are foundation support, excavation support, lateral earth pressures for retaining walls, the presence of undocumented fill, and the potential for strong seismic shaking generated from earthquakes on active faults in the region.

### 4.2 SITE SEISMIC CHARACTERIZATION

The site is not located within an Earthquake Fault Zone as defined by the California Division of Mines and Geology. However, because of the proximity to the Hayward, Concord-Green Valley, Calaveras, and San Andreas faults, the site is expected to be subjected to very strong ground shaking during a moderate to major earthquake on these or other active faults in the area. On the basis of current technology as well as historical evidence, it is reasonable to assume that, during the life of the proposed development, it will be subjected to at least one moderate to severe earthquake that could produce potentially damaging ground shaking at the site. Further, it is anticipated that the subject site will periodically experience small to moderate magnitude earthquakes. Therefore, the proposed structure should be designed to withstand the effects of the anticipated strong ground shaking.

Field and laboratory test data indicate that the site and proximity can be assigned a Site Class C based on average soil properties in the top 100 feet and Table 1613.5.2 of the 2006 International Building Code (IBC). Site Class C is defined as a profile consisting of very dense soil and soft rock with a shear wave velocity between 1,200 and 2,500 feet per second, a Standard Penetration Test, N (blows/foot), of greater than 50 and un-drained shear strength of greater than 2,000

pounds per square foot. The seismic design criteria are based upon the guidelines in ASCE 7 which is the basis of the International Building Code (IBC 2006). ASCE 7 is based upon a maximum considered earthquake ground motion, defined as the motion caused by an event with a 2% probability of exceedance within a 50 year period (recurrence interval of approximately 2,500 years). We have used the “Ground Motion Parameter Calculator” provided on the United States Geologic Survey (USGS) web site (<http://earthquake.usgs.gov/research/hazmaps/design/>) to calculate the site specific parameters based upon the site coordinates (longitude and latitude). Site coordinates used for this site are N37.87593, W122.25262. The resulting seismic design factors are summarized below.

**SUMMARY OF SEISMIC DESIGN FACTORS - ASCE 7**

Site Class	F <sub>a</sub>	F <sub>v</sub>	S <sub>s</sub>	S <sub>1</sub>	S <sub>MS</sub>	S <sub>M1</sub>	S <sub>DS</sub>	S <sub>D1</sub>
C	1.0	1.3	2.023	0.791	2.023	1.028	1.349	0.685

The factors are defined as follows:

F<sub>a</sub> – Short Period Coefficient to modify 0.2 second period of mapped spectral response accelerations for Site Class other than Site Class B.

F<sub>v</sub> – Long Period Coefficient modify 1.0 second period of mapped spectral response accelerations for Site Class other than Site Class B.

S<sub>s</sub> – Mapped maximum considered earthquake spectral response acceleration, 5 percent damped, at 0.2-second period for Site Class B (in %g).

S<sub>1</sub> – Mapped maximum considered earthquake spectral response acceleration, 5 percent damped, at 0.2-second period for Site Class B (in %g).

S<sub>MS</sub> – Maximum considered earthquake spectral response acceleration, 5 percent damped, at 0.2-second for Site Class effects (in %g).

S<sub>M1</sub> – Mapped maximum considered earthquake spectral response acceleration, 5 percent damped, at 1.0-second period for Site Class effects (in %g).

S<sub>DS</sub> – Design earthquake spectral response acceleration, 5 percent damped, at 0.2-second period (in %g).

S<sub>D1</sub> – Design earthquake spectral response acceleration, 5 percent damped, at 1.0-second period (in %g).

We understand that the building will have an Occupancy Category of III or IV per Table 1604.5 of the IBC. Since  $S_1$  is greater than 0.75, Section 1613.5.6 of the IBC requires that the structure be assigned a Seismic Design Category E for Occupancy Category III and Seismic Design Category F for Occupancy Category IV.

### 4.3 GEOLOGIC HAZARDS

The site will be founded on a hillside comprised of fractured bedrock. Therefore, the risk for the occurrence of liquefaction, seismically induced settlement, or lurching is negligible. A dormant landslide is located in the northwest corner of the building footprint. The stability of the dormant landslide is unknown. The dormant landslide is relatively shallow and should be completely removed if the finalized building footprint is located in this area. New foundations should also bear in undisturbed bedrock located below the dormant landslide.

The site is underlain by shale and siltstone that could have moderate expansion potential when used as fill. Re-use of excavated materials as structural fill should be tested and approved in writing by the Geotechnical Engineer.

### 4.4 DESIGN CONSIDERATIONS

#### 4.4.1 Undocumented Fill and Colluvium

Our borings indicate that upper portions of the site contain unknown thicknesses of fill. We have not been able to locate any documentation regarding the placement and compaction of this fill. Although the fill that we encountered in the borings appeared to be well consolidated, undocumented fills can contain deleterious conditions including, but not limited to, wood, topsoil, debris, voids, and other materials that could result in future settlements or foundation instability. Foundations supported on undocumented fill will be subject to risk of undetected conditions. To avoid such risks, it would be necessary to either remove the fill and reinstall it in a controlled manner or use deep foundations that derive their support below the fill soils. For our geotechnical report, we have assumed that the fill will be removed and replaced as engineered

fill beneath the building pad. For the purposes of this report, the building pad is defined as extending 5 feet beyond the structure footprint.

The site is blanketed by a layer of colluvial soils that is not suitable for support of structures. The colluvial soils should be excavated for their entire depth and, where required, replaced with engineered fill properly keyed and benched into the underlying bedrock. Recommendations for construction of engineered fills are provided in Section 5.0 of this report.

#### 4.4.2 Foundation Support

Details regarding the specific vertical configuration of the proposed structure are not currently available. However, we anticipate that the structure will be stepped up the hillside. Plates 3 through 6 present cross sections showing the interpreted configuration of the proposed excavations based upon the conceptual project renderings.

The selection of the appropriate foundation system will depend on the vertical configuration of the structure. To limit differential settlements, care must be taken so that the structure is supported on footings founded in competent bedrock. Temporary and permanent retaining structures will be required to support the proposed vertical cuts of up to 40 feet; recommendations are presented in the following section. The structure can be supported on spread footing foundations or drilled shafts (piers) bearing in undisturbed bedrock. Review of as-built drawings of adjacent structures indicates that structures are generally supported on isolated column and continuous perimeter foundations, except adjacent to existing slopes where the foundations are supported by drilled pier foundations. Building 50F is supported on drilled pier foundations to accommodate the lower parking level. The underlying bedrock is a highly fractured, highly weathered shale, siltstone, and sandstone. As a whole, the rock mass will provide adequate support for the structure. However, shear zones are present throughout the rock mass and an inactive fault (shear zone) has been mapped as crossing the site (Korbay, 1982). We understand that the fault was observed in the excavation of Building 50F (Korbay, personal communication). Although the fault was observed to be less than 3-feet-thick, it was also observed to be highly sheared and the source of a significant amount of seepage. We further judge that the CRT building can be supported on a combination drilled pier and spread footing

foundation. Special considerations may be required where weak shear zones are encountered. Recommendations for design of spread footing and drilled shaft foundations are provided in Section 5.0 of this report.

#### 4.4.3 Excavation Support

Open cutting of deep excavations for the entire project does not appear feasible based upon the current project configuration. Therefore, the deep excavations will need to be shored. Due to the fractured nature of the bedrock, a soldier beam and lagging wall incorporating tie-back rock anchors for lateral support will be required. Alternatively, a tied-back shotcrete shoring system that incorporates tie-back anchors may be feasible. Either system will require top down construction methodologies. The shoring system should be designed by a specialty Contractor experienced in the design, construction, and maintenance of the selected shoring system. We evaluated to rock quality and conducted stability analysis for the proposed 43-foot-tall cut slope; this analysis is described in greater detail in Appendix D. We utilized the computer programs, Rocplane V. 2.0 and Swedge V.4.0 by Rocscience® to complete the stability analyses on the potential wedge and planar-type failures. We assumed a cohesion of zero and a friction angle of 30 degrees for the strength of the rock. We assumed a slope height of 43 feet based on the cross-section of the excavation. Based upon our analysis, the model indicates a rectangular pressure distribution over the height of the upper cut slope (43 feet) of 1,200 pounds per square foot (psf). We also estimated the pressure based on a standard triangular pressure distribution. Based on our calculations, the equivalent fluid pressure is  $50H$  pounds per cubic foot (pcf), where  $H$  is the slope height. This analysis does not include any surcharge pressure from adjacent structures or proposed new foundations which would impose additional pressure on portions of the wall. We can assist in evaluating the affects of these additional pressures on cut slope stability if needed.

As currently proposed, the excavations will be located immediately adjacent to Building 70A (see Plate 3). Deep excavation could result in settlement of the adjacent ground and underpinning of existing foundations may be required. Recommendations for underpinning foundations are beyond the scope of this report but should be developed as part of a final design report.

#### 4.4.4 Groundwater

We judge that groundwater seepage will be encountered in the excavations. The Building 50 addition included an extensive drainage gallery to control groundwater. We recommend that observation wells be installed as part of additional investigations to develop a design groundwater level elevation for the project. As previously discussed, the site is crossed by a shear zone/fault. Based upon our review of available data and our experience on the LBNL site, the groundwater level on the east side of the fault may be higher than the west side. It is also unknown how the drainage gallery for the Building 50 addition may have affected the groundwater levels as represented in past geotechnical studies and groundwater levels could be lower than indicated in the historic geotechnical data.

### 4.5 CONSTRUCTION CONSIDERATIONS

#### 4.5.1 Excavatability

The site is underlain by weathered, highly fractured bedrock. Geophysical surveys conducted for this investigation and immediately north of the site indicate that the bedrock underlying the site has a shear wave velocity of between about 3,000 feet-per-second and 5,000 feet per second within the proposed depth of excavation. We judge that the excavations can be performed using conventional excavation equipment including a D8 dozer with a single ripper and large excavators having a bucket equipped with rock teeth. Locally, harder/stronger zones of bedrock could be encountered, especially at depth. The use of hoe rams should be anticipated in harder bedrock zones; the need for blasting is not anticipated.

#### 4.5.2 Temporary Excavations

All excavations or cuts greater than 5 feet should be shored or sloped for safety in accordance with Occupational Safety and Health Administration (OSHA) standards. The design and maintenance of all necessary shoring, underpinning, and temporary excavation slopes should be the responsibility of the Contractor unless the retaining structure will be part of the permanent structure.

#### 4.5.3 Temporary Slopes

The design and maintenance of temporary slopes is generally considered to be the responsibility of the Contractor. The Contractor should be familiar with applicable local, state, and federal regulations, including the current OSHA Excavation and Trench Safety Standards. In no case should slope height, inclination, and excavation depth exceed those specified in local, state, or federal safety regulations. Specifically, the Contractor should be familiar with the current Cal/OSHA regulations as described in California Code of Regulations, Title 8.

The soil conditions, as encountered in our investigation, vary across the site. The majority of the near surface soils are stiff clays/silts and clay/silt fill that may be considered to be Type “B” under OSHA regulations. The Contractor should verify soil conditions throughout the proposed limits of construction.

The Contractor’s “responsible person,” as defined in 29 CFR Part 1926, should evaluate the soil exposed in the excavations as part of the Contractor’s safety procedures. The Contractor’s responsible person should establish a minimum lateral distance from the crest of the slope for all vehicles, equipment, and spoil piles. Likewise, the Contractor’s responsible person should establish protective measures for exposed slope faces. We recommend that the Contractor, or his specialty subcontractor, design temporary construction slopes to conform to OSHA’s “Guidelines for Excavations and Temporary Sloping.” The temporary slope inclination should be determined by the Contractor or responsible subcontractor based on the soil conditions exposed at the time of construction. If an excavation, including trenches, is extended to a depth of more than twenty (20) feet, it will be necessary to have the side slopes designed by a Civil Engineer registered in the State of California.

#### 4.5.4 Drilled Piers

Drilled piers will be drilled almost entirely within the underlying fractured rock. Hard drilling conditions should be anticipated in the bedrock, especially at depths greater than 30 feet. The bottoms of pier excavations should be relatively free of loose cuttings or slough prior to placing reinforcing steel and concrete. Standing water, if present, should be pumped from the

excavations prior to placing the concrete. We recommend that the Contractor review the boring logs prior to establishing a method of drilling the piers. Care must be taken to support the pier holes in the underlying fractured bedrock. We recommend that Kleinfelder observe the excavation of the piers to check that they are founded in suitable materials and constructed in accordance with the recommendations presented in this report.

#### 4.5.5 Groundwater Control

Groundwater was encountered during our investigation in boring K-3 at a depth of 27 feet. Based upon review of other available geotechnical reports and the groundwater encountered in Boring K-3, we judge that groundwater/seepage zones will generally be encountered within the proposed deep excavations. It should be the Contractor's responsibility to monitor the groundwater elevation and control groundwater during construction.

## 5.0 RECOMMENDATIONS

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### 5.1 GENERAL

In developing the following recommendations, we understand that specific details regarding the configuration of the hillside structure have not been determined. However, we understand that the structure will be excavated into the slope and, as such, key geotechnical considerations include 1) temporary and permanent support of excavations, 2) providing underpinning or other means of support for adjacent structures, 3) groundwater control, 4) placement of fills on slopes (if required), and 5) strong ground shaking. These and other considerations are discussed in the following sections.

### 5.2 SITE PREPARATION AND GRADING

#### 5.2.1 Clearing and Site Preparation

The site should be cleared of existing structures and their foundations. Existing and abandoned utilities should also be removed or properly abandoned. Voids created by removal of existing structure foundations and utilities located below the depth of proposed grading should be backfilled with material meeting the requirements for structural (engineered) fill placed in accordance with recommendations for fill placement (Section 5.2.5).

The site should then be stripped of vegetation and organic or construction debris before grading commences. We anticipate that the stripping operation will require the removal of 2 to 3 inches of topsoil in most areas. Deeper stripping or grubbing will be required where concentrations or pockets of organic-laden soil, tree root balls, or old fills are encountered. The stripped, organic-rich material may be stockpiled and used for future landscaping purposes; however, this material should not be used as engineered fill.

### 5.2.2 Removal of Unsuitable Soils and Subgrade Preparation

Undocumented fills and unstable native soils should be removed for their entire depth and properly moisture conditioned and compacted as engineered fill. A Kleinfelder representative should be on-site to observe fill removal. We expect that grading will include the excavations for removing the undocumented fill, replacement with engineered fill, and minor site preparation for sidewalks and pavement approaches. In areas to receive fill, the soil should be scarified at least 6 inches deep. The soil should then be moisture conditioned to near optimum or slightly above optimum moisture content and compacted to at least 90 percent of the maximum dry density as described in ASTM D-1557. If areas appear to be yielding and/or saturated, deeper re-compaction may be required, as determined by Kleinfelder.

### 5.2.3 Benching Into Native Soil

On slopes inclined 6H:1V or steeper, structural fills should be benched into the native soil. Benches cut for the construction of the fills should be at least 5-feet-wide and a maximum of 5-feet-high unless supported by shoring. Bench back slopes should be inclined no steeper than 0.75H:1V or as required by the State of California Construction Safety Orders. Care should be taken to adequately compact the fill against the exposed bedrock. The upper 8 inches of the exposed surface should be scarified, moisture conditioned, and compacted to 90 percent relative compaction (ASTM D1557) prior to placement of structural fill.

### 5.2.4 Structural Fill

On-site soils that are free of organic matter and do not contain rocks over 6 inches in diameter will generally be satisfactory for re-use as general structural fill. However, on-site expansive soils (typically boring log classification symbols CH and CL with plasticity indices greater than 15 percent) should not be used as structural fill. Structural fill should meet the following gradation requirements:

<u>Sieve Analysis</u>	<u>Percent Passing</u>
6 inch	100
4 inch	85-100
No. 200	30-50

Additional processing may be required to break large rock fragments to conform to the above specified gradation. Processing may entail track-walking of rock fill materials either in the fill area or on a processing pad to breakdown large rock fragments with the oversize fraction being disposed of off-site.

#### 5.2.5 Select fill

Imported or on-site select fill should be of low expansion potential and free of organic matter, and should conform, in general, to the following requirements:

Plasticity Index	less than 15%
Liquid Limit	less than 40%
Percent Soil Passing #200 Sieve	between 15% and 60%
Maximum Aggregate Size	4 inches

#### 5.2.6 Fill Placement

Fill should be spread in thin lifts, moisture conditioned to near optimum moisture content, and compacted to the relative compaction specifications presented below. Fill (or cut) subgrade soils should be finished to present smooth, unyielding surfaces. Subgrade soils should be maintained at their moist or above optimum moisture contents and be free of shrinkage cracks until covered by permanent construction. A summary of our compaction requirements is presented in the following Table 1.

**TABLE 1  
SUMMARY OF COMPACTION REQUIREMENTS**

<b>Area</b>	<b>Compaction Requirements</b>
Structural and Select Engineered Fill	In lifts, a maximum of 8 inches loose thickness, compact to a minimum of 90 percent relative compaction at or within 2 percent of the optimum moisture content for select (non-expansive) fill.
Trenches	In lifts, a maximum of 8 inches loose thickness, compact to at least 90 percent relative compaction at or within 2 percent of the optimum moisture content for select, non-expansive fill.
Parking and Access Driveway	Compact the top 6 inches of subgrade soil to at least 95 percent relative compaction at or within 2 percent of optimum moisture content for select (non-expansive) fill. Soils should be maintained at their moist or above optimum moisture contents until covered by permanent construction.
Concrete Slab-On-Grade	Compact the top 6 inches of select subgrade soil to at least 90 percent relative compaction at or within 2 percent of the optimum moisture content.
Exterior Flatwork	Compact the top 6 inches of select subgrade soils to at least 90 percent relative compaction at or within 2 percent of the optimum moisture content for select (non-expansive) fill.

When grading is performed in the winter, spring, or early summer, there is a risk that the site soils/weathered rock may be saturated and too soft to support construction equipment. Normally suitable fill material may be too wet to properly compact and excavation bottoms can become unstable. Such soil/weathered rock conditions could be mitigated by over-excavation and backfilling with more expensive, imported fill and/or other means.

Permanent cut and fill slopes should be constructed no steeper than 2.5H:1V. If steeper slopes are needed, soil reinforcing will be required. To limit erosion and the transport of sediment into local drainage systems, erosion protection measures (such as hydroseeding and/or straw mats to be determined by the project Civil Engineer) should be installed in graded areas in accordance with state and local government ordinances.

Site preparation and grading operations should be observed by a representative of Kleinfelder. This will allow us to check whether unforeseen or detrimental materials are exposed by the construction equipment and to modify our recommendations, if necessary.

### 5.3 TEMPORARY EXCAVATION AND BACKFILL

Shallow excavations for footings and utility trenches can readily be made with either a backhoe or trencher. We expect the walls of trenches less than 5-feet-deep to remain in a near-vertical configuration during utility construction provided equipment or excavated spoil surcharges are not located near the top of the excavation. Where trenches extend deeper than 5 feet, the excavation can become unstable. Groundwater may also be encountered in shallow excavations for utilities depending on the time of year construction commences. As such, the Contractor should be prepared to dewater the utility excavations by pumping from temporary sumps or by other similar methods, at the Contractor's option.

We recommend a minimum compaction of trench backfill as previously presented in Section 5.2.6, Table 1. Care should be taken to adequately compact utility trench backfill in all structure areas including pavements. Poor compaction will likely cause subsequent settlement of the trench, resulting in possible distress cracking to the overlying structure or pavement.

### 5.4 FOUNDATIONS

#### 5.4.1 Spread Footing Design Criteria

The bottom of the excavations for the building will be in bedrock. The building should be supported on either spread footings bottomed on bedrock, drilled piers supported in bedrock, or a combination of the two. Small structures, not critical to differential settlement, can be supported on spread footings bottomed in either compacted fill or stiff native soil. Where structures are supported on or into bedrock, we estimate that post-construction total foundation settlement for column loads up to 100 kips will be less than ½-inch. Post-construction differential settlement between foundation elements is estimated to be less than ¼-inch. Settlement for column loads

greater than 200 kips will need to be evaluated on a case-by-case basis. The following bearing capacity values in pounds per square foot (psf) should be used for spread footings:

<u>LOAD</u>	<u>ENGINEERED FILL</u>	<u>BEDROCK</u>
Dead Load	2000 allow. 7500 ult.	4500 allow. 15000 ult.
Dead +Live Load	2500 allow. 7500 ult.	5000 allow. 15000 ult.
Total Load	3000 allow. 7500 ult.	5500 allow. 15000 ult.

Individual spread footings should be at least 18 inches wide and continuous footings should be at least 12 inches wide. All footings should be bottomed at least 24 inches below the lowest adjacent grade. The bottom of footing excavations should be firm and free of loose soil, rock, and standing water.

#### 5.4.2 Lateral Resistance

Lateral loads on spread footing foundations should be resisted by friction on the bottoms of footings and passive pressure on the sides of footings. A factor of 0.3 times the dead load should be used to determine allowable friction resistance in soil and bedrock; the ultimate friction value is 0.58. A value of 350 pounds per cubic foot (pcf) equivalent fluid weight (efw) should be used to determine allowable passive pressure in soil. The ultimate value is 700 pcf. A value of 2000 psf should be used in bedrock to determine the passive pressure resistance; the ultimate value is 4000 psf. The upper foot of soil or bedrock should be neglected in computing passive pressure resistance unless covered with pavement or concrete slabs.

Lateral loads can be resisted by the drilled piers. The lateral capacity of piers should be determined by using LPILE\* when the pier sizes, vertical and lateral loads, allowable deflections, and location of piers in relation to the existing or proposed slopes are known.

\* LPILE is a program licensed by ENSOFT, INC., P.O. Box 180348, Austin, Texas 78718

Lateral loads also can be resisted by passive pressure against grade beams and pile caps as previously described above for spread footings.

#### 5.4.3 Drilled Pier Design Criteria

A drilled pier foundation should consist of piers drilled at least 10 feet into bedrock. The piers should be at least 18 inches in diameter and should be connected with grade beams. The pier capacity should be obtained from skin friction in the bedrock. Piers should be spaced at least three diameters on-center for vertical loads. The upper two diameters of the pier and lower one pier diameter should be neglected in computing the capacity. End bearing should not be included in the design. The skin friction values shown in the following table should be used.

<u>LOAD</u>	<u>BEDROCK</u>
Dead +Live Load	1000 allow. 2000 ult.
Total Load	1300 allow. 2000 ult.

The above values should be decreased by 30 percent when used for resisting uplift loads.

Piers should be deepened as necessary so as to provide at least seven feet of horizontal confinement to the nearest slope face. For closer pier spacing, there will be a capacity reduction factor that should be applied to the pier capacities as determined by the Geotechnical Engineer. Piers should be reinforced for their full depth as designed by the Structural Engineer. The above pressures are design values for use in working stress analyses and contain an appropriate factor of safety.

To retard wet concrete from settling, the pier holes should not contain more than three inches of slough or cuttings. It may be necessary to remove or tamp the slough prior to concrete placement as determined in the field by the Geotechnical Engineer.

No caving soils or bedrock were encountered during our exploration. However, groundwater and/or caving soils or bedrock could be encountered and should be anticipated during pier

drilling. If caving soils or bedrock are encountered, it may be necessary to case the holes. If groundwater is present, it may be necessary to dewater the holes or to place the concrete by an approved pumping method.

### 5.5 RETAINING WALLS (15 FEET OR LESS IN HEIGHT)

Lateral pressures on retaining walls that are free to rotate or fixed at the top should be determined using the equivalent fluid weights shown in the following table.

<u>BACKFILL SLOPE</u>	<u>FREE TO ROTATE</u>	<u>FIXED</u>
Level	45 pcf	60 pcf
3H:1V	50 pcf	70 pcf
2H:1V	60 pcf	80 pcf

A traffic surcharge should be added to the walls where appropriate. The surcharge load can be estimated by assuming two feet of additional soil load behind the wall. If a seismic load is applied to the wall, the pressure should be determined by using  $12H$  psf, where  $H$  is the wall height. The seismic load should be applied as a uniform pressure over the wall height.

Lateral wall forces can be resisted by friction on the bottom of the foundation and passive pressure against the face of the foundation. The allowable friction factor should be 0.3 and the allowable passive pressure should be determined using an equivalent fluid weight of 350 pcf. The upper foot of soil should be neglected unless it is covered with pavement. If piers are used for wall foundations, the lateral resistance can be provided by the piers.

Retaining wall foundations should be designed as previously described in Section 5.4.

### 5.6 TIE-BACK WALLS

Where the wall height is about 15 feet or more, a tie-back anchored or soil nail wall may be appropriate. We recommend that tie-back anchored walls be used for the higher walls. If soldier piles are used, they should extend a minimum of 10 feet below the bottom of excavation.

### 5.6.1 Lateral Pressures

Anchored tie-back walls should be designed for a soil pressure (p) using the formula  $p=0.65K_a\delta H$ , where  $K_a$  is the coefficient of active pressure,  $\delta$  is the effective unit weight (130 pcf), and H is the wall height. The soil pressure should be greater than 50H, as discussed in Appendix D, and provide a factor of safety greater than 1.5 for permanent construction.  $K_a$  values for various backfill slopes are shown in the following table. The soil pressure increase should be applied as a triangular pressure down to the first anchor and then as a uniform pressure over the remaining wall height. If a seismic load is to be applied to the wall, the pressure should be determined using the formula 12H psf. This pressure should be applied over the wall height. These recommendations do not include loads imposed by adjacent structures. We can assist in evaluating the affects of these additional loads on the wall design if needed.

<u>BACKFILL SLOPE</u>	<u><math>K_a</math> VALUES</u>
Level	0.46
3H:1V	0.56
2H:1V	0.76

### 5.6.2 Rock Bolts/Tie-backs

Allowable bond-strength values of 1000 psf for dead load plus live load and 1300 psf for total load should be used for estimating tie-back anchor and rock bolt capacities in bedrock. The ultimate bond-strength value is 2000 psf. The values given are for gravity grouting. Somewhat higher values would normally be obtained with pressure grouting. All bond-strength values should be confirmed by field performance tests. The specifications should be written so that the Contractor determines the size of the anchor and grouting procedure within Post Tension Institute (PTI, 2004) guidelines.

The anchors should be bonded in bedrock and the anchor tip should extend at least to a 2H:1V influence line that slopes up from the toe of the wall, or greater as more detailed analysis dictates. Bonded lengths should be a minimum of 20-feet-long. At least 10 percent of the anchors should be performance tested and **all** of the anchors should be proof tested in accordance with PTI standards (2004).

Anchors in the type of bedrock encountered at the site are frequently about six feet on-center, both vertically and horizontally, with the top row about three feet below the ground surface. The anchors and wall at each level should be installed and the anchors tested before the next level is excavated.

## 5.7 WALL DRAINAGE

All retaining walls should be backdrained with a drain rock and pipe system or with geotextile drain material. Backdrains should consist of 4-inch-diameter pipe embedded in drain rock. The pipe should be PVC or ABS with a SDR of 35 or better, and the pipe should be sloped to drain to outlets by gravity. Drain rock should consist of clean, free-draining crushed rock or gravel. The rock should be wrapped in filter fabric, Mirafi 140N or equivalent. Caltrans Class 2 Permeable Material can be used in place of the crushed rock or gravel. If Caltrans Class 2 Permeable Material is used, the fabric can be omitted. The top of the pipe should be at least eight inches below the lowest adjacent grade. The drain rock should extend to within one foot of the surface. The upper one-foot should be backfilled with compacted clayey soil to exclude surface water unless capped by a paved surface. Where migration of moisture through retaining or basement walls would be detrimental, the walls should be waterproofed.

Backdrainage for anchored walls should be a strip of Ameridrain 500 or equal with a drain-rock surrounded PVC or ABS perforated collector pipe at the wall bottom. The drain strips should be at least 2-feet-wide and centered between tie-backs. Drain strips should be wider than 2 feet, if practical.

## 5.8 CONCRETE SLABS-ON-GRADE

Concrete slabs-on-grade for this project may consist of interior living space, exterior flatwork, and walkways. To minimize the potentially adverse affects of expansive soil shrink/swell movement on slabs, we recommend that either native or imported select fill be placed and compacted as previously described in Section 5.2 of this report.

Interior concrete slabs-on-grade should be supported on at least 4 inches of slab base rock to provide a capillary moisture break from the underlying soil. This rock should be graded such that 100 percent passes the 1½-inch sieve and no more than 5 percent passes the No. 4 sieve. If the subgrade soils are allowed to dry out prior to concrete slab-on-grade construction, they should be re-moisture conditioned. The slab subgrade soils should be maintained as smooth, unyielding, free of loose materials and in a moist condition until slab base rock and concrete are placed.

Interior concrete slabs should be a minimum of 6-inches-thick and should be reinforced according to the recommendations set forth by the project Structural Engineer. In addition, slabs should be scored for crack control as recommended by the project Structural Engineer and/or Architect.

We recommend exterior slabs that may be subjected to traffic loads be a minimum of six inches thick and reinforced according to the recommendations set forth by the Structural Engineer. During construction, care should be taken such that reinforcement is placed at the slab mid-height, particularly when using welded-wire fabric. The exterior slabs should be separated from adjacent foundations by low-friction felt or mastic materials to allow for some differential movement at this interface.

Subsurface moisture and moisture vapor naturally migrate upward through soil and, where the soil is covered by a building or pavement, this subsurface moisture will collect. To reduce the impact of subsurface moisture and the potential impact of moisture that could be introduced in the future (such as landscape irrigation, precipitation, or leaking pipes), the current industry standard is to place a vapor retardant on the crushed rock layer. This membrane typically consists of visquene or polyvinyl plastic sheeting at least 20-mil in thickness. It should be noted that, although vapor barrier systems are currently the industry standard, this system may not be completely effective in preventing floor slab moisture problems. These systems typically will not necessarily assure that floor slab moisture transmission rates will meet floor-covering manufacturer standards and that indoor humidity levels be appropriate to inhibit mold growth. The design and construction of such systems are totally dependent on the proposed use and

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

Special precautions must be taken during the placement and curing of 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 concrete placement and curing operations be performed in accordance with the American Concrete Institute (ACI) Manual.

It should be emphasized that Kleinfelder personnel are not moisture proofing experts for floors and/or retaining walls. We make no guarantee, nor provide any assurance, that use of the capillary break/vapor retardant 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.

## 5.9 SURFACE AND SUBSURFACE DRAINAGE CONTROL

The area adjacent to the building should be sloped so as to provide positive surface drainage. Slope gradients should be a minimum of four percent extending at least five feet out from the buildings. In general, the roofs should be provided with gutters and downspouts that discharge

into rigid pipes with watertight joints that outflow into the site storm drain system or an appropriate outlet area.

Where irrigated landscape areas abut the building, excess water can be introduced into soil layers along building edges, tending to soften soils and increase the risk of potential migration of moisture into under-floor areas. Planned landscaping may need surface and subsurface drainage improvements. We can evaluate the need for such drainage facilities during final design, if requested.

Beneath the perimeter of the building, all utility trenches should be backfilled with compacted non-pervious fill material a minimum of five feet on either side of the foundation to reduce water infiltration into the under-floor area of the buildings. Special care should be taken during installation of sub-floor water and sewer lines to reduce the possibility of leaks.

## 6.0 ADDITIONAL SERVICES AND LIMITATIONS

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### 6.1 ADDITIONAL SERVICES

Kleinfelder recommends that we be retained to review the preliminary structural design to provide supplemental geotechnical investigation and recommendations, as required. Kleinfelder should also be retained to review the final project plans and specifications to determine if they are consistent with the recommendations presented in this report. In addition, we should be retained to observe and test during project construction. Additional information on subsurface conditions at the site will become available during the course of construction. As such, the review of project plans and specifications, along with field observation and testing during earthwork by Kleinfelder, are an integral part of the conclusions and recommendations made in this report and are required for Kleinfelder to remain Engineer-of-Record throughout the project. If Kleinfelder is not retained for these services, then Lawrence Berkeley National Laboratory (Owner) will be assuming Kleinfelder's responsibility for any potential claims that may arise during, or after, construction and Kleinfelder will cease to be the geotechnical Engineer-of-Record. The recommended tests, observations, and consultation by Kleinfelder prior to and during construction include but are not limited to:

- Prior to selection of the structure foundation system, our office should review the plans and provide supplemental geotechnical recommendations, as required.
- Prior to construction, our office should review the plans and specifications to check that they are in conformance with the recommendations contained in this Geotechnical Report.
- During construction, our engineer should observe and/or test the following:
  - Pre-construction meeting.
  - Site grading and excavation, including site stripping, removal of undocumented fill, and engineered fill construction.
  - Sub-grade preparation.
  - In-place density testing of fills, backfills, and finished sub-grades.
  - Observation of all foundation excavations.

These supplemental services would be performed on an as-requested basis and would be in addition to the fee charged for this geotechnical investigation. If others perform such construction observation, we cannot be responsible for their interpretation of our conclusions and recommendations presented herein.

We have provided the Client with 4 bound original copies of this report. If additional copies are required, we can provide them at an additional fee (in accordance with our current fee schedule) after receipt of a written request from our Client. **Under no circumstances will we provide a copy of the report to other design consultants or contractors without written permission from our Client.**

If more than 36 months have elapsed between the submission of this report and the start of subsequent project construction, or if conditions have changed because of natural causes or other construction operations at or adjacent to the site, the recommendations made in this report may no longer be valid or appropriate. In such cases, we recommend that this report be reviewed by us to determine the applicability of the conclusions and recommendations considering the time lapsed or changed conditions. The recommendations made in the report are contingent upon such a review.

## 6.2 LIMITATIONS

This report has been prepared by Kleinfelder for the exclusive use of Lawrence Berkeley National Laboratory and their consultants for development of the proposed project described in this report. In addition, a brochure prepared by ASFE (Association of Firms Practicing in the Geosciences) has been included at the front of this report. We recommend that all individuals reading this report also read this attached brochure.

Our services consist of professional opinions and conclusions developed in accordance with generally accepted geotechnical engineering principles and practices. We provide no other warranty, either expressed or implied. The conclusions and recommendations contained in this report are based on our new exploratory borings, laboratory testing, and engineering analysis.

We have also reviewed existing subsurface data from a number of sources listed in Section 2.3 of this report that we have assumed is representative of bedrock conditions at the site but it is possible that subsurface conditions could vary between or beyond the points explored. If soil and groundwater conditions are encountered during construction that differ from those described herein, our firm should be notified immediately in order that a review may be made and supplemental recommendations provided, if warranted. If the scope of the proposed construction, including the type of structures and planned grading, changes from that described in this report, our recommendations should also be reviewed and modified where necessary. Verification of our conclusions and recommendations is subject to our review of the project plans and specifications, and our observation of subsequent project construction.

Site conditions and cultural features described in the text of this report are those existing at the time of our investigation and as encountered in our subsurface exploration for this study and may not necessarily be the same or comparable at other times.

The scope of our services did not include an environmental assessment or an investigation of the presence or absence of hazardous or toxic materials in the soil, surface water, groundwater, or air on, below, or around this site.

This report may be used only by Lawrence Berkeley National Laboratory and only for the purposes stated, within a reasonable time from its issuance, but in no event later than 36 months from the date of the report. Land or facility use, on- and off-site conditions, regulations or other factors may change over time and additional work may be required with the passage of time. Any party other than the client who wishes to use this report shall notify Kleinfelder of such intended use. Based on the intended use of the report, Kleinfelder may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the client or anyone else will release Kleinfelder from any liability resulting from the use of this report by any authorized party; and client agrees to defend, indemnify, and hold harmless Kleinfelder from any claim or liability associated with such unauthorized use or non-compliance.

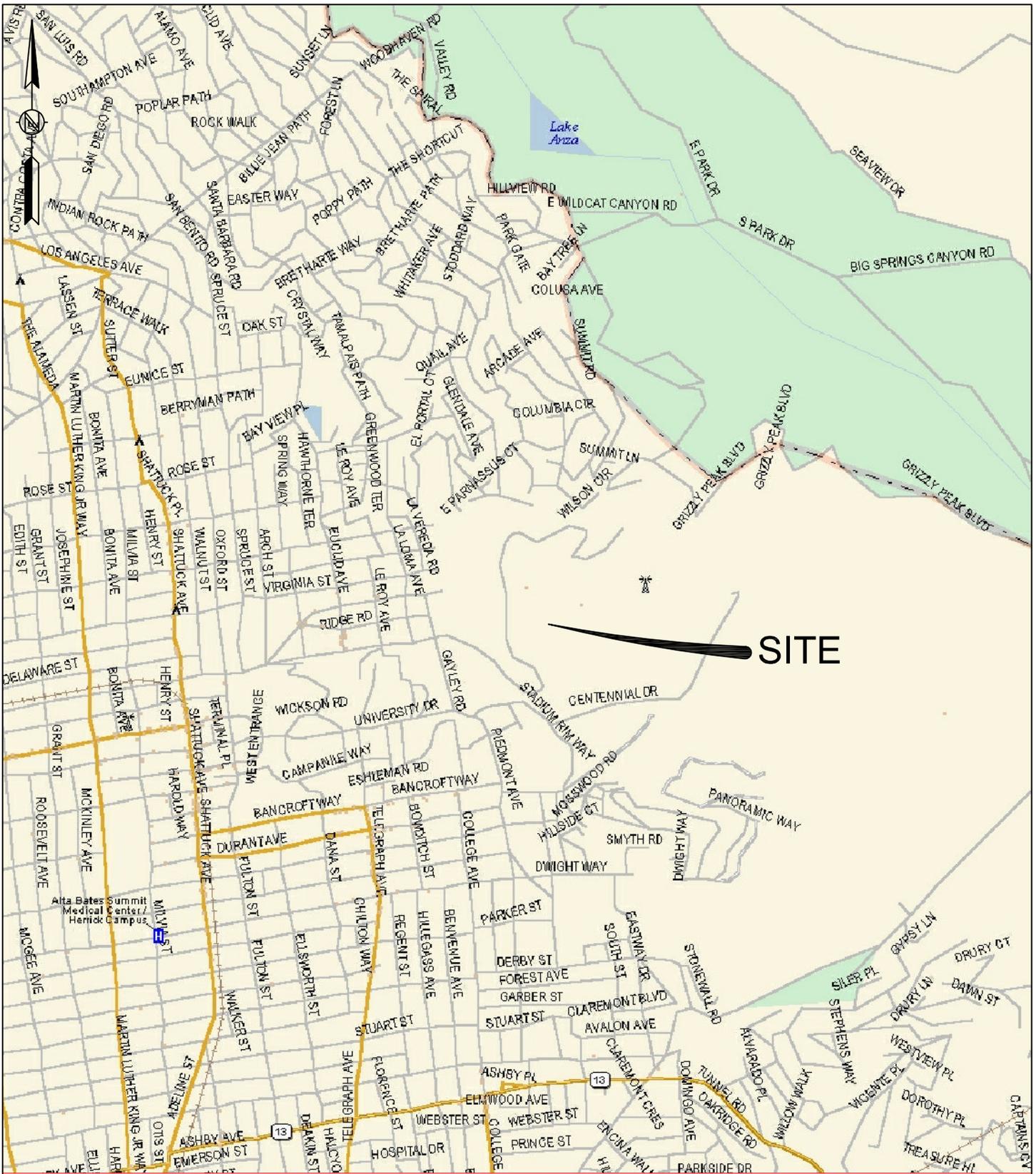
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# PLATES

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Base: Delorme.



**KLEINFELDER**

PROJECT NO. 74911

DATE JUNE 2007

**SITE LOCATION**

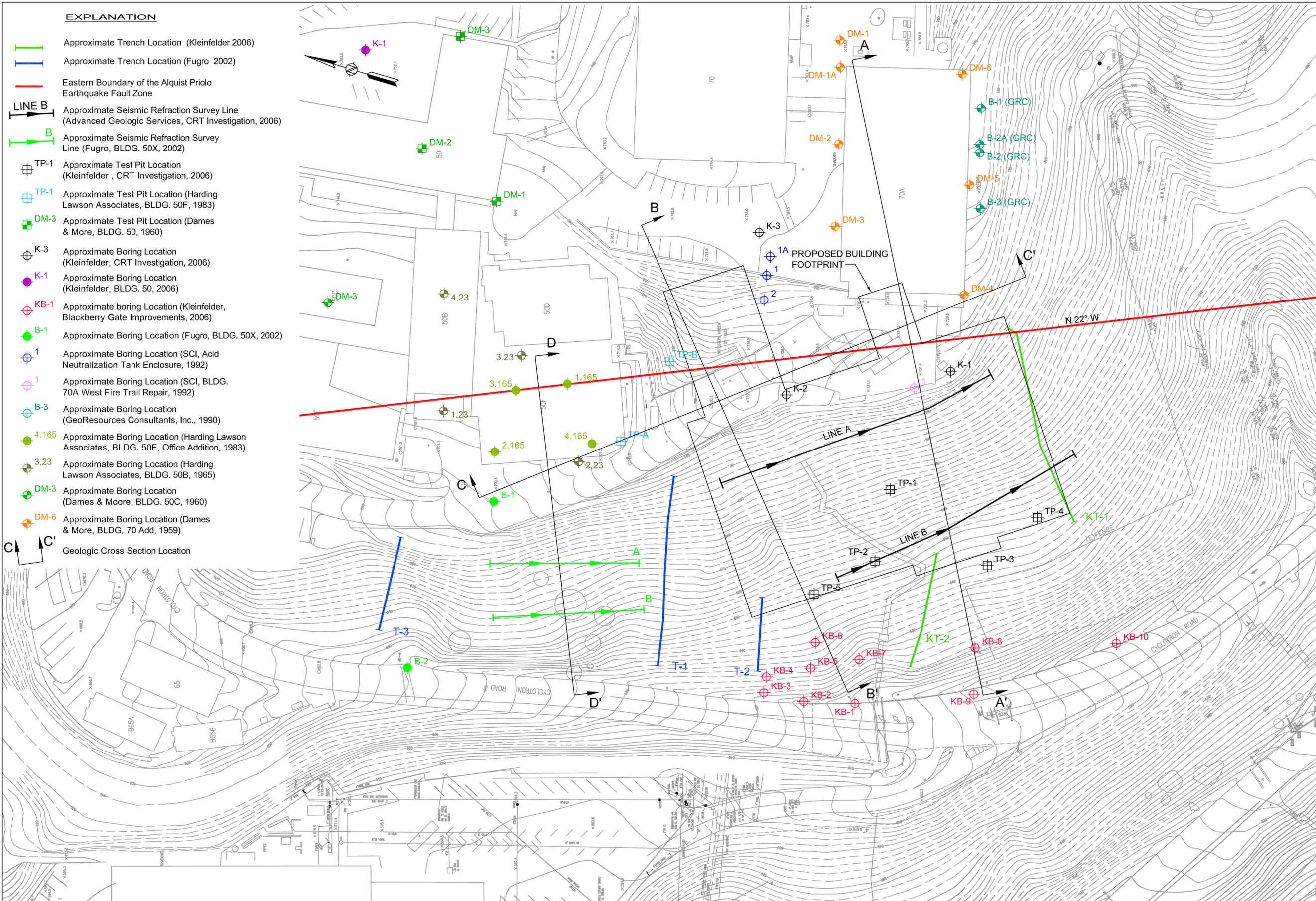
CRT Building  
 Lawrence Berkeley National Laboratory  
 Berkeley, California

PLATE

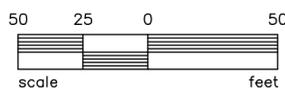
1

**EXPLANATION**

-  Approximate Trench Location (Kleinfelder 2006)
-  Approximate Trench Location (Fugro 2002)
-  Eastern Boundary of the Alquist Priolo Earthquake Fault Zone
-  **LINE B** Approximate Seismic Refraction Survey Line (Advanced Geologic Services, CRT Investigation, 2006)
-  **B** Approximate Seismic Refraction Survey Line (Fugro, BLDG. 50X, 2002)
-  TP-1 Approximate Test Pit Location (Kleinfelder, CRT Investigation, 2006)
-  TP-1 Approximate Test Pit Location (Harding Lawson Associates, BLDG. 50F, 1983)
-  DM-3 Approximate Test Pit Location (Dames & More, BLDG. 50, 1960)
-  K-3 Approximate Boring Location (Kleinfelder, CRT Investigation, 2006)
-  K-1 Approximate Boring Location (Kleinfelder, BLDG. 50, 2006)
-  KB-1 Approximate boring Location (Kleinfelder, Blackberry Gate Improvements, 2006)
-  B-1 Approximate Boring Location (Fugro, BLDG. 50X, 2002)
-  1 Approximate Boring Location (SCI, Acid Neutralization Tank Enclosure, 1992)
-  1 Approximate Boring Location (SCI, BLDG. 70A West Fire Trail Repair, 1992)
-  B-3 Approximate Boring Location (GeoResources Consultants, Inc., 1990)
-  4.165 Approximate Boring Location (Harding Lawson Associates, BLDG. 50F, Office Addition, 1983)
-  3.23 Approximate Boring Location (Harding Lawson Associates, BLDG. 50B, 1965)
-  DM-3 Approximate Boring Location (Dames & Moore, BLDG. 50C, 1960)
-  DM-6 Approximate Boring Location (Dames & More, BLDG. 70 Add, 1959)
-  **C** Geologic Cross Section Location



Base: Lawrence Berkeley National Laboratory, 2006

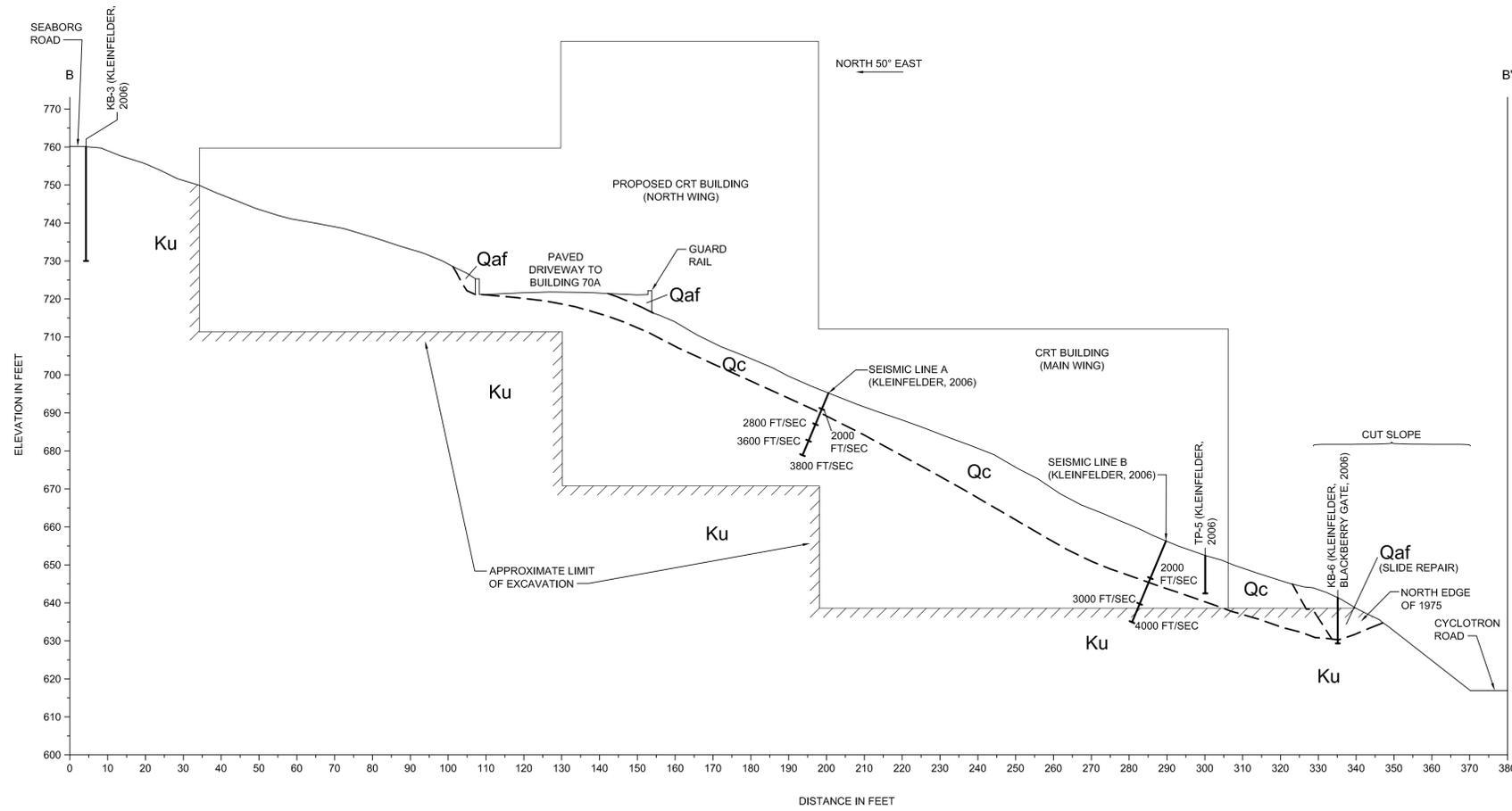


 **KLEINFELDER**  
 PROJECT NO. 74911      DATE JUNE 2007

**SITE PLAN**  
 Geotechnical Investigation  
 CRT Building  
 Lawrence Berkeley National Laboratory  
 Berkeley, California

PLATE  
**2**





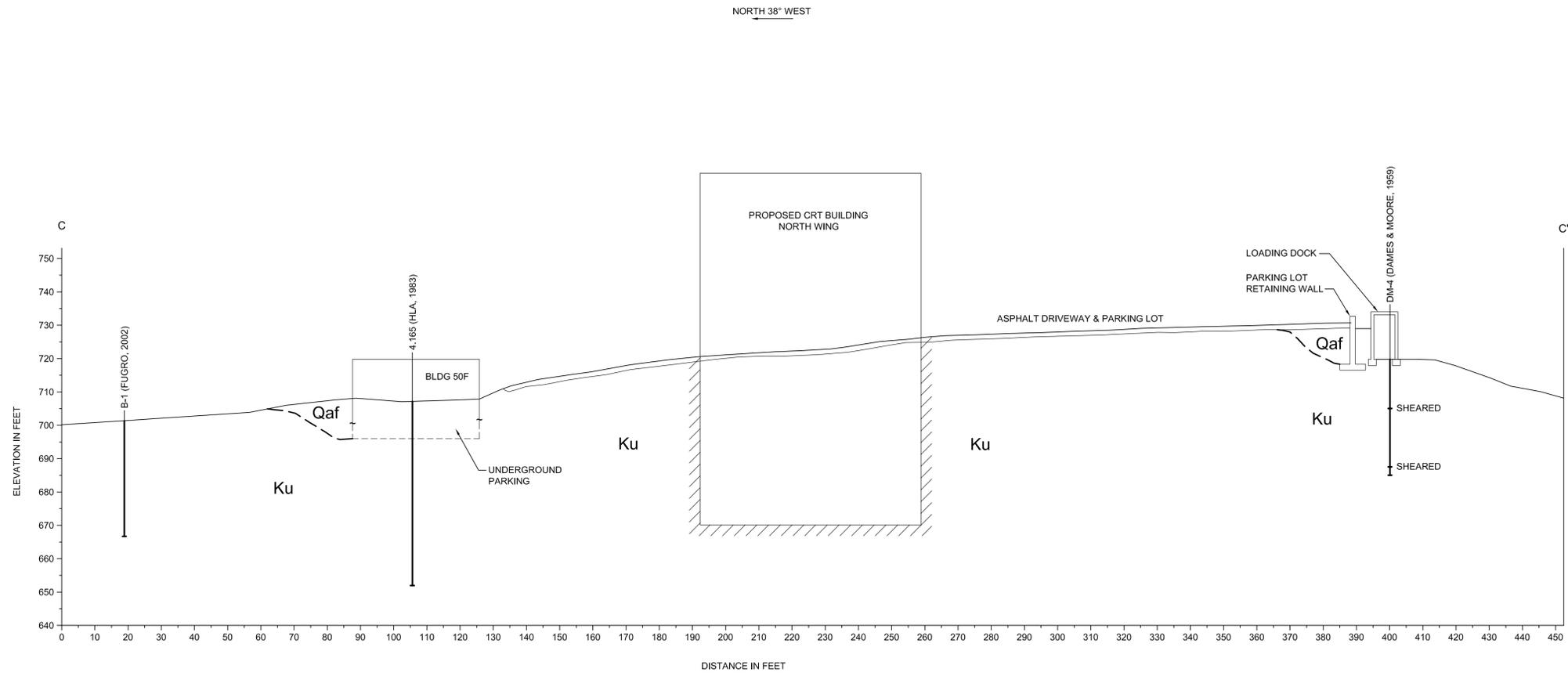
**EXPLANATION**

<b>Qaf</b>	Artificial Fill
<b>Qc</b>	Colluvium
<b>Ku</b>	Cretaceous Undifferentiated (Sandstone, Siltstone, Shale)



	<b>GEOLOGIC SECTION B - B'</b> CRT BUILDING LAWRENCE BERKELEY NATIONAL LABORATORY Berkeley, California		PLATE <b>4</b>
	PROJECT NO. 74911	DATE JUNE 2007	

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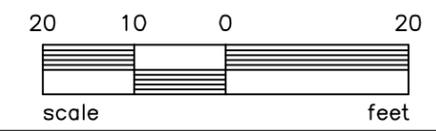
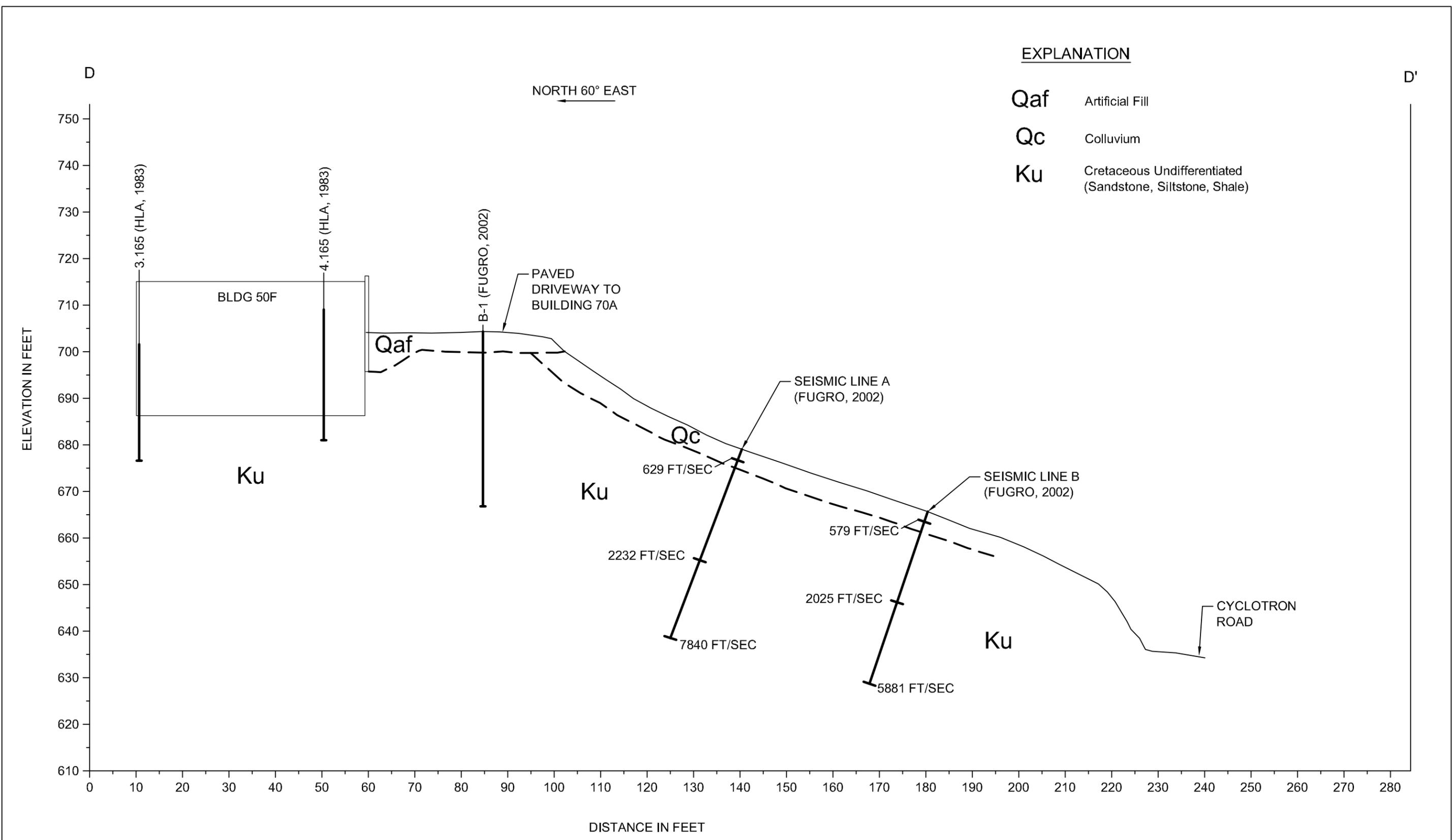
**EXPLANATION**

- Qaf** Artificial Fill
- Qc** Colluvium
- Ku** Cretaceous Undifferentiated (Sandstone, Siltstone, Shale)



	<b>GEOLOGIC SECTION C - C'</b> CRT BUILDING LAWRENCE BERKELEY NATIONAL LABORATORY Berkeley, California	PLATE <b>5</b>
PROJECT NO. 74911	DATE JUNE 2007	

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	<b>GEOLOGIC SECTION D - D'</b> CRT BUILDING LAWRENCE BERKELEY NATIONAL LABORATORY Berkeley, California	PLATE <b>6</b>
	PROJECT NO. 74911	DATE JUNE 2007

---

# APPENDIX A

---

## UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS		DESCRIPTIVE NAMES		
<b>COARSE GRAINED SOILS</b> More than Half > #200 sieve	<b>GRAVELS</b>  MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS WITH LITTLE OR NO FINES	<b>GW</b> WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES	
		GRAVELS WITH OVER 15% FINES	<b>GP</b> POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES	
		<b>SANDS</b>  MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS WITH LITTLE OR NO FINES	<b>SW</b> WELL GRADED SANDS, GRAVELLY SANDS
			SANDS WITH OVER 15% FINES	<b>SP</b> POORLY GRADED SANDS, GRAVELLY SANDS
	<b>FINE GRAINED SOILS</b> More than Half < #200 sieve	<b>SILTS AND CLAYS</b>  LIQUID LIMIT LESS THAN 50	<b>GM</b> SILTY GRAVELS, POORLY GRADED GRAVEL-SAND-SILT MIXTURES	
			<b>GC</b> CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES	
			<b>SM</b> SILTY SANDS, POORLY GRADED SAND-SILT MIXTURES	
		<b>SILTS AND CLAYS</b>  LIQUID LIMIT GREATER THAN 50	<b>SC</b> CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES	
<b>ML</b> INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS WITH SLIGHT PLASTICITY				
<b>CL</b> INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS				
<b>SILTS AND CLAYS</b>  LIQUID LIMIT GREATER THAN 50	<b>OL</b> ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY			
	<b>MH</b> INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS			
	<b>CH</b> INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS			
<b>HIGHLY ORGANIC SOILS</b>		<b>OH</b> ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS		
		<b>Pt</b> PEAT AND OTHER HIGHLY ORGANIC SOILS		

### FIELD SAMPLING

	CALIFORNIA SAMPLE 2.5" I.D.
	MODIFIED CALIFORNIA SAMPLE 2" I.D.
	DISTURBED, BAG OR BULK SAMPLE
	STANDARD PENETRATION TEST
	SHELBY TUBE SAMPLE
	3-1/2" I.D. CONTINUOUS CORE SAMPLE
	UNRETAINED PORTION OF SAMPLE
	WATER LEVEL OBSERVED IN BORING (at given post-drilling time)
	WATER LEVEL OBSERVED IN BORING (at time of drilling)

### LABORATORY TESTS

LL	LIQUID LIMIT
PI	PLASTICITY INDEX
SA	SIEVE ANALYSIS
#200	PERCENT PASSING #200 SIEVE
RV	RESISTANCE VALUE
EI	EXPANSION INDEX
DS	DIRECT SHEAR
Tx/UU	TRIAxIAL SHEAR-UNCONSOLIDATED UNDRAINED
UC	UNCONFINED COMPRESSION
SG	SPECIFIC GRAVITY
PP	POCKET PENETROMETER SHEAR STRENGTH (tsf)

NOTES:

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 strata and groundwater observed at the boring location on the date of drilling only.



### BORING LOG LEGEND

**CRT Building**  
**Lawrence Berkeley National**  
**Laboratory**  
**Berkeley, California**

PLATE

**A-1**

PROJECT NUMBER **74911**

DATE **Jun 2007**

GRAPHIC ROCK SYMBOLS

 SHALE OR CLAYSTONE	 CHERT	 SERPENTINITE
 SILTSTONE	 PYROCLASTIC	 METAMORPHIC ROCKS
 SANDSTONE	 VOLCANIC FLOWS	 ALTERED ROCKS
 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 LITTLE FRACTURED	Greater than 4.0 feet
OCCASIONALLY FRACTURED	1.0 to 4.0 feet
MODERATELY FRACTURED	0.5 to 1.0 feet
CLOSELY FRACTURED	0.1 to 0.5 foot
INTENSELY FRACTURED	0.05 to 0.1 foot
CRUSHED	Less than 0.05 foot

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



**ROCK DESCRIPTION CRITERIA**

**CRT Building**  
**Lawrence Berkeley National**  
**Laboratory**  
**Berkeley, California**

PLATE

**A-2**

PROJECT NUMBER **74911**

DATE **Jun 2007**

LABORATORY				FIELD		Depth (feet)	Lithology Symbol	U.S.C.S. Designation	SOIL DESCRIPTION
Dry Density (pcf)	Moisture Content (%)	Shear Strength (ksf)	Other Tests	Blows/foot *	Sample				
						1	GM/GC		<b>ASPHALT CONCRETE</b> = 7 inches
						2			<b>AGGREGATE BASE</b> = 12 inches
				18		3			<b>GRAVELLY SANDY CLAY-</b> dark yellow brown, moist, very stiff, subangular gravels up to 1" (fill)
						4			
115	12.8			25		5	CL		
						6			
						7			<b>SILTSTONE-</b> dark yellow brown, completely weathered, very closely fractured, very weak
						8			
116	8.8			35		9			
						10			
						11			
						12			
						13			<b>SHALE-</b> gray, highly weathered, very closely fractured, weak, thinly bedded
						14			
111	13.5			55		15			
						16			
						17			
						18			
						19			very difficult drilling
						20			
				55		21			
						22			
						23			
						24			
						25			<b>SANDSTONE-</b> yellow brown, highly weathered, very closely fractured, weak
						26			
				70/3"		27			BOTTOM OF BORING K-1 @ 27 FEET No Free Water Encountered
						28			
						29			
						30			
						31			
						32			
						33			
						34			
						35			

\* The blow counts have been converted to standard N-value blow counts.  
 \*\* LBNL Datum

SURFACE ELEVATION: 712' \*\*  
 TOTAL DEPTH: 27.0 feet  
 GROUND WATER DEPTH: ∇ feet at time of drilling  
 ∇ feet

LOGGED BY: R. Padgett  
 EQUIPMENT: Mobile B-53  
 DIAMETER of BORING: 6  
 DATE DRILLED: 9-18-06



**LOG OF EXPLORATION  
 BORING K-1  
 CRT Building  
 Lawrence Berkeley National  
 Laboratory  
 Berkeley, California**

PLATE  
**A-3**  
 1 of 1

PROJECT NUMBER 74911 DATE Jun 2007

LABORATORY				FIELD		Depth (feet)	Lithology Symbol	U.S.C.S. Designation	SOIL DESCRIPTION
Dry Density (pcf)	Moisture Content (%)	Shear Strength (ksf)	Other Tests	Blows/foot *	Sample				
						0			<b>ASPHALT CONCRETE</b> = 5 inches
						1	GM/GC		<b>AGGREGATE BASE</b> = 9 inches
						2			<b>CLAYEY SANDY GRAVEL-</b> light brown, dry, medium dense, fine to coarse grained sand, fine rounded gravel to 1/2" angular gravel (fill)
						3			
						4	GC		
				18		5			Encountered Concrete Duct Bank Boring Terminated
						6			BOTTOM OF BORING K-2 @ 5.5 FEET No Free Water Encountered
						7			
						8			
						9			
						10			

\* The blow counts have been converted to standard N-value blow counts.  
 \*\* LENL Datum

SURFACE ELEVATION: **719'** \*\*  
 TOTAL DEPTH: **5.5 feet**  
 GROUND WATER DEPTH: ∇ **feet at time of drilling**  
 ∇ **feet**

LOGGED BY: **R. Padgett**  
 EQUIPMENT: **Mobile B-53**  
 DIAMETER of BORING: **6**  
 DATE DRILLED: **9-18-06**



**LOG OF EXPLORATION  
 BORING K-2  
 CRT Building  
 Lawrence Berkeley National  
 Laboratory  
 Berkeley, California**

PLATE  
**A-4**  
 1 of 1

LABORATORY				FIELD		Depth (feet)	Lithology Symbol	U.S.C.S. Designation	SOIL DESCRIPTION
Dry Density (pcf)	Moisture Content (%)	Shear Strength (ksf)	Other Tests	Blows/foot*	Sample				
117	14.2			42		1	GM/GC	<b>ASPHALT CONCRETE</b> = 4 inches <b>AGGREGATE BASE</b> = 12 inches	
124	11.6			54		2		<b>SILTSTONE</b> - yellow brown with blue/gray clay seams, highly weathered, very closely fractured, very weak	
						3			
						4			
						5			
						6			
						7			
						8		stiffer drilling	
						9			
				53/8"		10		<b>SANDSTONE</b> - yellow brown to brown, highly to completely weathered, very closely fractured, very weak	
						11			
						12			
						13		<b>SILTSTONE</b> - yellow brown with blue/gray clay seams, highly weathered, very closely fractured, very weak	
118	13.4			42		14			
						15			
						16			
						17			
				50/3"		18		<b>SANDSTONE</b> - yellowish brown to reddish brown, moderately weathered, very closely fractured, friable	
				50/3"		19			
						20			
						21			
						22			
						23			
						24			
						25			
						26			
						27		<b>SHALE</b> - blue gray, highly to completely weathered, very closely fractured, very weak, highly sheared with strong, rounded corestones	
				34		28			
						29			
				72/7"		30			
						31			
						32		BOTTOM OF BORING K-3 @ 30 FEET. Groundwater Encountered at 27 FEET	
						33			
						34			
						35			

\* The blow counts have been converted to standard N-value blow counts.  
\*\* LBNL Datum

SURFACE ELEVATION: 762' \*\*  
TOTAL DEPTH: 30.0 feet  
GROUND WATER DEPTH: ▽ 27.0 feet at time of drilling  
▽ feet

LOGGED BY: R. Padgett  
EQUIPMENT: Mobile B-53  
DIAMETER of BORING: 6  
DATE DRILLED: 9-18-06



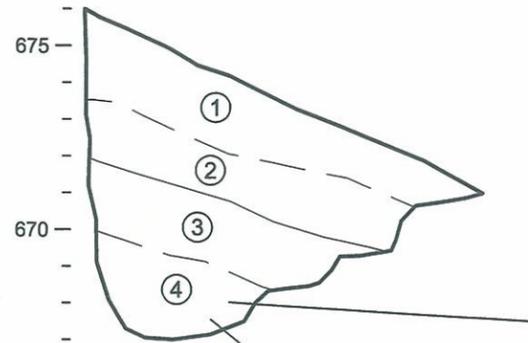
**LOG OF EXPLORATION  
BORING K-3  
CRT Building  
Lawrence Berkeley National  
Laboratory  
Berkeley, California**

PLATE  
**A-5**  
1 of 1

PROJECT NUMBER 74911 DATE Jun 2007

**TP-1**

← NORTH 57° EAST ;SOUTHEAST FACE



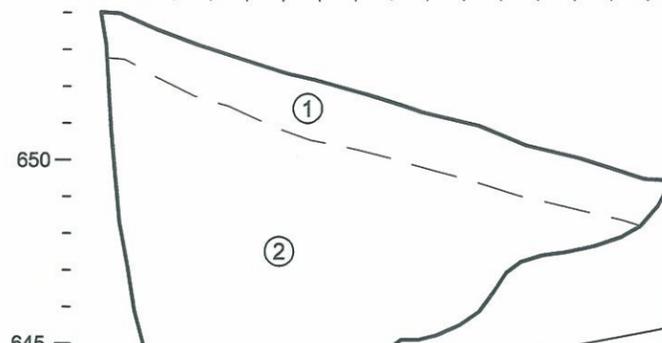
Depth (feet)	P.P. (tsf)
0.5	3.5
1.0	4.5
2.5	4.5+
3.0	4.5+

Fracture or Bedding Orientation in Unit 4	
Dip Direction	DIP
289°	64°
145°	60°
345°	76°
185°	29°

Fracture per Foot in Unit 4	
Direction	Number of Fractures
X	6
Y	7
Z	6 TO 9

**TP-2**

← NORTH 58° EAST ;SOUTHEAST FACE



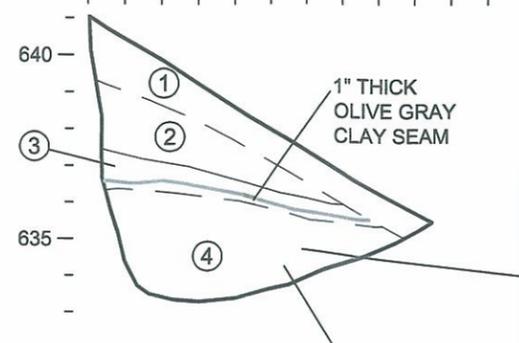
Depth (feet)	P.P. (tsf)
0.5	4.0
2.0	4.5+
4.5	4.5+
6	4.5+

Fracture or Bedding Orientation in Unit 4	
Dip Direction	DIP
206°	39°
210°	60°
41°	72°
212°	50°
40°	90°
228°	40°

Fracture per Foot in Unit 4	
Direction	Number of Fractures
X	9
Y	5
Z	8 TO 11

**TP-3**

← NORTH 64° EAST ;SOUTHEAST FACE



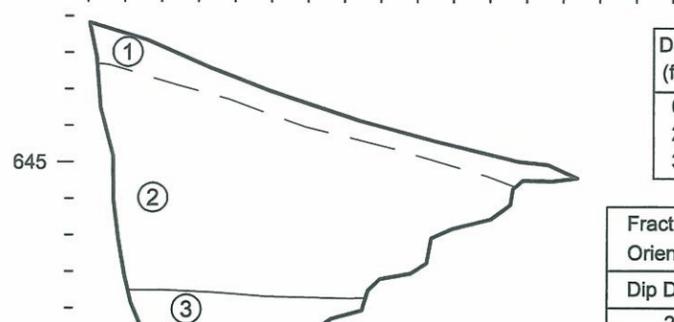
Depth (feet)	P.P. (tsf)
0.5	3.0
2.0	4.5+

Fracture or Bedding Orientation in Unit 4	
Dip Direction	DIP
282°	88°
85°	66°
302°	56°
223°	50°
250°	68°
16°	30°
249°	64°

Fracture per Foot in Unit 4	
Direction	Number of Fractures
X	6
Y	8
Z	6

**TP-4**

← NORTH 60° EAST ;SOUTHEAST FACE



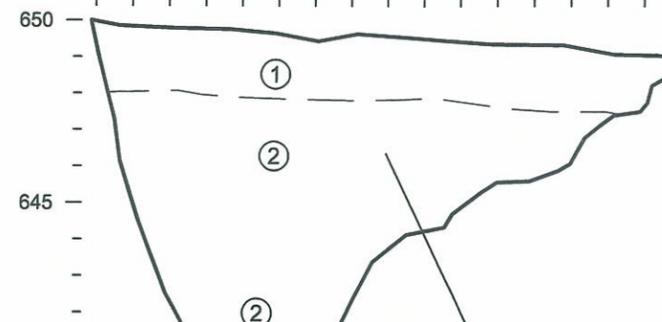
Depth (feet)	P.P. (tsf)
0.5	3.0
2.0	4.5+
3.5	4.5+

Fracture or Bedding Orientation in Unit 4	
Dip Direction	DIP
281°	80°
0°	90°
95°	6°
105°	88°
295°	28°

Fracture per Foot in Unit 4	
Direction	Number of Fractures
X	5
Y	6
Z	8

**TP-5**

← NORTH 37° WEST ;NORTHEAST FACE

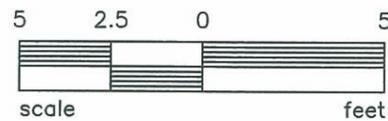


Depth (feet)	P.P. (tsf)
1.0	4.5
2.0	4.5+
4.0	4.5+
5.0	4.5+

1. SANDY SILT WITH GRAVEL (ML) - Dark brown, dry, very stiff to hard (3.0 to >4.5 tsf) sub-angular gravels up to 1-inch diameter, moderate rootlets, slightly porous.
2. GRAVELLY SANDY CLAY (CL) - Dark yellow brown, moist, hard (>4.5 tsf), sub-angular gravels up to 2.5-inch diameter (colluvium).
3. SILTSTONE - Dark, yellow brown, completely weathered, extremely weak rock, no visible bedding and relic fractures. Very close, very tight aperture - JRL 6-8
4. SILTSTONE - Dark, yellow brown, completely weathered, very weak to weak rock. Bedding close spacing. Fractures very close.

**EXPLANATION**

- Seepage
- DD Dry Density (pounds per square foot)
- MC Moisture Content (%)
- Tube Sample
- P.P. Pocket Penetrometer
- tsf Tons per square foot
- Groundwater



**KLEINFELDER**

PROJECT NO. 74911

DATE JUNE 2007

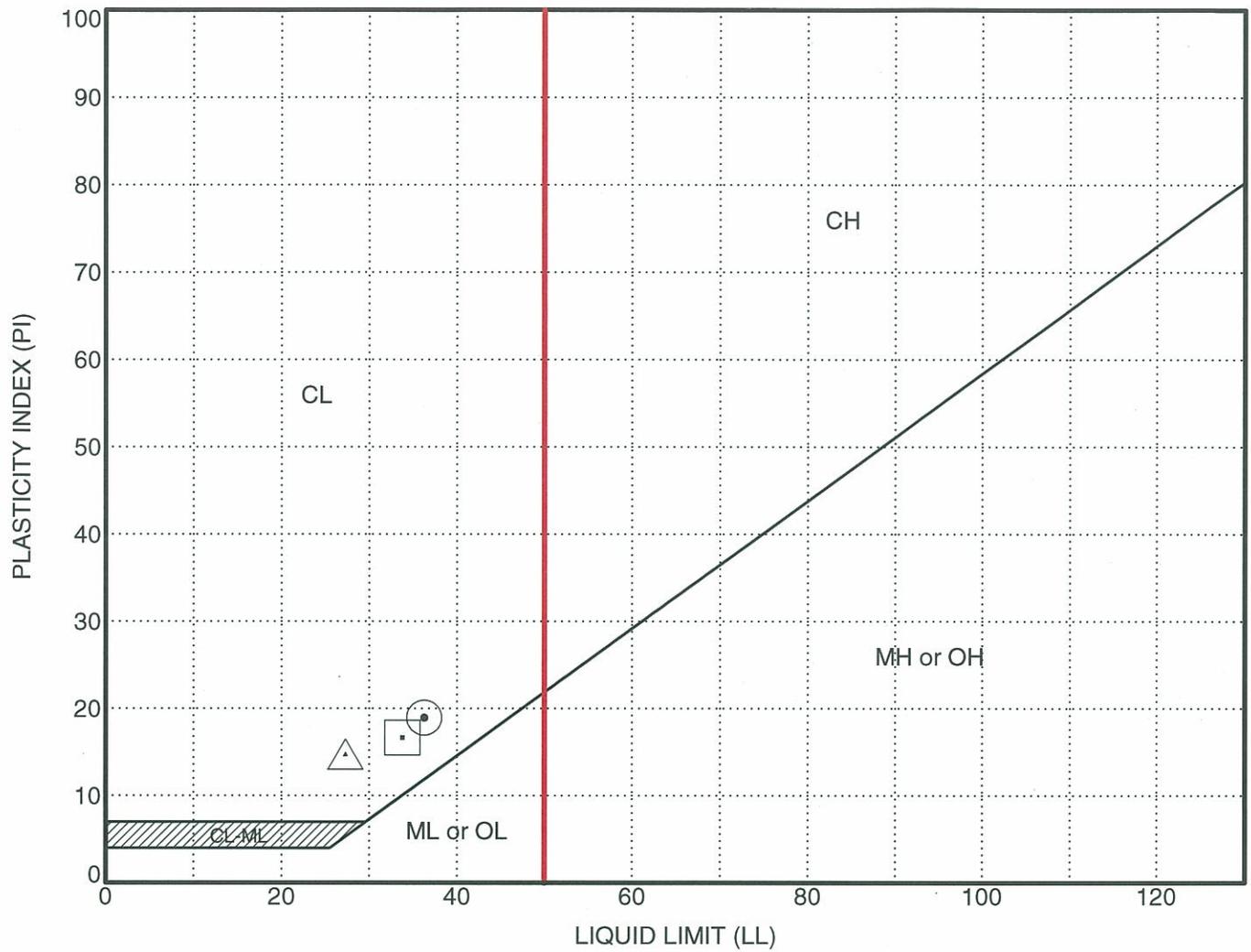
LOG OF TEST PITS TP-1 THROUGH TP-5  
 CRT BUILDING  
 LAWRENCE BERKELEY NATIONAL  
 LABORATORY  
 Berkeley, California

PLATE  
**A-6**

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# APPENDIX B

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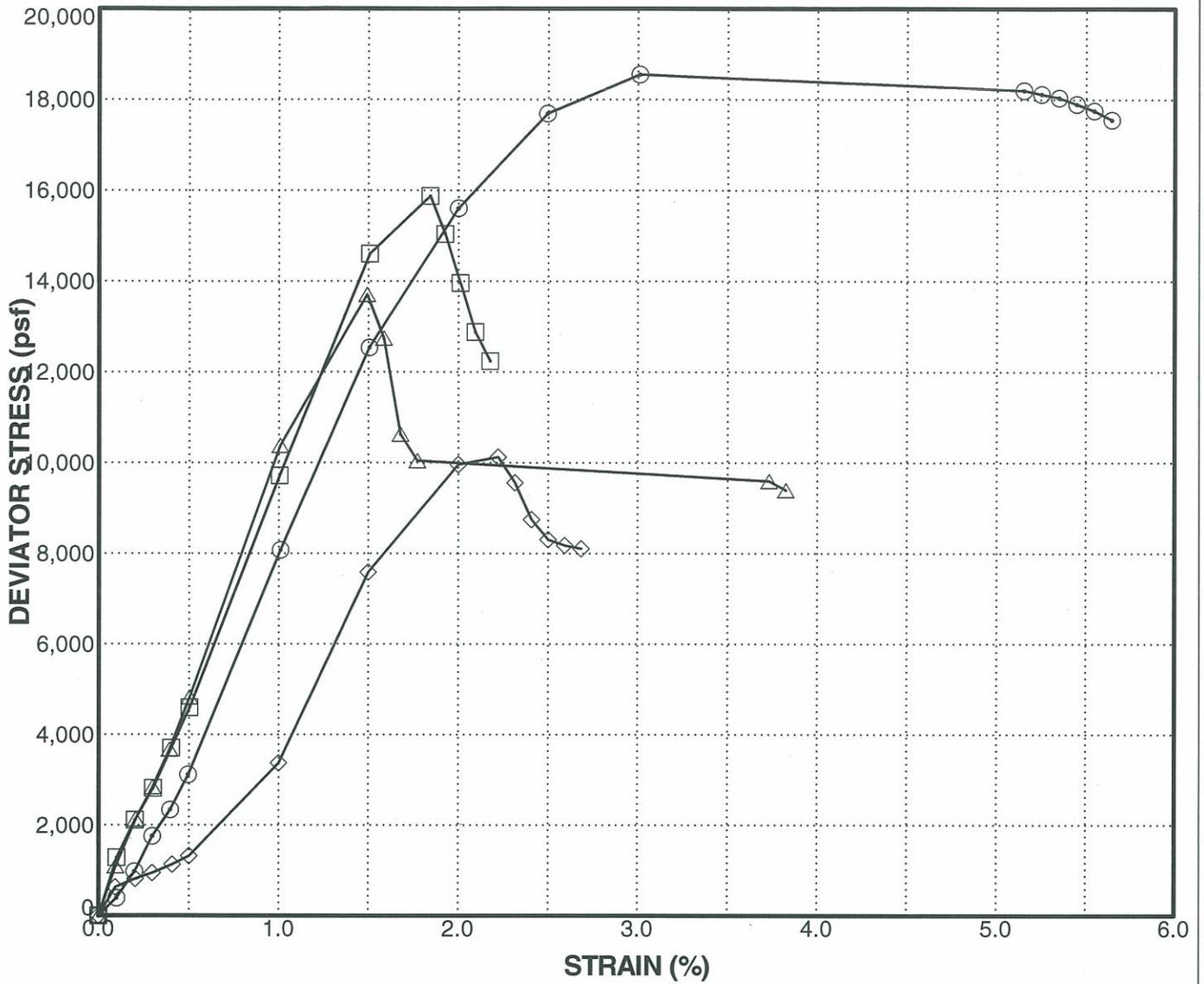


SAMPLE SOURCE	CLASSIFICATION	LIQUID LIMIT (%)	PLASTIC LIMIT (%)	PLASTICITY INDEX (%)	% PASSING #200 SIEVE
⊙ K-1 @ 10.0'	Siltstone	36	17	19	
□ K-3 @ 14.5'	Siltstone	34	17	17	
△ K-3 @ 28.0' - 29.5'	Shale	27	13	14	



**PLASTICITY CHART**  
 CRT Building  
 Lawrence Berkeley National  
 Laboratory  
 Berkeley, California

PLATE  
**B-1**



Sample Source	Classification	Type of Test	Confinement Pressure (psf)	Shear Strength (psf)	Strain (%)	Dry Density (pcf)	Moisture Content (%)
⊙ K-1 @ 5.0'	Gravelly Sandy Clay	TXUU	1440	9276	3	115	12.8
□ K-3 @ 3.0'	Siltstone	TXUU	374	7939	2	117	14.2
△ K-3 @ 4.5'	Siltstone	TXUU	720	6856	1	124	11.6
◇ K-3 @ 14.5'	Siltstone	TXUU	2592	5063	2	118	13.4

UC = Unconfined Compression

TX/UU = Unconsolidated Undrained Triaxial



**STRENGTH TEST DATA**

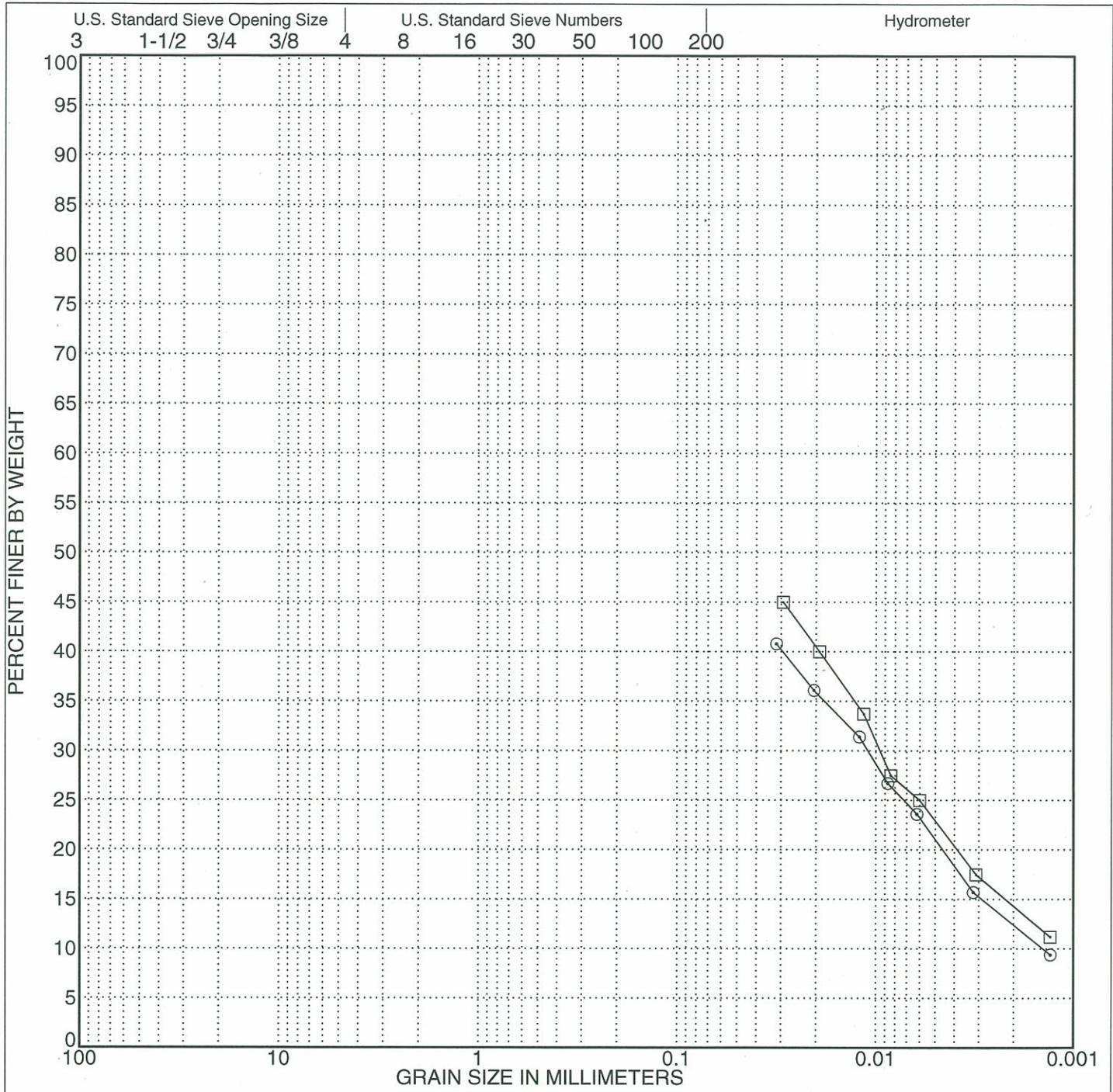
CRT Building  
Lawrence Berkeley National  
Laboratory  
Berkeley, California

PLATE

**B-2**

PROJECT NUMBER 74911

DATE Jun 2007



Cobbles	GRAVEL		SAND			SILT	CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE		

SYMBOL	SAMPLE SOURCE	CLASSIFICATION
○	K-1 @ 10.0'	Siltstone
□	K-3 @ 28.0' - 29.5'	Shale

	<b>PARTICLE SIZE ANALYSIS</b> CRT Building Lawrence Berkeley National Laboratory Berkeley, California	PLATE
		<b>B-3</b>
PROJECT NUMBER 74911	DATE Jun 2007	

---

# APPENDIX C

---



800 SFD Blvd, Ste. E  
San Anselmo, CA 94960  
(415) 453-2800 (ph.)

September 19, 2006  
Reference: 06-148-1CA

Mr. Jared Pratt  
Kleinfelder, Inc.  
2240 Northpoint Parkway  
Santa Rosa, CA 95407

Subject: Seismic Refraction Survey  
CRT Building, LBNL  
Berkeley, California

Dear Mr. Pratt:

Advanced Geological Services, Inc. (AGS) presents this letter report to Kleinfelder, Inc. describing the results of a seismic refraction (SR) survey completed by geophysicist Dan Jones and field technician Steve Ward on August 31, 2006 at the subject site in Berkeley, California. Background information and site orientation were provided by Kleinfelder representatives.

## **SITE DESCRIPTION**

The project site is located on a sloping hillside between Cyclotron Road and Building 70A at Lawrence Berkeley National Laboratory (LBNL). In general, the grassy ground surface is steeply sloping to the south in this area, with scattered eucalyptus trees. There is a sidewalk crossing the hillside toward the north-central portion of the site. A site map showing the seismic refraction survey location, overlain on a topographic map provided by Kleinfelder, is included as Figure 1. Locations of geotechnical borings and test pits were unavailable at the time this map was completed. Test pit information provided by Kleinfelder indicates approximately two feet of topsoil overlying colluvium, that in turn overlies an undifferentiated Cretaceous mixed sandstone-siltstone bedrock.

## **PURPOSE**

Site information indicates that a new building is planned for the project site, with the approximate footprint as shown on Figure 1. It is AGS's understanding that this installation may involve maximum cuts to depths of 30 to 40 feet bgs. The purpose of the seismic refraction survey was to measure the (compressional or p-wave) velocity of subsurface seismic layers. AGS understands that this information will be used in conjunction with the boring and test pit results to evaluate the subsurface conditions, specifically the excavation characteristics, or rippability, of bedrock for pre-construction planning.

## **SEISMIC REFRACTION (SR) METHOD**

The seismic refraction method provides information regarding the seismic velocity structure of the subsurface. The method is based upon the generation and propagation of an elastic wave (compressional P-wave) into the subsurface. The P-wave propagates through the ground and is refracted along interfaces that mark an increase in velocity. Part of the P-wave energy is refracted back to the ground surface and is subsequently monitored by a series of co-linear, vibration-sensitive devices called geophones that are placed at the ground surface. The resulting seismic waveforms are recorded on a seismograph and analyzed to determine the depth and velocities of subsurface seismic layers.

The physical properties of earth materials (fill, sediment, rock) such as compaction, density, hardness, and induration dictate the corresponding seismic velocity of the material. Additionally, other factors such as bedding, fracturing, weathering, and saturation can also affect seismic velocity. In general, low velocities are typically indicative of loose soil, poorly compacted fill material, poorly to semi-consolidated sediments, deeply weathered, and highly fractured rock. Conversely, high velocities are indicative of competent rock or dense and highly compacted sediments and fill. The highest velocities are measured in unweathered and little fractured rock.

There are certain limitations associated with the SR method as applied for this investigation. These limitations are primarily based on assumptions that are made by the data analysis routine. The data analysis routine assumes that the velocity of subsurface materials typically increase with depth. Therefore, if a layer exhibits velocities that are slower than those of the material above it, the slower layer may not be resolved. Also, a velocity interval may simply be too thin to be detected, for instance the upper weathered portion of the bedrock surface, if only a few feet thick.

The quality of the field data is critical to the construction of an accurate depth and velocity profile. Strong, clear "first-break" information from refracted interfaces will make the data processing, analysis, and interpretation much more accurate and meaningful. Vibrational noise or poor subsurface conditions can decrease the ability to accurately locate and pick seismic waves from the interfaces.

Due to these and other limitations inherent to the seismic refraction method, resultant velocity cross-sections should be considered only as approximations of the subsurface conditions. The actual conditions may vary locally. The actual conditions may vary locally and could warrant follow-up intrusive work for verification.

## **SEISMIC REFRACTION FIELD SURVEY**

AGS obtained seismic refraction data along two (2) lines as shown on the Site Map (Figure 1). These lines are labeled A and B and are oriented roughly parallel to the topographic contours. Line A is located approximately 10-35 feet west of the head of the slope. Line B occurs further downslope by 85 to 100 feet. The approximate positions of these lines were chosen by

Kleinfelder to provide information in the desired locations and adjusted in the field based on accessibility. The approximate final position of each line was recorded on a topographic map supplied by Kleinfelder. The lines were each 225 feet in length (from end-shot to end-shot) to provide the desired depth of investigation (30-40 feet bgs). They were each comprised of one seismic spread consisting of 24 geophones and five shot points. The geophones were coupled to the ground surface in collinear arrays at 9 foot intervals. Shot points were located at the each end of each line and then every 54 to 58.5 feet along the lines. AGS used the supplied topographic map and some on-site hand-leveling to assign elevations to each geophone and shotpoint.

AGS produced P-waves through multiple impacts with a 16-lb sledge hammer against a metal plate placed on the ground surface. An accelerometer-switch attached to the hammer transmits a triggering pulse to the seismograph each time it strikes the plate. AGS detected the P-waves produced by the hammer impacts using 10-Hz high output geophones. The detected seismic signals were digitized and recorded using a DAQ Link II Seismograph. The data were recorded on an internal hard drive for later analysis.

## DATA ANALYSIS

AGS downloaded the seismic data to a computer and processed it using the program SeisOpt Picker by *Optim, Inc.* to determine the shot point to geophone travel times. These travel times represent the first arrival of the P-wave energy to each geophone along the 24 channel spread. For each line, all first arrivals were determined in this fashion and combined to plot travel time versus geophone distance graphs, or "TD" graphs. These values, the travel times, and the location and relative elevation of each shot point and geophone were then entered into the computer program SeisOpt@2D (also by *Optim, Inc.*). The computer program uses a tomographic method, "discretizing" the subsurface below the line into grid cells of an appropriate size and assigns a velocity to each cell. Forward modeling is performed, generating test velocity models, through which travel times are calculated. These calculated travel times are compared with the observed data. The program then uses the generalized simulated annealing technique to perform a nonlinear optimization. It iteratively adjusts the velocities of each cell, recalculates the travel times, and reduces the error between the calculated and observed travel times. The optimal solution is the velocity model with the minimum acceptable travel-time error.

The final results of the seismic velocity tomographic modeling are presented as velocity cross sections of the subsurface. Each cross section contains a grid of seismic velocities as a function of horizontal distance and depth (or elevation). The grid is color contoured using SURFER (by *Golden Software*) to show an areal distribution of seismic velocity along each line, as opposed to a simple layered model. The same velocity contour scale is used for each line to allow for side-by-side comparison of the profiles.

The tomographic method, unlike traditional refraction processing techniques that employ either the generalized reciprocal method (GRM) or variations of the time-delay method, images velocity gradients in the subsurface. As is often the case in reality, the tomographic method reveals subsurface velocities as gradients and not solid layers. An appropriate gradient is

introduced between horizons defined by discrete velocities. The interface between layers is the depth at which the gradient change is the steepest. In general, this will be shallower than the actual depth at which the layer velocity is encountered in the resulting velocity contour cross-section. For example, the interface between layers that have average velocities of 500 ft/s and 2000 ft/s, will not be at the 2000 ft/s contour line, but rather shallower where velocities are about 1,250-1,500 ft/s (i.e. typically where the contours are most closely spaced). Aside from increased resolution of velocities with depth, an additional benefit of the tomographic method includes the ability to incorporate lateral changes in seismic velocities across a bedrock refractor.

## **RESULTS AND DISCUSSION**

The results of the seismic refraction survey along Lines A and B are represented by the seismic velocity cross-sectional profiles shown on Figures 2 and 3, respectively. The velocity cross-sectional profiles are presented as contoured results, with south to the left in each figure. This presentation is meant to show increased resolution of the velocities along each line, as opposed to a standard two or three layer earth model. Small contour closures, or very erratic contour lines along the edges of the cross sections may be attributable to processing artifacts. The overall cross-sections from this type of presentation, however, may be more representative of true subsurface conditions, particularly in the near-surface where gradational weathering may be experienced.

The cross-sections on Figures 2 and 3 indicate subsurface velocities ranging from 1,000 to 7,000 feet per second (ft/s). The estimated depth of investigation is approximated at 20 to 35 feet for Line A and 40 to 50 feet for Line B. The shallower depth of investigation for Line A may be attributable to refractor geometry or other unknown subsurface conditions toward the top of the slope. Along Line A, at depths below ground surface greater than 10-to-15 feet, the velocity ranges from 3,000 to 3,800 feet per second. Along Line B, higher velocity material is observed at depth, ranging up to 7,000 feet per second. The maximum velocity observed within forty feet of the ground surface (proposed maximum cut depth) along Line B is approximately 5,800 feet per second. The highest velocity material along Line B appears to be slightly deeper toward the middle of the profile than at the north and south ends of the line.

## **EXCAVATION CHARACTERISTICS (RIPPABILITY)**

Seismic velocity charts relating seismic velocity and excavation characteristics have been developed from field tests by others. These charts list the seismic velocity of various types of bedrock materials and their relative ease of excavation using different types of rippers. Caterpillar Tractor Company publishes a performance manual that lists ripper performance charts for various size tractors and types of rippers. A review of a ripper performance chart from the Caterpillar Performance Handbook (October, 1997) indicates that with a D8R, sedimentary rock is rippable up to 6,300 ft/s and marginally rippable to 8,500 ft/s. Similarly, with a D11R, sedimentary rock is rippable up to 9,700 ft/s and marginally rippable to 12,000 ft/s.

This information should only be used as a general guide, however, as many other factors should also be considered. These factors include the rock jointing and fracture patterns, the experience of the equipment operator, and the equipment and excavation methods selected. This

information should be combined with a complete and thorough analysis of test pit information, the geotechnical boring data, as well as local ripping experience (if available) to make a final assessment.

## DATA QUALITY AND ADDITIONAL DISCUSSION

Overall, the seismic refraction data quality for this project was good. This estimation was determined based on the strength and relative obviousness of the refraction arrivals from the multiple velocity interfaces. AGS made a considerable effort to increase the signal-to-noise ratio where the data appeared of lesser quality by stacking additional impacts of the sledgehammer beyond what was necessary under normal conditions. Frequency filters were applied to the collected seismic data to attempt to minimize the effects of the observed noise.

## CLOSING

All geophysical data and field notes collected as a part of this investigation will be archived at the AGS office. The data collection and interpretation methods used in this investigation are consistent with standard practices applied to similar geophysical investigations. The correlation of geophysical responses with probable subsurface features is based on the past results of similar surveys although it is possible that some variation could exist at this site. Due to the nature of geophysical data, no guarantees can be made or implied regarding the targets identified or the presence or absence of additional objects or targets.

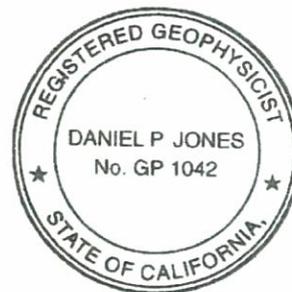
It was a pleasure working with you on this project and we look forward to being able to provide you with geophysical services in the future.

Respectfully,

Advanced Geological Services, Inc.

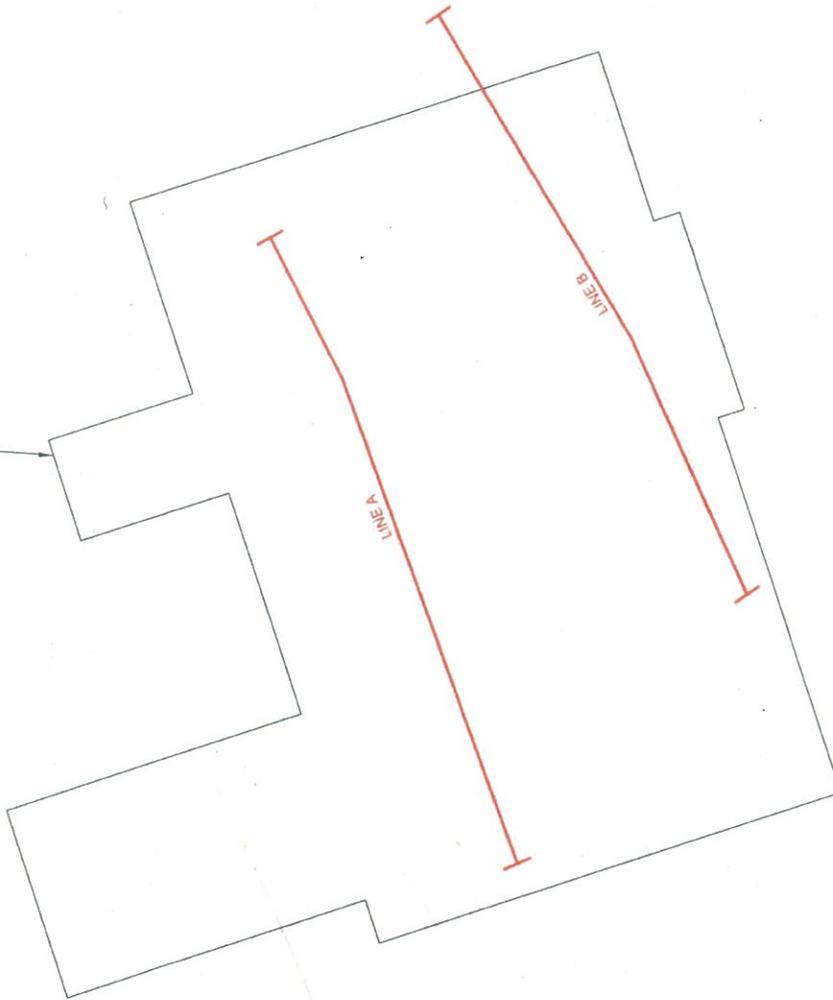


Daniel P. Jones  
Senior Geophysicist, PGp



Enclosure:    Figures 1:    Seismic Refraction Site Map  
                  Figures 2-3:    Seismic Refraction Profiles

APPROXIMATE PROPOSED  
BUILDING FOOTPRINT



SCALE  
0 20 40 80  
(1 inch = 40 feet)

LEGEND	
	SEISMIC REFRACTION (SR) LINE



SEISMIC REFRACTION SURVEY  
SITE MAP  
CRT BUILDING - LBNL

LOCATION: BERKELEY, CALIFORNIA

CLIENT: KLEINFELDER, INC.

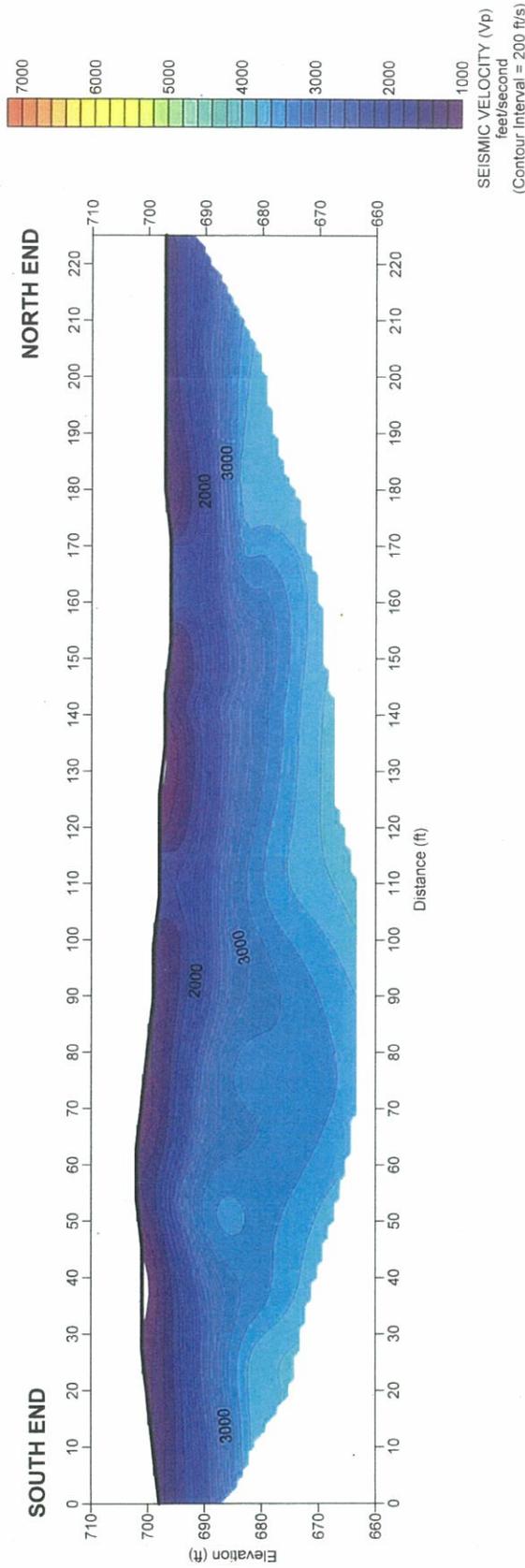
ADVANCED GEOLOGICAL SERVICES, INC.

DRAWN BY: D. JONES APPROVED BY: D. JONES

PROJECT # 06-148-1CA  
DATE: SEPTEMBER 2006

FIGURE  
1

# SEISMIC REFRACTION PROFILE - LINE A



**NOTE:**

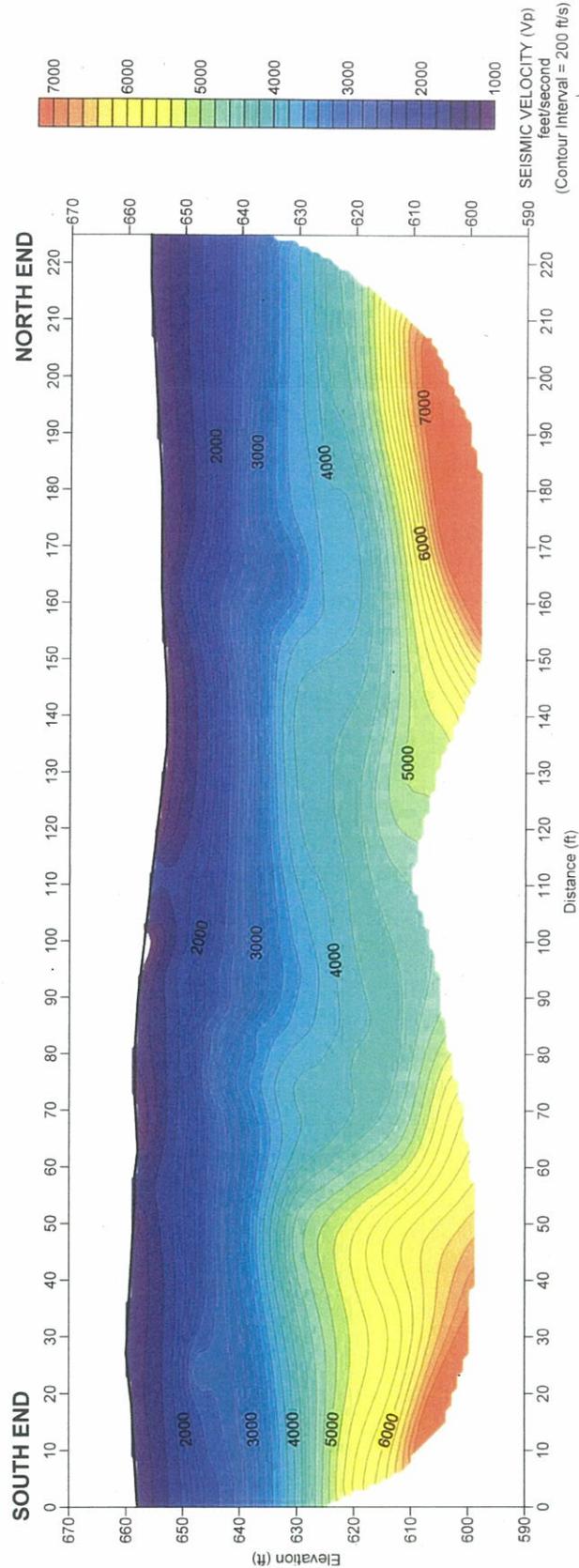
- 1) Data were acquired with a 24 Channel DAQ Link II Seismograph coupled with 10 Hz geophones at 9 feet intervals. Data collection involved performing five shotpoints per line spaced every 54-58.5 feet.
- 2) Surface elevations are approximate.
- 3) Data were processed with SeisOpt@2D refraction interpretation software by Optim, Inc. Due to the limitations inherent to the seismic refraction method the velocity cross-section, although very detailed, should be considered an approximation of subsurface conditions.



**SEISMIC PROFILE - LINE A**  
**CRT BUILDING - LBNI**

LOCATION: BERKELEY, CALIFORNIA	FIGURE
CLIENT: KLEINFELDER, INC.	<b>2</b>
PROJECT #	
DATE: SEPTEMBER 2008	
DRAWN BY: D. JONES	
APPROVED BY: D. JONES	

# SEISMIC REFRACTION PROFILE - LINE B



**NOTE:**  
 1) Data were acquired with a 24 Channel DAQ Link II Seismograph coupled with 10 Hz geophones at 9 feet intervals. Data collection involved performing five shotpoints per line spaced every 54-58.5 feet.  
 2) Surface elevations are approximate.  
 3) Data were processed with SeisOpt@2D refraction interpretation software by Optim, Inc. Due to the limitations inherent to the seismic refraction method the velocity cross-section, although very detailed, should be considered an approximation of subsurface conditions.

	<b>SEISMIC PROFILE - LINE B</b>	
	<b>CRT BUILDING - LBNI</b>	
LOCATION: BERKELEY, CALIFORNIA		
CLIENT: KLEINFELDER, INC.		
PROJECT # 06-146-1CA	ADVANCED GEOLOGICAL SERVICES, INC.	FIGURE
DATE: SEPTEMBER 2008	DRAWN BY: D. JONES	3

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# APPENDIX D

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intersection is less than the friction angle, the factor of safety is greater than 1.0. In either case, the structure will not daylight the slope if the dip or plunge is greater than the dip of the slope face.

We assumed friction angles of 30 degrees based on the detailed geomechanical information that we collected in the field, experience with similar rock types, and guidance from the Rock Slopes Reference Manual (FHWA, 1998). Vertical cut slopes are planned for the back wall and sidewall of the excavation. By inspection of the stereonets, we concluded that the proposed slope inclinations have the potential for wedge-, planar-, and toppling-type failures from the back wall and sidewall (Plate D-2).

### *Geomechanical Classifications*

For the second part of our analysis, we completed geomechanical rock mass classifications. These classifications are accomplished using the field data and therefore are more of a design tool than actual field data collection. Two of the more widely accepted classifications systems are the Rock Mass Rating System (RMR) by Bieniawski (1989) and the Geological Strength Index (GSI) from Hoek (1997).

The RMR, also referred to as the geomechanics classification system, is based on the algebraic sum of six rock mass property ratings, namely:

- Strength of intact rock material (uniaxial compressive strength)
- Rock quality designation (RQD)
- Spacing of discontinuities
- Condition of discontinuities
- Groundwater conditions
- Orientation of discontinuities relative to the excavation or rock slope

To estimate the RMR, we compared field data to published tables by Bieniawski (1989). Values for RMR can range from 0 to 100. From the ratings, rock class and corresponding descriptions and engineering properties are assigned to the overall rock mass. Bieniawski's RMR classification can be related to Hoek's (1997) Geological Strength Index (GSI). If the 1989 version of Bieniawski's RMR classification is used, the  $GSI = RMR_{89} - 5$  where  $RMR_{89}$  has the "Groundwater" rating set to 15 and "Adjustment for Joint Orientation" set to zero. For



example, assume a RMR is calculated to be 30 with the groundwater rating set to 15 and the joint orientation adjustment set to -5 for favorable joint orientations. Correcting the RMR as stated above, it would equal 35 (groundwater set to 15 and joint orientation adjustment set to zero). The calculated GSI would then be calculated as  $RMR - 5$ , or 30. The GSI rating can also be estimated directly from the information that we collected during our field mapping.

The rock mass information collected at site indicates a siltstone with rock strength of 0.25 to 1 MPa (36 to 145 psi), or extremely weak rock. The RQD was estimated to be 5, which indicates a highly fractured rock mass. The RMR and GSI were estimated to be approximately 40 +/- 5, which indicated a very blocky to disturbed, fair quality rock mass with fair quality, moderately weathered discontinuity surfaces.

### ***Slope Stability and Wall Pressure***

For the third part of our analysis, we completed slope stability analyses for the potential wedge and planar-type failures along with a global stability analysis for failure through the rock mass. We utilized the computer programs, Rocplane V. 2.0 and Swedge V.4.0 by Rocscience® to complete the stability analyses on the potential wedge and planar-type failures. We assumed cohesion of zero and a friction angle of 30 degrees. We assumed a slope height of 43 feet based on the cross-section of the excavation. Plates D-3 and D-4 display the wedge and planar-type failures that are possible from the back wall and sidewall.

The geometry of each potential failure outlined in Plates D-3 and D-4 were analyzed using the computer programs discussed above to estimate the factor of safety. If the factor of safety was less than 1.0, we estimated a wall pressure to achieve a factor of safety greater than 1.0. Table 1 summarizes the failure geometries and the required wall pressures. Additionally, we estimated an equivalent earth pressure distribution for the design of a retention system during construction based on the "Plane 1" block geometry of the cut slope back wall listed below. Based on our calculations, the equivalent fluid pressure (safety factor of 1.0) is 50H (PCF).

**Table 1. Summary of Potential Wedge and Planar Type Failures**

<b>Failure Geometry</b>	<b>Factor of Safety</b>	<b>Pressure Required for Factor of Safety &gt;1.0 – rectangular distribution (tsf)</b>	<b>Width of Wedge on Upper Slope Surface (ft)</b>
<b>Cut Slope Back Wall</b>			
Plane 1	0.4	0.50	29
Wedge 1	0.3	0.25	16
Wedge 2	1.1	NA	21
Wedge 3	1.0	NA	30
Wedge 4	0.4	0.35	29
Wedge 5	1.5	NA	45
Wedge 6	0.4	0.35	29
Wedge 7	0.2	0.22	11
<b>Cut Slope Sidewall</b>			
Plane 1	0.2	0.50	12
Wedge 1	2.0	NA	35
Wedge 2	0.3	0.25	14
Wedge 3	0.2	0.25	11
Wedge 4	0.2	0.19	8
Wedge 5	1.7	NA	18

We utilized the computer program Slide V. 5.0 by Rocscience® to complete the global stability analysis of the proposed cut slope. The proposed slope is stepped with a 43-foot high upper wall and a 30-foot high lower wall separated by a 45-foot wide horizontal bench. We estimated the Hoek-Brown strengths using the geomechanical data that we collected in the field. The following list presents specific design data for the cut slope.

- Siltstone
  - Rock Strength = 0.7 MPa (102 psi)
  - GSI = 40



Figure 5 displays the stability analysis of the proposed slope and the estimated wall pressure required for a factor of safety greater than 1.0. The model indicates a rectangular pressure distribution over the height of the upper cut slope (43 feet) of 1,200 psf. We also estimated the pressure based on a standard triangular pressure distribution. Based on our calculations, the equivalent fluid pressure is 50H (PCF).

**Foundation Bearing Pressure**

The bearing pressure of footings on rock depends on the presence of discontinuities, weathering and the quality of the rock mass. The Caltrans Bridge Design Specifications dated April 2000 provides presumptive bearing pressure values for preliminary design of simple structures on good quality rock masses (Caltrans Table 4.11-4.1.4-1). In his book, Foundations on Rock (1999), Dr. Duncan Wylie provides estimates for bearing pressures based on rock strength and rock quality designation (RQD). Dr. Wylie states that bearing pressure estimates should be reduced for fractured rock based on the RQD. He states that for a RQD less than 50, the bearing pressure estimate should be reduced by a factor of 0.25 to 0.1. We estimated a RQD of 5 for the siltstone. Table 2 summarizes the pressure ranges based on Caltrans and the RQD adjusted ranges possible for the siltstone present at the site. We assumed a bearing pressure adjustment of 0.25 for the RQD.

**Table 2. Summary of Allowable Bearing Pressures (Based on Caltrans, 2000)**

Type of Material	Consistency in Place	Ordinary Range (tsf)	RQD Adjusted Range (tsf)	Recommended Value (tsf)
Sedimentary Rock: Siltstone	Hard Sound Rock	15 to 25	3.75 to 6.25	5
Weathered or Broken Bedrock	Medium Hard Rock	8 to 12	2 to 3	2.5

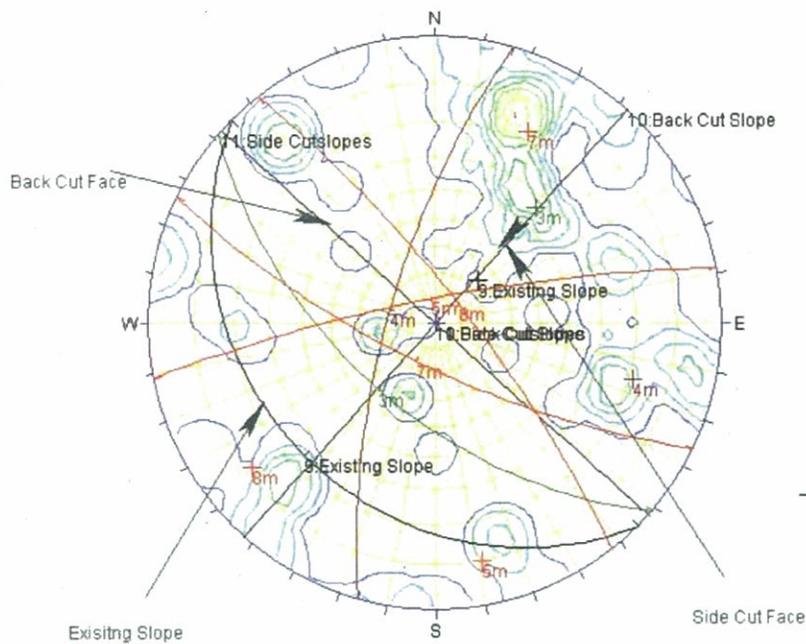
We also calculated the ultimate bearing pressure of the rock using methods in Wylie, 1999 based on the Hoek-Brown Failure Criterion for the rock mass. We estimated an ultimate bearing capacity ranging from 1.5 to 3 tsf.



Additionally, we completed an estimate of the bearing capacity based on bearing capacity theory (for soil). We assumed a friction angle of 35 degrees and cohesion of zero. Based on our calculations, the ultimate bearing capacity is approximately 12.0 tsf.

It appears reasonable to use a value of 10 tsf for the ultimate rock bearing capacity. The ultimate bearing capacity in sheared rock should be limited to 2.5 tsf.

One important consideration is that the values above are for "flat" ground where the distance of the footing from the top of a cut slope is greater than six times the width of the footing. If the distance is less, the ultimate bearing capacity would be lowered based on the slope geometry and distance. Alternatively, drilled piers can be used to support these foundations.



Orientations	
ID	Dip / Direction
9	24 / 225
10	90 / 222
11	90 / 132
3 m	56 / 221
4 m	71 / 286
5 m	80 / 349
7 m	73 / 206
8 m	78 / 052

Equal Angle  
Lower Hemisphere  
62 Poles  
62 Entries



PROJECT NO. 74911

September 2006

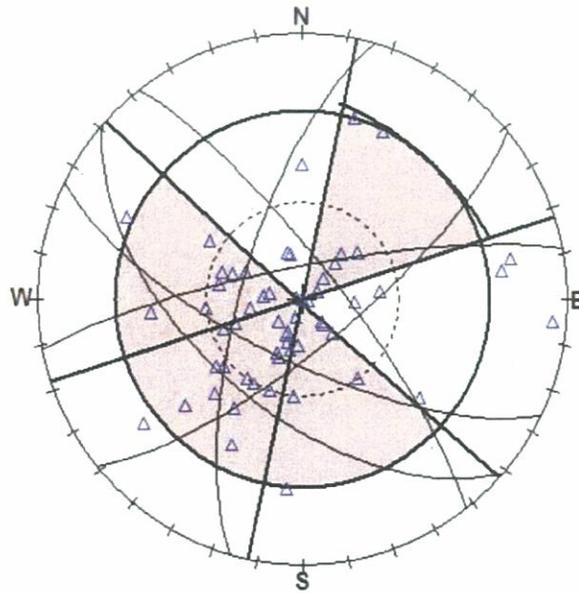
### Stereonet

CRT Building  
Lawrence Berkeley National Lab  
Berkeley, California

PLATE

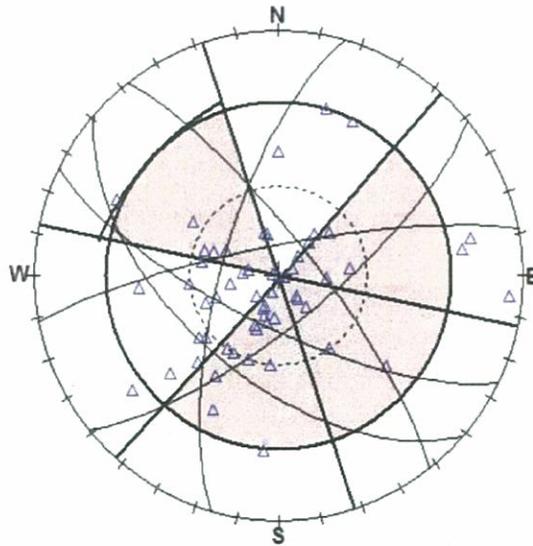
**D-1**

Backwall Cut Slope



Dip =  $90^{\circ}$   
Direction =  $222^{\circ}$   
Friction =  $30^{\circ}$

Sidewall Cut Slope



Dip =  $90^{\circ}$   
Direction =  $132^{\circ}$   
Friction =  $30^{\circ}$



**KLEINFELDER**

PROJECT NO. 74991

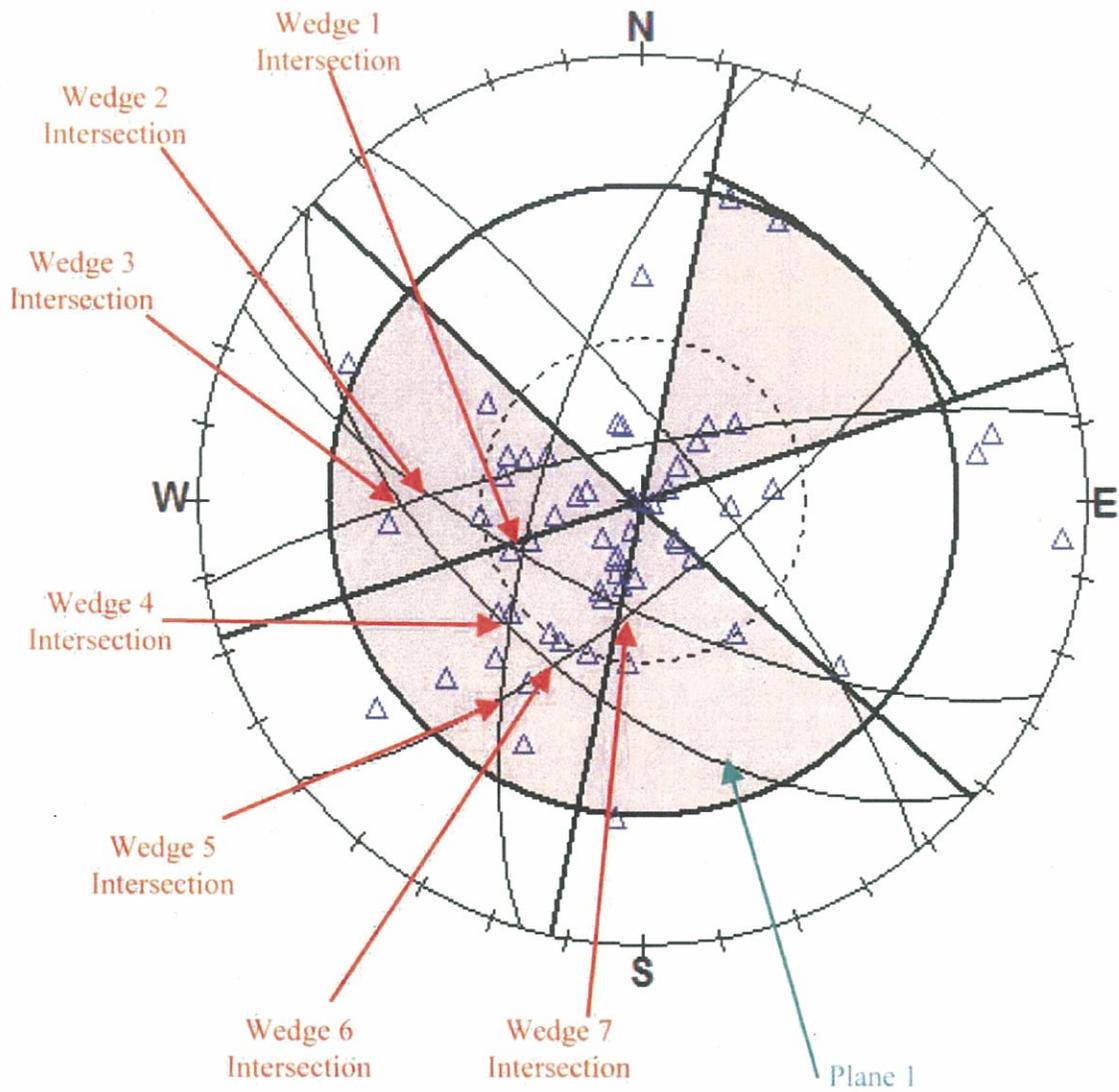
September 2006

**Markland Analyses**

CRT Building  
Lawrence Berkeley National Lab  
Berkeley, California

**PLATE**

**D-2**



**KLEINFELDER**

PROJECT NO. 74911

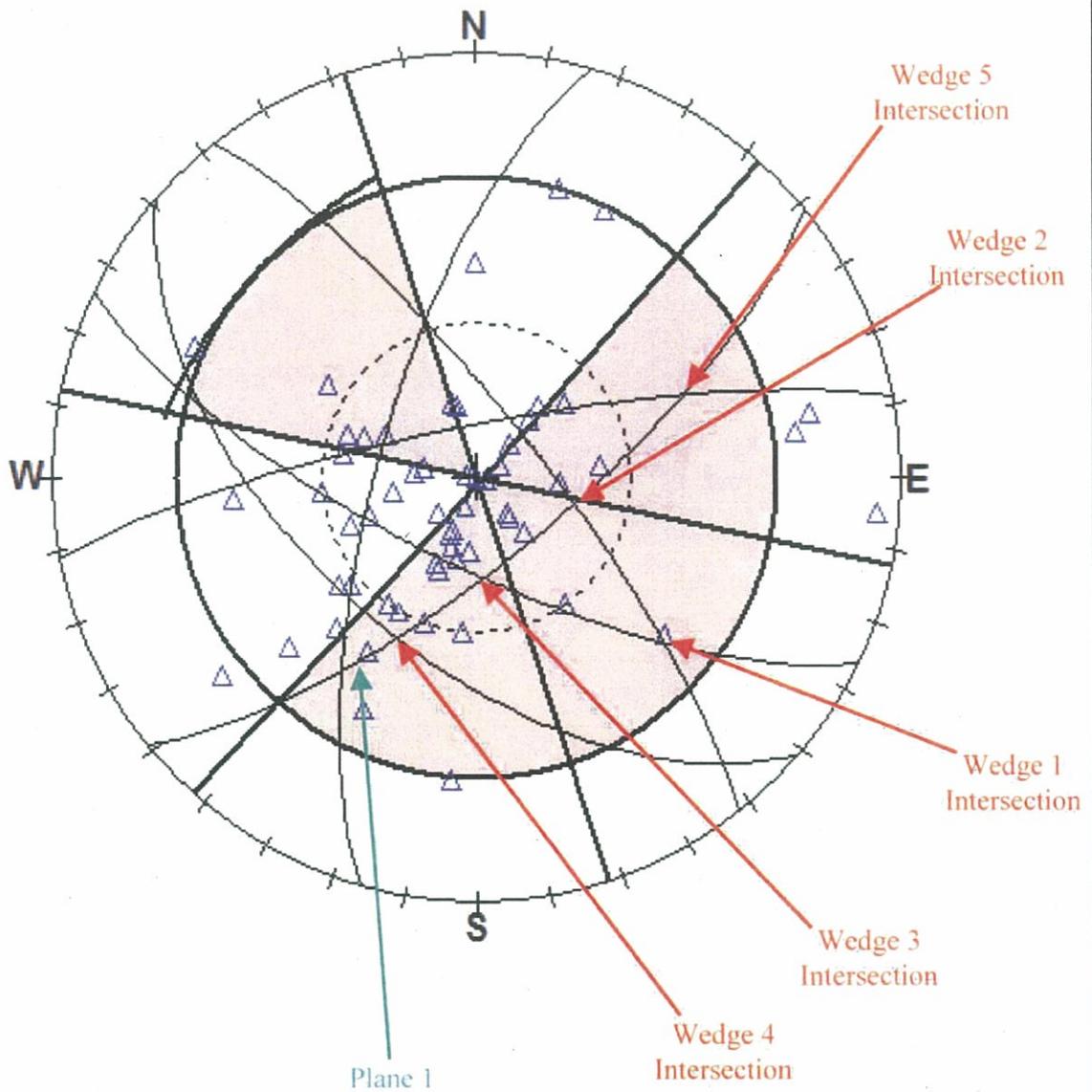
September 2006

**Back Wall Failure Potential**

CRT Building  
Lawrence Berkeley National Lab  
Berkeley, California

**PLATE**

**D-3**



**KLEINFELDER**

PROJECT NO. 74911

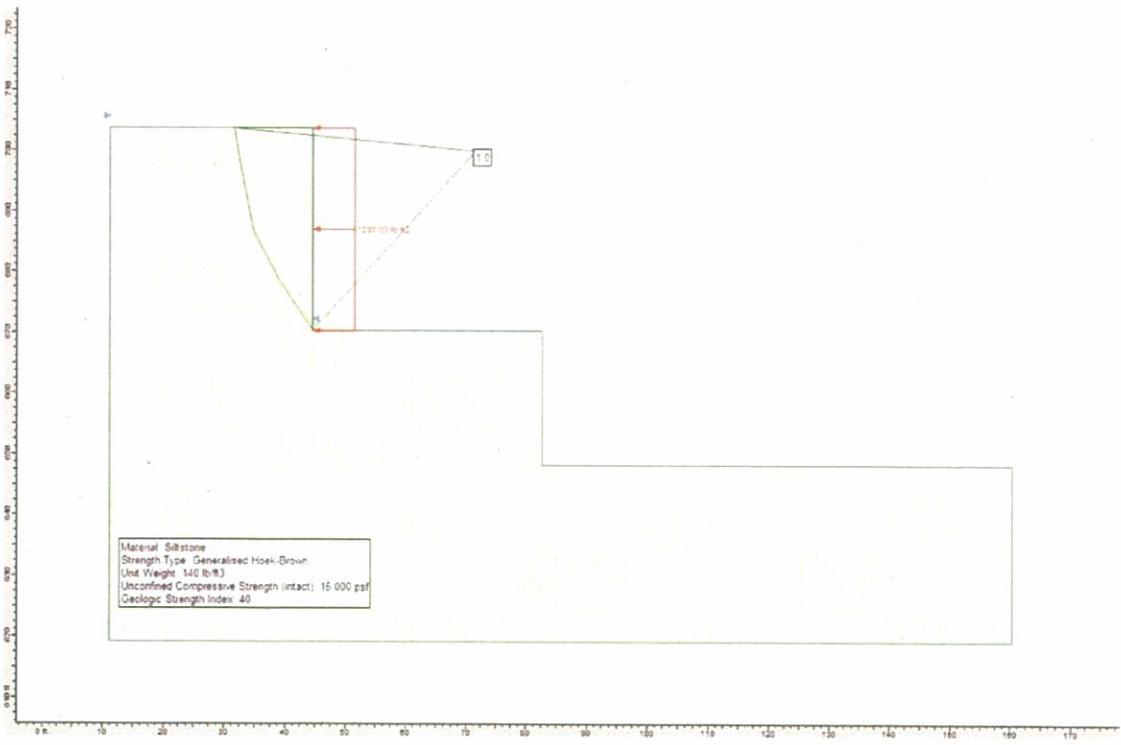
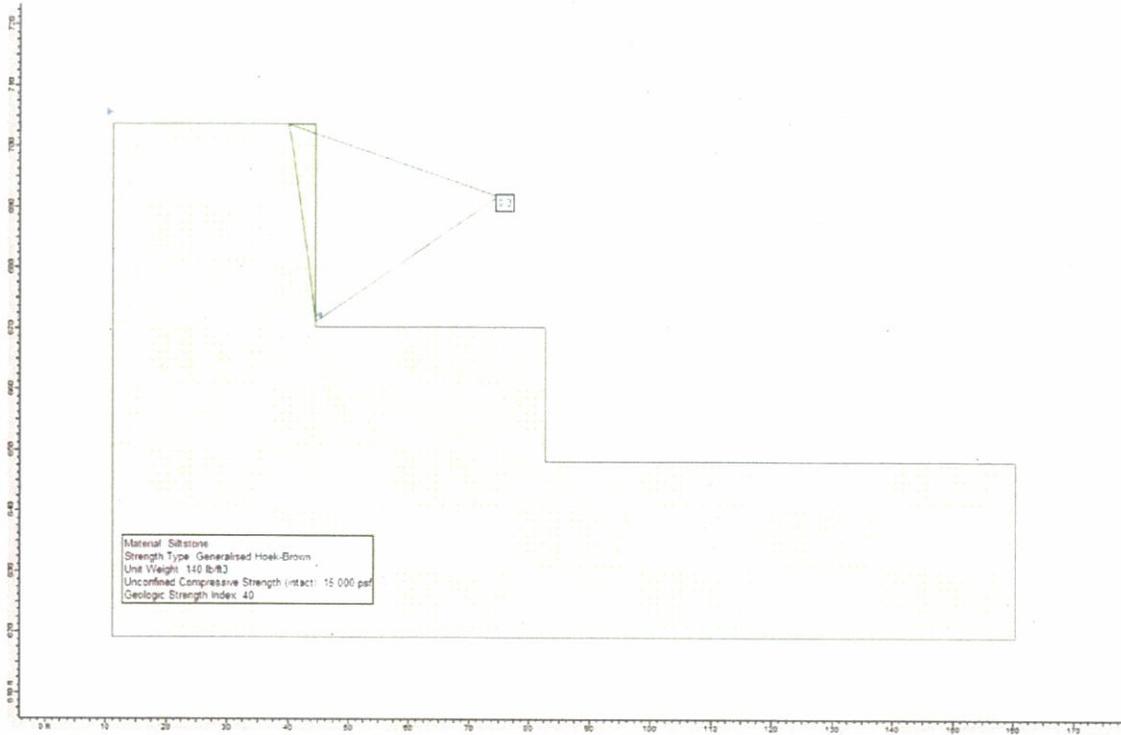
September 2006

**Sidewall Failure Potential**

CRT Building  
Lawrence Berkeley National Lab  
Berkeley, California

**PLATE**

**D-4**



**KLEINFELDER**

PROJECT NO. 74911

September 2006

**Global Stability Analysis**

CRT Building  
 Lawrence Berkeley National Lab  
 Berkeley, California

**PLATE**

**D-5**

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# APPENDIX E

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# ETS

## Environmental Technical Services

- Soil, Water & Air Testing & Monitoring
- Analytical Labs
- Technical Support

1343 Redwood Way  
Petaluma, CA 94954

(707) 795-9605 / FAX 795-9384

**Serving people and the environment  
so that both benefit.**

COMPANY: Kleinfelder Associates, 2240 Northpoint Parkway, Santa Rosa, CA 95407	ANALYST(S) D. Salinas S. Santos	SUPERVISOR D. Jacobson LAB DIRECTOR G.S. Conrad PhD
ATTN: Mark H. Stanley	DATE of COMPLETION 10/2/2006	
JOB SITE: CRT Bldg, LBNL, Berkeley, California	DATE RECEIVED 9/22/2006	
FILE #: 74911-1		

LAB SAMPLE NUMBER	SAMPLE ID	DESCRIPTION of SOIL and/or SEDIMENT	SOIL pH -log[H+]	NOMINAL RESISTIVITY ohm-cm	ELECTRICAL CONDUCTIVITY µmhos/cm	SULFATE SO4 ppm	CHLORIDE Cl ppm
02207-1	CRT1/LBNL	K-3 @ 15'	6.45	2,440	[410]	84	40
Method	Detection	Limits →	---	1	0.1	1	1
LAB SAMPLE NUMBER	SAMPLE ID	DESCRIPTION of SOIL and/or SEDIMENT	SALINITY ECe mmhos/cm	SOLUBLE SULFIDES (S=) ppm	SOLUBLE CYANIDES (CN=) ppm	REDOX mV	PERCENT MOISTURE %
02207-1	CRT1/LBNL	K-3 @ 15'				+328.9	
Method	Detection	Limits →	-	0.1	0.1	1	0.1

COMMENTS

Resistivity is nearly 2,500 ohm-cm which is fair, but soil reaction (i.e., pH) is mildly acidic which does not help; both sulfate and chloride are quite low, and redox is very mild. The Cal Trans times to perforation for this soil are as follows; 18 ga steel the time to perforation is nearly 17 yrs, and for 12 ga it goes up to about 37 yrs. Also, the average pitting rate determination for ductile iron and mild steel in this soil material is approximately at 0.130 mm/yr, thus pitting to a depth of 2 mm would be about 15 yrs; and to a 4 mm depth it would be 30 yrs. Chloride is low enough that it should not have any significant corrosion impact on concrete steel reinforcement; and sulfates are low enough that there should not be any adverse impact on concrete, mortar, grout or cement. The redox value indicates the soil is only very mildly reduced, thus there should be no significant adverse impact here either. As concerns buried metals, this soil would benefit greatly from alkaline treatment in that pushing its pH up to the 7.5-8.5 range would increase the 18 ga time to perforation to ≈36 yrs which is more than double the native soil time. Other than alkaline treatment, to increase metals longevity any more in this soil would require further upgrading (i.e., heavier gauge or more resistant steel); and/or that other actions be taken (e.g. special engineering fill, coating steel, cathodic protection, etc.). Last, standard concrete mixes (and related materials) do not appear to be at risk in this soil based on these results.

\\NOTES: Methods are from following sources: extractions by Cal Trans protocols as per Cal Test 417 (SO4), 422 (Cl), and 532/643 (pH & resistivity); &/or by ASTM Vol. 4.08 & ASTM Vol. 11.01 (=EPA Methods of Chemical Analysis, or Standard Methods); pH - ASTM G 51; Spec. Cond. - ASTM D 1125; resistivity - ASTM G 57; redox - Pt probe/ISE; sulfate - extraction Title 22, detection ASTM D 516 (=EPA 375.4); chloride - extraction Title 22, detection ASTM D 512 (=EPA 325.3); sulfides - extraction by Title 22, and detection EPA 376.2 (=SMEWW 4500-S D); cyanides - extraction by Title 22, and detection by ASTM D 4374 (=EPA 335.2).

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# APPENDIX F

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**Kleinfelder, 2006**  
**Test Boring K-1**

REFERENCE:

Kleinfelder, Inc., *Geotechnical Evaluation, Building 50 Seismic Retrofit, Lawrence Berkeley National Laboratory, Berkeley, California (KA 75173)* bound report dated September 29, 2006

SYMBOL	ROCK TYPE	SYMBOL	ROCK TYPE	SYMBOL	ROCK TYPE
	BRECCIA		SILTSTONE		PHYLLITE
	CLAYSTONE		MUDSTONE		SANDSTONE
	CONGLOMERATE		SHALE		GREENSTONE
	GRANITE		BEDROCK		VOLCANIC

**WEATHERING**

**Fresh** - No visible sign of rock material weathering; perhaps slight discoloration on major discontinuity surfaces. **Weathering Grade I.**

**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 externally than in its fresh condition. **Weathering Grade II.**

**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 continuous framework or corestones. **Weathering Grade III.**

**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. **Weathering Grade IV.**

**Completely Weathered** - All rock material is decomposed and/or disintegrated to a soil. The original mass structure is still largely intact. **Weathering Grade V.**

**Residual Soil** - All rock material is converted to a soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported. **Weathering Grade VI.**

STRENGTH (OF INTACT ROCK PIECES)			Approx. UCS (Mpa)	Approx. UCS (psi)
Grade	Description	Field Identification		
R0	Extremely Weak Rock	Idented by thumbnail	0.25 - 1.0	50 - 150
R1	Very Weak Rock	Crumbles under firm blows with point of geological hammer	1.0 - 5.0	150 - 750
R2	Weak Rock	Can be peeled by a pocket knife, specimen can be fractured with single firm blow of geological hammer	5.0 - 25	750 - 3,500
R3	Moderately Strong Rock	Cannot be scraped or peeled with pocket knife, specimen can be fractured with single firm blow of geological hammer	25 - 50	3,500 - 7,500
R4	Strong Rock	Specimen requires more than one blow of geological hammer to fracture it	50 - 100	7,500 - 15,000
R5	Very Strong Rock	Specimen requires many blows of geological hammer to fracture it	100 - 250	15,000 - 35,000
R6	Extremely Strong Rock	Specimen can only be chipped with geological hammer	>250	>35,000

DISCONTINUITY SPACING		
	English	Metric
1. Extremely close	<1.0 in.	(<20 mm)
2. Very close	1.0 - 2.5 in.	(20 - 60 mm)
3. Close	2.5 - 8.0 in.	(60 - 200 mm)
4. Moderately	8.0 in - 2.0 ft.	(200 - 600 mm)
5. Wide	2.0 - 6.5 ft.	(600 - 2,000 mm)
6. Very wide	6.5 - 20.0 ft.	(2 - 6 m)
7. Ext. wide	>20.0 ft.	(>6 m)

APERTURE WIDTH	
Very tight	<1.0 mm
Tight	0.1 - 0.25 mm
Partly open	0.25 - 0.5 mm
Open	0.5 - 2.5 mm
Moderately wide	2.5 - 10 mm
Wide	10 mm - 1 cm
Very wide	1 - 10 cm
Extremely wide	10 - 100 cm
Cavernous	>1 m

ROCK QUALITY DESIGNATION	
RQD%	Rock Quality
90 - 100	Excellent
75 - 90	Good
50 - 75	Fair
25 - 50	Poor
0 - 25	Very Poor
RQD = $\frac{\text{Sum of Intact Pieces} > 4 \text{ inches (100 mm)}}{\text{Total Core Run Length}}$	

ROCK KEY BORING.GPJ 10/9/06



**ROCK DESCRIPTION CRITERIA**  
 Building 50 Seismic Retrofit  
 Lawrence Berkeley National Laboratory  
 Berkeley, California

PLATE

A-1

Drafted By: R. Padgett      Project No.: 75173  
 Date: 09/29/06              File Number: 75173

Date Completed: 9/25/2006 Logged By: R. Padgett  
 Boring Location: N37.87697 W122.25183  
 Driller: Pitcher Drilling Drilling Method/Size: Pitcher Barrel (2 7/8" diameter), Diamond Core (3" diameter)  
 Elev. Top of Hole: Approximately 752.6 ft. Total Depth: 61 feet Groundwater Depth: NA  
I  $\nabla$  NA  
F  $\nabla$  NA

Elevation (ft., msl)	Depth (feet)	Time/Core Run Length	Recovery		RQD (%)	Fractures per foot	Graphic Log	ROCK DESCRIPTION/NOTES	DISCONTINUITIES
			(in.)	(%)					
750							<b>AGGREGATE BASE-</b> <b>SILTSTONE-</b> olive gray & redbrown, very fine grained, slightly to moderately weathered, extremely weak, very closely fractured (Augered to 4-feet)	4.5 tsf (pocket penetrometer)	
5			31	100	50	5	<b>SILTSTONE-</b> olive gray & redbrown, very fine grained, slightly to moderately weathered, closely fractured, weak, iron and manganese staining on fracture faces	Joint, 60 degrees to axis, closely spaced, smooth, undulating (JRC 4-6), thin clay infill, iron staining on joint faces, dry  Joint, 60 degrees to axis, moderately spaced, slightly rough, undulating (JRC 8-10), dry Pitcher core advanced at 400 psi	
745			30	100	76	>10	<b>SHALE-</b> red brown & olive gray, moderately weathered, extremely weak, highly sheared, sandstone corestones up to 2.5" diameter <b>CLAYEY SILTSTONE-</b> red brown, yellow brown and gray, moderately weathered, extremely weak, moist, blocky	Shear zone, highly sheared, moist, zone extends from 6.5 to 7 feet bgs, moist  Joint, 50 degrees to axis, 2.0 tsf, moist  Joint, 60 degrees to axis  Joint, 50 degrees to axis, closely spaced, slightly rough to smooth, undulating (JRC 6-8), no infill, partly open to open, iron and manganese staining on joint faces Pitcher core advanced at 700 psi Fracture, 60 degrees to axis, extremely closely spaced, smooth, undulating (JRC 4-6), thin clay infill, dry to moist Pitcher core advanced at 700 psi Very closely spaced fractures	
10			24	80	33	>10		Joint, 60 degrees to axis, slightly rough to rough, undulating (JRC 8-10), no infill, dry	
740			24	100	33	>10			
15			24	80	72	5			
735			30	100	26	>10	<b>SHALE-</b> dark gray, moderately weathered, closely fractured, weak	Pitcher core advanced at 700 psi  Fracture, 180 degrees to axis, very closely spaced, rough, undulating (JRC 8-10), tight, iron and manganese staining on fracture faces Shear zone, extremely closely spaced, zone extends from 17 to 18 feet bgs	
20								Joint, 60 degrees to axis, smooth, undulating (JRC 2-4), no infill Pitcher core advanced advanced at 800 psi Joint, 30 degrees to axis, closely spaced, smooth, undulating (JRC 6-8), no infill, iron	

CORING LOG BORING.GPJ 10/6/06



**LOG OF BORING K-1**  
 Building 50 Seismic Retrofit  
 Lawrence Berkeley National Laboratory  
 Berkeley, California

PLATE  
1 of 3

Drafted By: R. Padgett Project No.: 75173  
 Date: 10/6/2006 File Number: 75173

**A-2**

CORING LOG BORING GPJ 10/6/06

Elevation (ft., msl)	Depth (feet)	Time/Core Run Length	Recovery		RQD (%)	Fractures per foot	Graphic Log	ROCK DESCRIPTION/NOTES	DISCONTINUITIES
			(in.)	(%)					
725	25	24	100	100	23	>10	<p><b>SHALE-</b> dark gray, moderately weathered, very closely fractured, extremely weak, pressure facets present</p>	<p>staining on joint surfaces</p> <p>Pitcher core practical refusal, switched to NQ Diamond core at 21 feet bgs Core rate 4 min/ft</p> <p>Fracture, 20 and 50 degrees to axis, very closely spaced, smooth to slickenside (JRC 2-4), tight, clay infill, iron and manganese staining Core rate 4 min/ft Core rate 4 min/ft</p>	
720	30	60	100	100	48	>10	<p><b>SHALE-</b> dark gray, slightly weathered, very weak to weak, closely fractured</p>	<p>Core rate 3 min/ft</p> <p>Joint, 60 degrees to axis, closely spaced, slightly rough to smooth, undulating (JRC 8-10), tight, no infill, iron staining on joint faces Core rate 3 min/ft</p> <p>Core rate 4 min/ft at 400 psi</p> <p>Joint, 60 degrees to axis, closely spaced, rough to very rough (JRC 18-20) Core rate 3 min/ft</p>	
715	35	60	100	100	63	>10	<p><b>SHALE-</b> dark gray, moderately to highly weathered, extremely weak, closely fractured</p>	<p>Core rate 4 min/ft</p> <p>Core rate 3 min/ft</p> <p>Core rate 4 min/ft</p> <p>Joint, 60 degrees to axis, smooth (JRC 2-4), no infill</p> <p>Shear zone, 30 to 50 degrees to axis, extremely closely spaced, smooth to slickensides (JRC 2-4), thin clay infill, iron staining and pressure facets on shears, shear zone extends from 31 to 36 feet bgs Core rate 3 min/ft Core rate 4 min/ft Core rate 4 min/ft</p>	
710	40	60	100	100	21	>10	<p><b>SHALE-</b> dark gray, highly weathered, extremely weak, extremely closely sheared</p>	<p>Core rate 4 min/ft</p> <p>Core rate 4 min/ft</p> <p>Joint, 50 degrees to axis, closely spaced, smooth (JRC 2-4)</p> <p>Core rate 4 min/ft</p> <p>Core rate 5 min/ft</p>	
		60	100	100	0	>10	<p><b>SHALE-</b> dark gray, highly weathered, extremely weak, extremely closely sheared</p>	<p>Shear Zone, extremely closely spaced, slickensides, wet, shear zone extends from 41 to 43 feet bgs Core rate 5 min/ft Core rate 5 min/ft</p>	



**LOG OF BORING K-1**  
Building 50 Seismic Retrofit  
Lawrence Berkeley National Laboratory  
Berkeley, California

PLATE  
2 of 3

Drafted By: R. Padgett      Project No.: 75173  
Date: 10/6/2006              File Number: 75173

**A-2**

CORING LOG BORING GP-1 10/6/06

Elevation (ft., msl)	Depth (feet)	Time/Core Run Length	Recovery		RQD (%)	Fractures per foot	Graphic Log	ROCK DESCRIPTION/NOTES	DISCONTINUITIES
			(in.)	(%)					
705	45		60	100	6	>10	<p><b>SHALE-</b> dark gray, moderately weathered, weak, closely fractured</p>	<p>Joint, 50 degrees to axis, closely spaced, smooth, undulating (JRC 4-6), tight, no infill, dry Core rate 4 min/ft Core rate 5 min/ft</p>	
							<p><b>SHALE-</b> dark gray, highly weathered, extremely weak, pervasively sheared</p>	<p>Shear Zone, extremely closely spaced, slickensides, moist, zone extends from 46 to 49 feet bgs Core rate 6 min/ft</p>	
700	50		60	100	30	>10	<p><b>SANDSTONE-</b> light gray, slightly weathered, moderately strong, closely fractured</p>	<p>Joint, 15 degrees to axis, rough, undulating (JRC 10-12), no infill, dry Core rate 3 min/ft Core rate 2 min/ft</p>	
							<p><b>SHALE-</b> dark gray, moderately weathered, extremely weak, extremely closely sheared</p>	<p>Shear zone, extremely closely spaced, slickensides, shear zone extends from 51 to 52.5 feet bgs Core rate 1 min/ft</p> <p>Joint, 40 degrees to axis, closely spaced, slightly rough (JRC 10-12) Core rate 2 min/ft</p>	
695	55		60	100	18	>10	<p><b>SANDSTONE-</b> gray, highly to completely weathered, very closely fractured, extremely weak</p>	<p>Core rate 2 min/ft</p>	
							<p><b>SANDSTONE-</b> light gray, slightly to moderately weathered, very weak, closely fractured</p>	<p>Shear zone, extremely closely spaced, slickensides, shear zone extends from 56.5 to 58 feet bgs Core rate 2 min/ft</p> <p>Core rate 2 min/ft</p>	
690	60		60	100	10	>10	<p><b>SANDSTONE-</b> light gray, slightly to moderately weathered, very weak, closely fractured</p>	<p>Shear, 30 degrees to axis, very closely spaced, rough, undulating (JRC 10-12), clay infill, slickensides on shear faces Core rate 2 min/ft</p>	
							<p>Boring completed at a depth of approximately 61 feet below existing site grade.</p>		
685	65								



**LOG OF BORING K-1**  
 Building 50 Seismic Retrofit  
 Lawrence Berkeley National Laboratory  
 Berkeley, California

PLATE  
3 of 3

Drafted By: R. Padgett      Project No.: 75173  
 Date: 10/6/2006              File Number: 75173

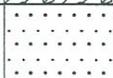
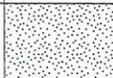
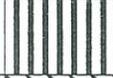
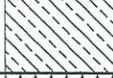
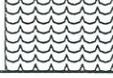
**A-2**

**Fugro, 2002**  
**Test Boring B-1 and B-2**

REFERENCE:

Fugro West, Inc., *Geotechnical Investigation, Proposed Building 50X, Lawrence Berkeley National Laboratory, Berkeley, California (FW 658.052)* bound report dated August 5, 2002

# UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D2487-93)

MAJOR DIVISIONS			GROUP NAMES		
COARSE-GRAINED SOILS More than 50% retained on the No. 200 sieve	GRAVELS  MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	Clean gravels less than 5% fines	GW		Well-graded gravel, Well-graded gravel with sand
			GP		Poorly graded gravel, Poorly graded gravel with sand
		Gravels with more than 12% fines	GM		Silty gravel, Silty gravel with sand
			GC		Clayey gravel, Clayey gravel with sand
	SANDS  MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	Clean sand less than 5% fines	SW		Well-graded sand, Well-graded sand with gravel
			SP		Poorly graded sand, Poorly graded sand with gravel
		Sands with more than 12% fines	SM		Silty sand, Silty sand with gravel
			SC		Clayey sand, Clayey sand with gravel
FINE-GRAINED SOILS 50% or more passes the No. 200 sieve	SILTS AND CLAYS  Liquid Limit Less than 50%		ML		Silt, Silt with sand or gravel, Sandy or gravelly silt, Sandy or gravelly silt with gravel or sand
			CL		Lean clay, Lean clay with sand or gravel, Sandy or gravelly lean clay, Sandy or gravelly lean clay with gravel or sand
			OL		Organic silt or clay, Organic silt or clay with sand or gravel, Sandy or gravelly organic silt or clay, Sandy or gravelly organic silt or clay with gravel or sand
	SILTS AND CLAYS  Liquid Limit Greater than 50%		MH		Elastic silt, Elastic silt with sand or gravel, Sandy or gravelly elastic silt, Sandy or gravelly elastic silt with gravel or sand
			CH		Fat clay, Fat clay with sand or gravel, Sandy or gravelly fat clay, Sandy or gravelly fat clay with gravel or sand
			OH		Organic silt or clay, Organic silt or clay with sand or gravel, Sandy or gravelly organic silt or clay, Sandy or gravelly organic silt or clay with gravel or sand
HIGHLY ORGANIC SOILS			PT		Peat

For definition of dual and borderline symbols, see ASTM D2487-93.

## KEY TO TEST DATA AND SYMBOLS

Perm - Permeability Consol - Consolidation LL - Liquid Limit PI - Plasticity Index Gs - Specific Gravity MA - Particle Size Analysis -200 - Percent Passing No. 200 Sieve ND - Not Detected ■ - Tube Sample ☒ - Bag or Bulk Sample □ - Lost Sample ▽ - First Groundwater ▾ - Stabilized Groundwater	Shear Strength (psf) TxUU 3200 TxCU 3200 TxCD 3200 SSCU 3200 SSCD 3200 DSCD 2700 UC 470 LVS 700 FV 300 RFV 300 TV 800 PP 400	Confining Pressure (psf) (2600) (2600) (2600) (2600) (2600) (2000) Unconfined Compression Laboratory Vane Shear Field Vane Shear Torvane Shear Pocket Penetrometer <i>(actual reading divided by 2)</i>
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FUGRO SOIL CLASSIFICATION 658-052.GPJ SCI\_CORP.GDT 5/15/02



**FUGRO WEST, INC.**  
 1000 Broadway, Suite 200, Oakland, California 94607  
 Tel: (510) 268-0461, Fax: (510) 268-0137

Building 50X  
 Berkeley, California

JOB NUMBER  
 658.052

DATE  
 5/02

APPROVED

PLATE

A1

**BEDDING OF SEDIMENTARY ROCKS**

- Very thick-bedded . . . . . Greater than 4.0
- Thick-bedded . . . . . 2.0 to 4.0
- Thin-bedded . . . . . 0.2 to 2.0
- Very thin-bedded . . . . . 0.05 to 0.2
- Laminated . . . . . 0.01 to 0.05
- Thinly laminated . . . . . less than 0.01



Bed thickness in feet

**FRACTURING**

- Very little fractured . . . . . Greater than 4.0
- Occasionally fractured . . . . . 1.0 to 4.0
- Moderately fractured . . . . . 0.5 to 1.0
- Closely fractured . . . . . 0.1 to 0.5
- Intensely fractured . . . . . 0.05 to 0.1
- Crushed . . . . . less than 0.05



Size of pieces in feet

**HARDNESS**

- Soft . . . . . reserved for plastic material alone.
- Low hardness . . . . . can be gouged deeply or carved easily with a knife blade.
- Moderately hard . . . . . can be readily scratched by a knife blade; scratch leaves a heavy trace of dust and is readily visible after the powder has been blown away.
- Hard . . . . . can be scratched with difficulty; scratch produces little powder and is often faintly visible.
- Very hard . . . . . cannot be scratched with knife blade; leaves a metallic streak.

**STRENGTH**

- Plastic . . . . . very low strength.
- Friable . . . . . crumbles easily by rubbing with fingers.
- Weak . . . . . an unfractured specimen of such material will crumble under light hammer blows.
- Moderately strong . . . . . specimen will withstand a few heavy hammer blows before breaking.
- Strong . . . . . specimen will withstand a few heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.
- Very strong . . . . . specimen will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.

**WEATHERING**

- Deep . . . . . moderate to complete mineral decomposition, extensive disintegration, deep and thorough discoloration, many fractures, all extensively coated or filled with oxides, carbonates and/or clay or silt.
- Moderate . . . . . slight change or partial decomposition of minerals, little disintegration; cementation little to unaffected. Moderate to occasionally intense discoloration. Moderately coated fractures.
- Little . . . . . no megascopic decomposition of minerals; little or no effect on normal cementation. Slight and intermittent, or localized discoloration. Few stains on fracture surfaces.
- Fresh . . . . . unaffected by weathering agents. No disintegration or discoloration. Fractures usually less numerous than joints.

**ROCK CLASSIFICATION CRITERIA**



**FUGRO WEST, INC.**  
 1000 Broadway Suite 200, Oakland, California 94607  
 Tel: (510) 268-0461, Fax: (510) 268-0137

Building 50X  
 Berkeley, California

JOB NUMBER  
 658.052

DATE  
 5/02

PLATE

**A2**

**LOG OF BORING**

Project Name & Location: Building 50X Berkeley, California		Ground Surface Elevation: 700 feet	
		Elevation Datum: LBL Project Datum	
Drilling Coordinates:		Start: Date	Time
Drilling Company & Driller: Gregg Jason/Lou		4/27/02	08:00
Rig Type & Drilling Method: Mobile 61, Hollow Stem Auger		Finish: Date	Time
		4/27/02	10:45
Sampler A) Modified California (3" O.D., 2.5" I.D.) Type(s): B) SPT (2" O.D., 1.4" I.D.)		Drilling Fluid:	Hole Diameter:
		NA	6.5"
Sampling Method(s): A) 140 lb hammer with 30" drop (Wireline) B) 140 lb hammer with 30" drop (Wireline)		Logged By:	
		AF	
		Backfill Method:	Date:
		Cement Grout	4/27/02

Depth (feet)	Sampler Type	Blows/6 inches or pressure	Blows/12 inches	Sample Interval	Graphic Log	SOIL DESCRIPTIONS		LABORATORY DATA			
						GROUP NAME (GROUP SYMBOL) color, consistency/density, moisture condition, other descriptions (Local Name or Material Type)	Moisture Content (%)	Dry Density (pcf)	Other		
0						ASPHALTIC CONCRETE 3 - INCHES THICK GRAVELLY LEAN CLAY (CL) Dark brown, soft, moist (fill)					
2	A	22	4				13.9	92			
5	A	12 17 25	42			GRAVELLY LEAN CLAY (CL) Brown, stiff to very stiff, moist With rock fragments at 5.0'	15.1	102			
10	A	8 7 8	15			With reddish brown sand pockets	18.3	102			
15	A	12 17 25	42								
20	A	18 35 40	75			SANDSTONE Brown, intensely to moderately fractured, low to moderately hard, friable, moderate to deep weathering	21.6	102	TxUU = 2830 psf PP = 3250 psf		
25	A	16 23 28	51			Trace reddish brown sandstone fragments Gray clay filled seams	18.8	108	PP = 3750 psf		
30	B	7 11 15	26			Color change to brownish gray at 29.0'					
32						Hard drilling at 32.0'					
33	B	50/4"	50/4"			Refusal at 33.0'					
35						Bottom of boring at 33.4 feet below ground surface. Notes Groundwater not encountered during drilling					

FUGRO LOG OF BORING 658-052.GPJ SCI\_CORP\_GDT 6/13/02



**FUGRO WEST, INC.**  
1000 Broadway, Suite 200, Oakland, California 94607  
Tel: (510) 268-0461, Fax: (510) 268-0137

Building 50X  
Berkeley, California

JOB NUMBER  
658.052

DATE  
6/02

BORING  
**B-1**

**LOG OF BORING**

Project Name & Location: Building 50X Berkeley, California		Ground Surface Elevation: 646 feet	
		Elevation Datum: LBL Project Datum	
Drilling Coordinates:		Start: Date	Time
		4/27/02	11:30
Drilling Company & Driller: Gregg Jason/Lou		Finish: Date	Time
		4/27/02	13:30
Rig Type & Drilling Method: Mobile 61, Hollow Stem Auger		Drilling Fluid:	Hole Diameter:
		NA	6.5"
Sampler A) Modified California (3" O.D., 2.5" I.D.) Type(s): B) SPT (2" O.D., 1.4" I.D.)		Logged By:	
		AF	
Sampling Method(s): A) 140 lb hammer with 30" drop (Wireline) B) 140 lb hammer with 30" drop (Wireline)		Backfill Method:	Date:
		Cement Grout	4/27/02

Depth (feet)	Sampler Type	Blows/6 inches or pressure	Blows/12 inches	Sample Interval	Graphic Log	SOIL DESCRIPTIONS		LABORATORY DATA		
						GROUP NAME (GROUP SYMBOL) color, consistency/density, moisture condition, other descriptions (Local Name or Material Type)	Moisture Content (%)	Dry Density (pcf)	Other	
0						ASPHALTIC CONCRETE 4 - INCHES THICK				
0 - 3	A	8 11 16	27			SANDY LEAN CLAY WITH GRAVEL (CL) Dark brown, stiff, moist, with siltstone at 3 feet (fill)	13.3	111		
3 - 5	A	6 8 13	21			SANDY LEAN CLAY WITH GRAVEL (CL) Brown, stiff, moist	16.7	110		
5 - 10	A	9 11 30	41			SANDSTONE Brown, intensely to moderately fractured, low to moderately hard, friable to weak, moderate weathering	18.4	104	TxUU = 1740 psf	
10 - 15	A	50/6"	50/6"				10.5	111	PP = 2000 psf	
15 - 20	B	17 18 22	40				18.4	99		
20 - 24	B	20 30 50	80				7	121		
24 - 25	B	50/4"	50/4"							
Bottom of boring at 24 feet below ground surface. Notes Groundwater not encountered during drilling										

FUGRO LOG OF BORING 658-052.GPJ SCI\_CORP.GDT 6/13/02

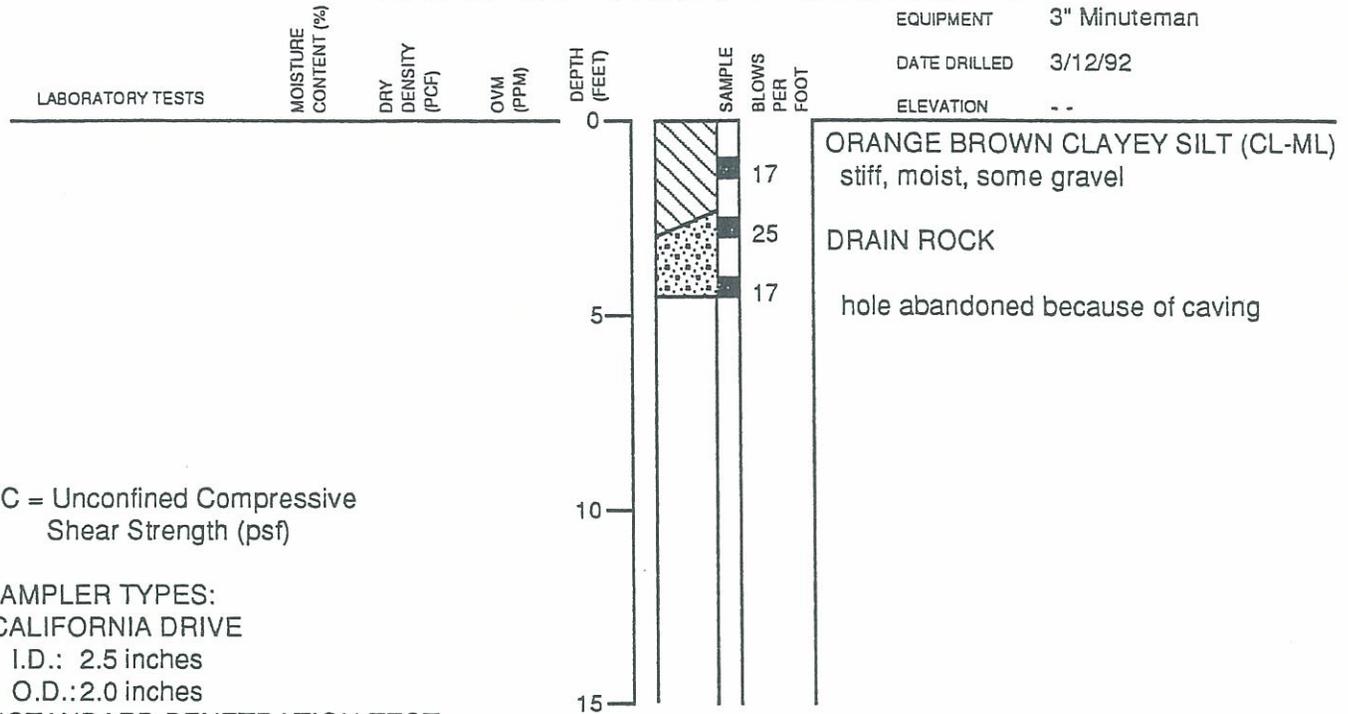
<b>FUGRO WEST, INC.</b> 1000 Broadway, Suite 200, Oakland, California 94607 Tel: (510) 268-0461, Fax: (510) 268-0137	Building 50X Berkeley, California		BORING <b>B-2</b>
	JOB NUMBER 658.052	DATE 6/02	

**SCI, 1992**  
**Test Boring B-1**

REFERENCE:

Subsurface Consultants, Inc., *Geotechnical Investigation, Acid Neutralization Tank Enclosure, Building 70a, Lawrence Berkeley Laboratory, Berkeley, California* (SCI 658.007) letter report dated March 30, 1992

# LOG OF TEST BORING 1



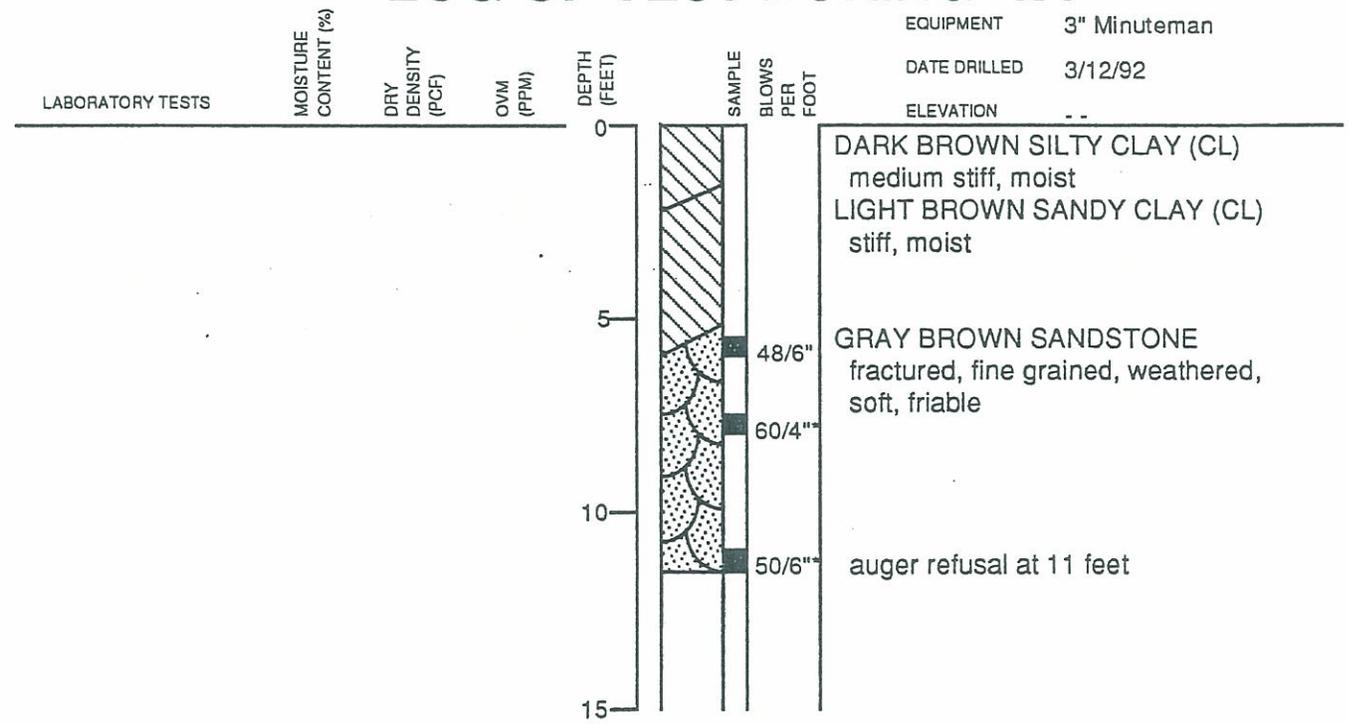
UC = Unconfined Compressive Shear Strength (psf)

SAMPLER TYPES:  
 CALIFORNIA DRIVE  
 I.D.: 2.5 inches  
 O.D.: 2.0 inches

\*STANDARD PENETRATION TEST  
 O.D.: 2.0 inches  
 I.D.: 1.4 inches

HAMMER WEIGHT: 70 pounds  
 HAMMER DROP: 30 inches

# LOG OF TEST BORING 1A



GENERAL SOIL CATEGORIES		SYMBOLS		TYPICAL SOIL TYPES
<b>COARSE GRAINED SOILS</b> More than half is larger than No. 200 sieve	<b>GRAVEL</b> More than half coarse fraction is larger than No. 4 sieve size	Clean Gravel with little or no fines	GW	Well Graded Gravel, Gravel-Sand Mixtures
			GP	Poorly Graded Gravel, Gravel-Sand Mixtures
		Gravel with more than 12% fines	GM	Silty Gravel, Poorly Graded Gravel-Sand-Silt Mixtures
			GC	Clayey Gravel, Poorly Graded Gravel-Sand-Clay Mixtures
	<b>SAND</b> More than half coarse fraction is smaller than No. 4 sieve size	Clean sand with little or no fines	SW	Well Graded Sand, Gravelly Sand
			SP	Poorly Graded Sand, Gravelly Sand
		Sand with more than 12% fines	SM	Silty Sand, Poorly Graded Sand-Silt Mixtures
			SC	Clayey Sand, Poorly Graded Sand-Clay Mixtures
<b>FINE GRAINED SOILS</b> More than half is smaller than No. 200 sieve	<b>SILT AND CLAY</b> Liquid Limit Less than 50%	ML	Inorganic Silt and Very Fine Sand, Rock Flour, Silty or Clayey Fine Sand, or Clayey Silt with Slight Plasticity	
		CL	Inorganic Clay of Low to Medium Plasticity, Gravelly Clay, Sandy Clay, Silty Clay, Lean Clay	
		OL	Organic Clay and Organic Silty Clay of Low Plasticity	
	<b>SILT AND CLAY</b> Liquid Limit Greater than 50%	MH	Inorganic Silt, Micaceous or Diatomaceous Fine Sandy or Silty Soils, Elastic Silt	
		CH	Inorganic Clay of High Plasticity, Fat Clay	
		OH	Organic Clay of Medium to High Plasticity, Organic Silt	
<b>HIGHLY ORGANIC SOILS</b>		PT	Peat and Other Highly Organic Soils	

UNIFIED SOIL CLASSIFICATION SYSTEM

Subsurface Consultants

LBL ACID TANK ENCLOSURE - BERKELEY, CA  
 JOB NUMBER 658.007      DATE 3/30/92      APPROVED SL

PLATE

4

**BEDDING OF SEDIMENTARY ROCKS**

Very thick-bedded .....	Greater than 4.0	} Bed thickness in feet
Thick-bedded .....	2.0 to 4.0	
Thin-bedded .....	0.2 to 2.0	
Very thin-bedded .....	0.05 to 0.2	
Laminated .....	0.01 to 0.05	
Thinly laminated .....	less than 0.01	

**FRACTURING**

Very little fractured .....	Greater than 4.0	} Size of pieces in feet
Occasionally fractured .....	1.0 to 4.0	
Moderately fractured .....	0.5 to 1.0	
Closely fractured .....	0.1 to 0.5	
Intensely fractured .....	0.05 to 0.1	
Crushed .....	less than 0.05	

**HARDNESS**

- Soft ..... reserved for plastic material alone.
- Low hardness ..... can be gouged deeply or carved easily with a knife blade.
- Moderately hard ..... can be readily scratched by a knife blade; scratch leaves a heavy trace of dust and is readily visible after the powder has been blown away.
- Hard ..... can be scratched with difficulty; scratch produces little powder and is often faintly visible.
- Very hard ..... cannot be scratched with knife blade; leaves a metallic streak.

**STRENGTH**

- Plastic ..... very low strength.
- Friable ..... crumbles easily by rubbing with fingers.
- Weak ..... an unfractured specimen of such material will crumble under light hammer blows.
- Moderately strong ... specimen will withstand a few heavy hammer blows before breaking.
- Strong ..... specimen will withstand a few heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.
- Very strong ..... specimen will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.

**WEATHERING**

- Deep ..... moderate to complete mineral decomposition, extensive disintegration, deep and thorough discoloration, many fractures, all extensively coated or filled with oxides, carbonates and/or clay or silt.
- Moderate ..... slight change or partial decomposition of minerals, little disintegration; cementation little to unaffected. Moderate to occasionally intense discoloration. Moderately coated fractures.
- Little ..... no megascopic decomposition of minerals; little or no effect on normal cementation. Slight and intermittent, or localized discoloration. Few stains on fracture surfaces.
- Fresh ..... unaffected by weathering agents. No disintegration or discoloration. Fractures usually less numerous than joints.

**ROCK CLASSIFICATION CRITERIA**

Subsurface Consultants

LBL ACID TANK ENCLOSURE - BERKELEY, CA

PLATE

JOB NUMBER  
658.007

DATE  
3/30/92

APPROVED  
SL

**5**

**GRC, 1990**  
**Test Borings B-1 through B-3**

REFERENCE:

Geo/Resource Consultants, Inc., *Proposed Air Handling Units (AHU), South of Building 70A, Lawrence Berkeley Laboratory, Berkeley, California* (GRC 1574-00-0) bound report dated May 22, 1990

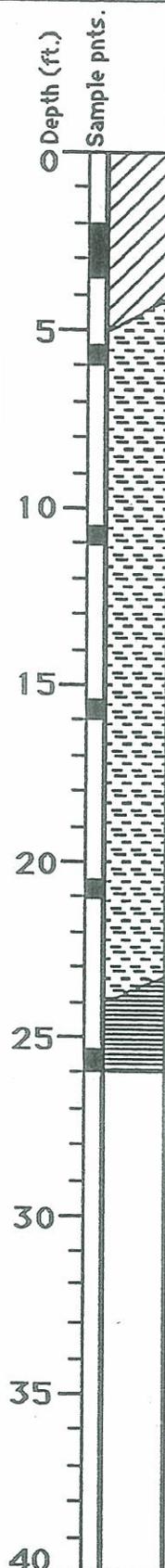
## LOG OF BORING B-1

Equipment Solid Flight Auger

Elevation ~ 730 ft. Date 4/5/90

### Laboratory Analyses

	Blows / ft.	Moisture Content (%)	Dry density (pcf)
Qu/2 = 4979 psf	50	12.2	115.0
	50/6"		
	35/6"		
DS/CU = 4342 psf @ 1 ksf = 3413 psf @ 2 ksf = 4361 psf @ 4 ksf	35/6"		
	40/6"		
	35/4"		



0	<b>BROWN SANDY CLAY (CL)</b> hard, with sandstone fragments, shale fragments, iron staining
5	<b>BROWN SILTSTONE</b> soft, friable, severely weathered, intensely fractured
10	
15	grades to moderately weathered @ 14 ft.
20	grades to moderately hard, moderately strong, moderately weathered, intensely fractured to crushed
25	<b>GRAY SHALE</b> soft, weak, slightly weathered, crushed; contains slickensides
30	Boring Terminated @ 26 ft. No groundwater was encountered during drilling
35	
40	

Blowcount is penetration resistance of California Modified Sampler driven by a 140 lbs hammer falling 30 inches.



**Geol/Resource Consultants, Inc.**  
Geologists / Engineers / Environmental Scientists

### LOG OF BORING B-1

LBL ADDITION A.H.U.  
LAWRENCE BERKELEY LABORATORY  
BERKELEY, CALIFORNIA

FIGURE

3

Job No. 1574-000-0 Appr: JY Date 4/10/90

## LOG OF BORING B-2

Equipment Solid Flight Auger

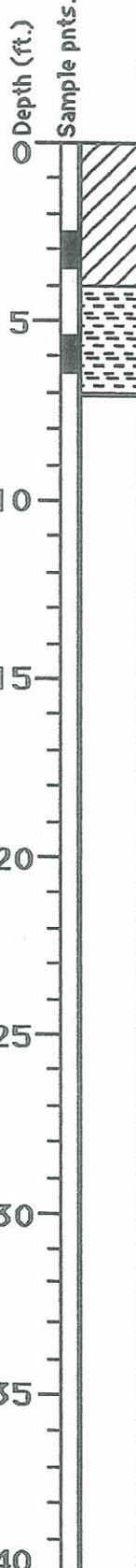
Elevation ~ 730 ft. Date 4/5/90

**Laboratory Analyses**

PL = 21  
 LL = 31    Qu/2  
 PI = 10    = 3400 psf

Blows / ft.  
 Moisture Content (%)  
 Dry density (pcf)

Blows / ft.	Moisture Content (%)	Dry density (pcf)
22	12.5	105
44		



0  
 BROWN SANDY CLAY (CL)  
 moist, very stiff, with sandstone fragments  
 occasional shale fragments, iron staining

5  
 BROWN SILTSTONE  
 moderately hard, moderately strong,  
 moderately weathered, closely fractured

10  
 Refusal @ 7 ft.  
 Boring Terminated @ 7 ft.  
 No groundwater was encountered during drilling

15

20

25

30

35

40

Blowcount is penetration resistance of California Modified Sampler driven by a 140 lbs hammer falling 30 inches.



### LOG OF BORING B-2

LBL ADDITION A.H.U.  
 LAWRENCE BERKELEY LABORATORY  
 BERKELEY, CALIFORNIA

**FIGURE**

**4**

Job No. 1574-000-0 Appr: JY Date 4/10/90

# LOG OF BORING B-2A

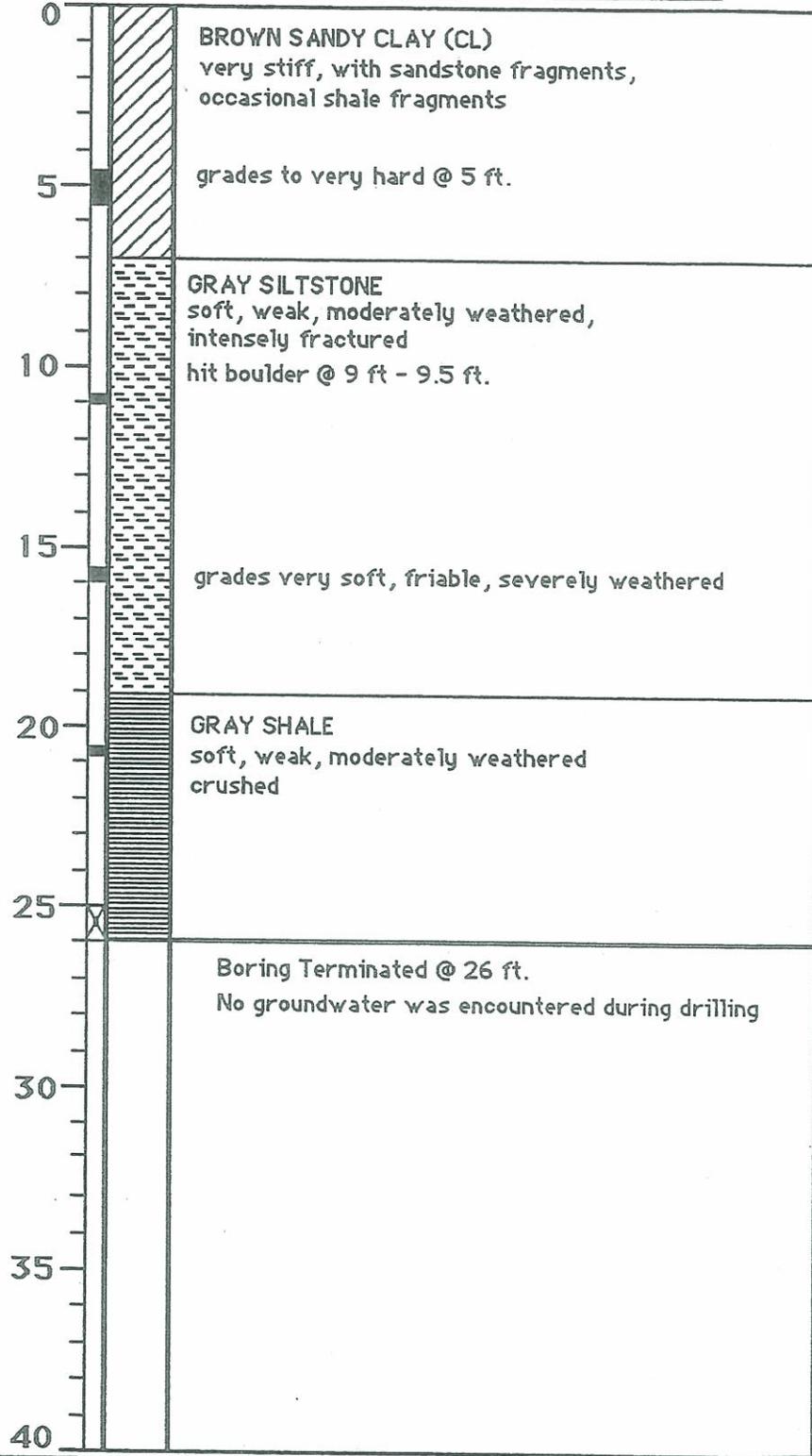
Equipment Solid Flight Auger

Elevation ~ 730 ft. Date 4/5/90

Laboratory Analyses

Blows/ft.  
Moisture Content (%)  
Dry density (pcf)

Depth (ft.)  
Sample pnts.



Blowcount is penetration resistance of California Modified Sampler driven by a 140 lbs hammer falling 30 inches.



**Geo/Resource Consultants, Inc.**  
Geologists / Engineers / Environmental Scientists

## LOG OF BORING B-2A

LBL ADDITION A.H.U.  
LAWRENCE BERKELEY LABORATORY  
BERKELEY, CALIFORNIA

FIGURE

5

Job No. 1574-000-0 Appr: JY Date 4/10/90

# LOG OF BORING B-3

Equipment Solid Flight Auger

Elevation ~ 730 ft. Date 4/5/90

**Laboratory Analyses**

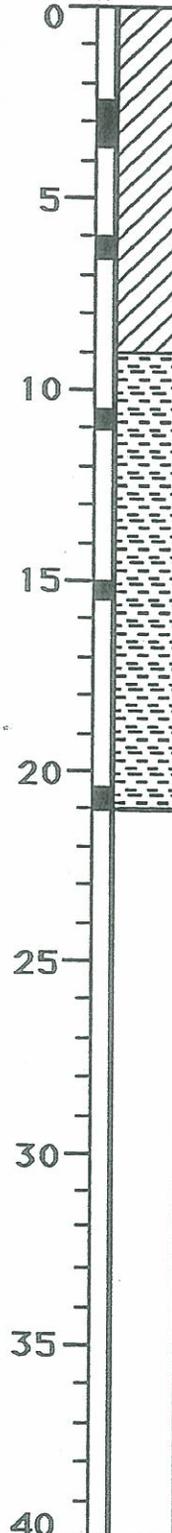
Blows/ft.  
Moisture Content (%)  
Dry density (pcf)

Depth (ft.)  
Sample pnts.

PL = 24  
LL = 40  
PI = 16

Qu/2 = 4392 psf

16		
30	12.3	106
45/6"		
55/6"		
60/6"		



**BROWN SANDY CLAY (CL)**  
moist, stiff, contains siltstone fragments

grades to very stiff @ 5 ft.

grades to hard @ 8.5 ft.

**BROWN SILTSTONE**  
soft, weak to moderately strong,  
moderately weathered,  
intensely fractured

becomes gray, moderately hard, moderately strong,  
slightly weathered

grades to gray shalestone @ 19 ft.

Boring Terminated @ 21 ft.  
No groundwater was encountered during drilling

Blowcount was obtained by 140 lbs. hammer, falling 30 inches, and S & H Sampler.



**Geo/Resource Consultants, Inc.**  
Geologists / Engineers / Environmental Scientists

## LOG OF BORING B-3

LBL ADDITION A.H.U.  
LAWRENCE BERKELEY LABORATORY  
BERKELEY, CALIFORNIA

**FIGURE**

**6**

Job No. 1574-000-0 Appr: JY Date 4/10/90

# UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS				SOIL DESCRIPTION	
COARSE GRAINED SOILS over 50% coarser than #200 sieve	GRAVELS over half of coarse fraction larger than No. 4 sieve	clean gravels with little or no fines	GW		Well Graded Gravels, Gravel - Sand Mixtures
			GP		Poorly Graded Gravels, Gravel - Sand Mixtures
		gravels with over 12% fines	GM		Silty Gravels, Poorly Graded Gravel - Sand - Silt Mixtures
			GC		Clayey Gravels, Poorly Graded Gravel-Sand-Clay Mixtures
	SANDS over half of coarse fraction finer than No. 4 sieve	clean sands with little or no fines	SW		Well Graded Sands, Gravelly Sands
			SP		Poorly Graded Sands, Gravelly Sands
		sands with over 12% fines	SM		Silty Sands, Poorly Graded Sand - Silt Mixtures
			SC		Clayey Sands, Poorly Graded Sand - Clay Mixtures
FINE GRAINED SOILS over 50% is finer than #200 sieve	SILTS AND CLAYS liquid limit less than 50		ML		Silts, Very Fine Sands, Silty or Clayey Fine Sands
			CL		Low Plasticity Clays, Sandy or Silty Clays
			OL		Low Plasticity Organic Silts and Clays
	SILTS AND CLAYS liquid limit greater than 50		MH		Micaceous or Diatomaceous Silts, Volcanic Ash, Elastic Silts
			CH		High Plasticity Clays - Fat Clays
			OH		High Plasticity Organic Silts and Clays
	HIGHLY ORGANIC SOILS		Pt		Peat and Other Fibrous Organic Soils

### KEY TO SAMPLES

	"Undisturbed" 2.5" sample
	Disturbed Sample
	Indicates depth of sampling with no recovery
	Indicates depth and location of coring run
	Indicates depth of Standard Penetration Test and 2" sample

### KEY TO TEST DATA

		Shear Strength, psf	
		Confining Pressure or Normal Load, psf	
TxUU	750 (2600)	Unconsolidated Undrained Triaxial	
TxCU	540 (2600)	Consolidated Undrained Triaxial	
TxCD	800 (2600)	Consolidated Drained Triaxial	
DS	500 (2000)	Direct Shear	
UC	400	Unconfined Compression	
FVS	470	Field Vane Shear	
FP	500	Field Penetrometer	
PI = Plasticity Index			
C = Consolidation Test			



**Geo/Resource Consultants, Inc.**  
Consulting Engineers, Geologists, Geophysicists

## SOIL CLASSIFICATION CHART AND KEY TO TEST DATA

FIGURE

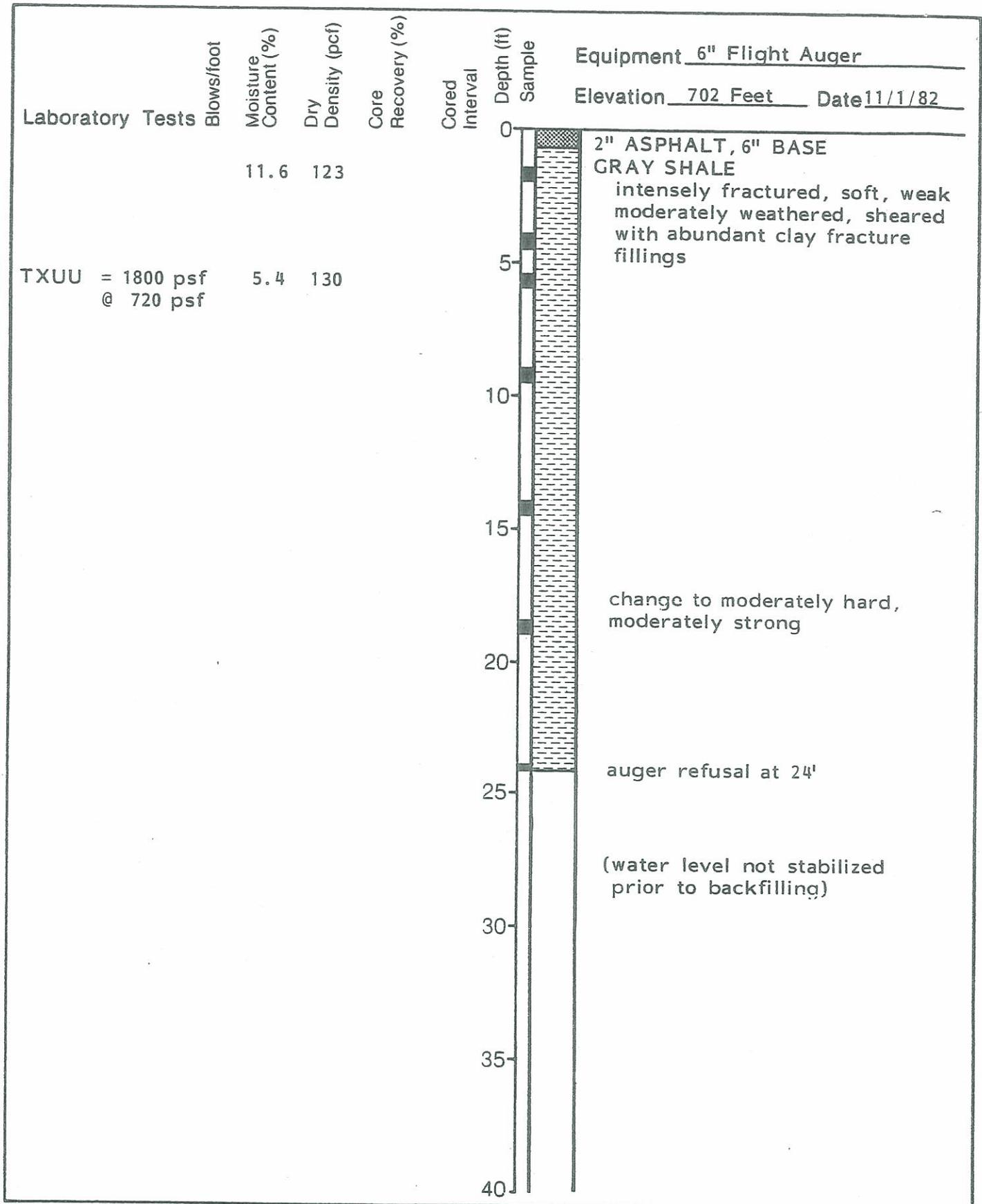
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Job No. 1574-00-0 Appr: JY Date 4/18/90

**HLA, 1983**  
**Test Borings 1.165 through 4.165**

REFERENCE:

Harding Lawson Associates, *Geotechnical Investigation, Building 50F, Office Addition, Lawrence Berkeley Laboratory, Berkeley, California*, (HLA 2000,165.01) bound report dated March 1, 1983



**Harding Lawson Associates**  
Engineers, Geologists  
& Geophysicists

**Log of Boring 1.165**  
Building 50F  
Lawrence Berkeley Laboratory  
Berkeley, California

PLATE

**2**

DRAWN  
J. Weitzel

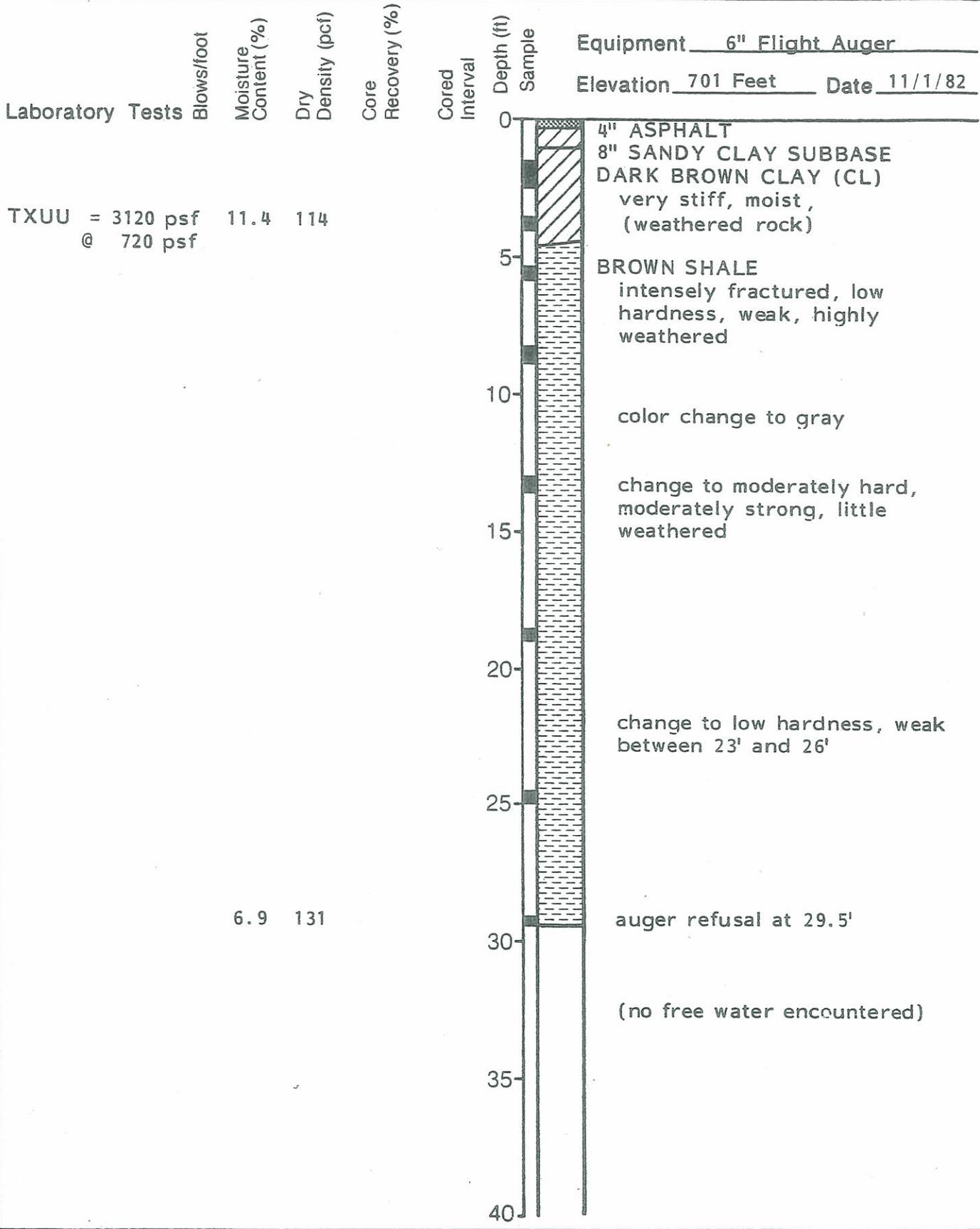
JOB NUMBER  
2000,165.01

APPROVED  
*LEL*

DATE  
1/83

REVISED

DATE



**Harding Lawson Associates**  
Engineers, Geologists  
& Geophysicists

**Log of Boring 2.165**  
Building 50F  
Lawrence Berkeley Laboratory  
Berkeley, California

PLATE

**3**

DRAWN  
J. Weitzel

JOB NUMBER  
2000,165.01

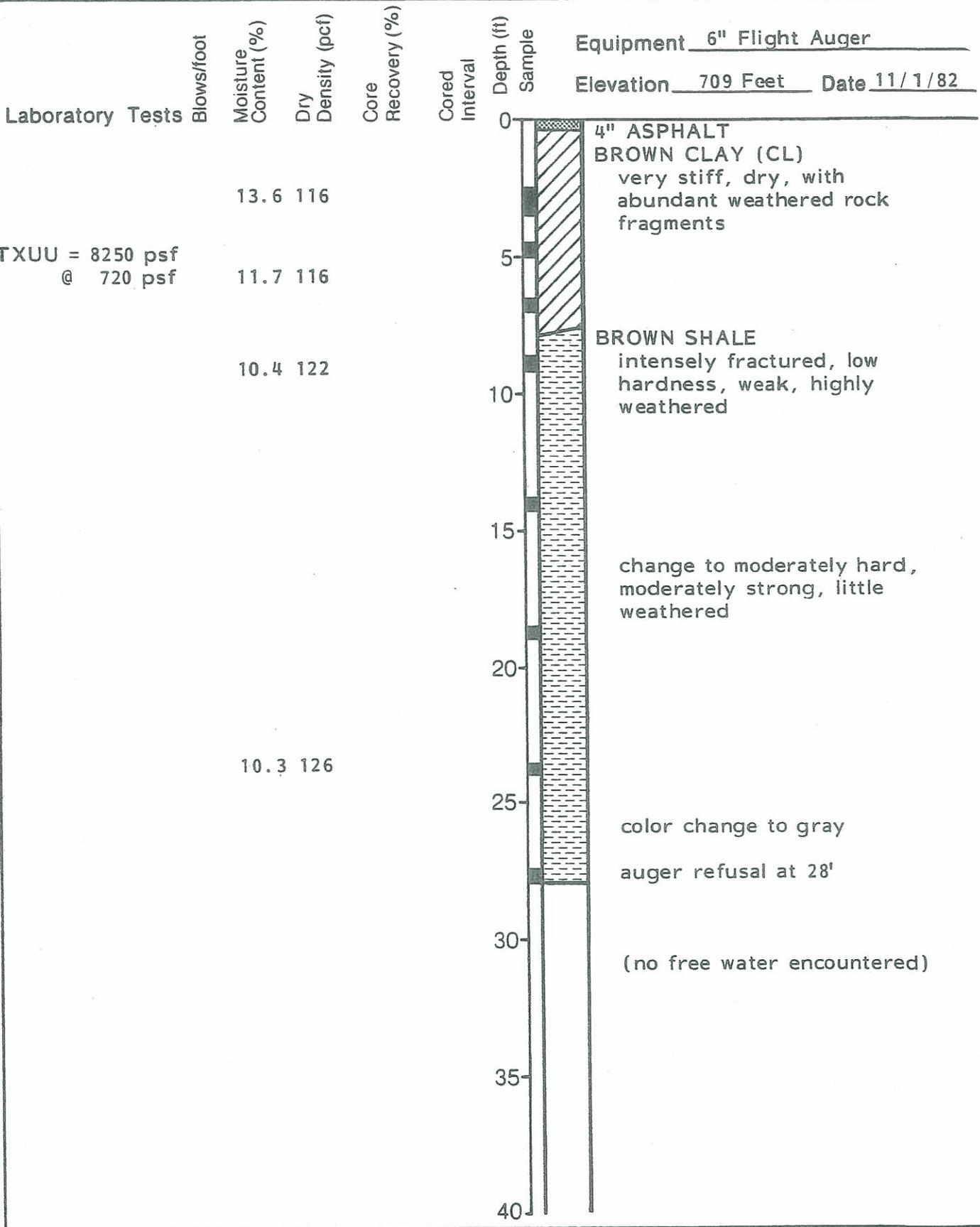
APPROVED  
LEL

DATE  
1/83

REVISED

DATE





**Harding Lawson Associates**  
Engineers, Geologists  
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**Log of Boring 4.165**  
Building 50F  
Lawrence Berkeley Laboratory  
Berkeley, California

PLATE  
**5**

MAJOR DIVISIONS				TYPICAL NAMES	
COARSE GRAINED SOILS MORE THAN HALF IS LARGER THAN NO. 200 SIEVE	GRAVELS  MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW		WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES
			GP		POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES
		GRAVELS WITH OVER 12% FINES	GM		SILTY GRAVELS, POORLY GRADED GRAVEL-SAND - SILT MIXTURES
			GC		CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND - CLAY MIXTURES
	SANDS  MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE	CLEAN SANDS WITH LITTLE OR NO FINES	SW		WELL GRADED SANDS, GRAVELLY SANDS
			SP		POORLY GRADED SANDS, GRAVELLY SANDS
		SANDS WITH OVER 12% FINES	SM		SILTY SANDS, POORLY GRADED SAND - SILT MIXTURES
			SC		CLAYEY SANDS, POORLY GRADED SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN HALF IS SMALLER THAN NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50	ML		INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS WITH SLIGHT PLASTICITY	
		CL		INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS	
		OL		ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY	
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50	MH		INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS	
		CH		INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS	
		OH		ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS	
HIGHLY ORGANIC SOILS		Pt		PEAT AND OTHER HIGHLY ORGANIC SOILS	

### UNIFIED SOIL CLASSIFICATION SYSTEM

Consol — Consolidation	Shear Strength, psf	Confining Pressure, psf	
LL — Liquid Limit (in %)	*Tx	320 (2600)	— Unconsolidated Undrained Triaxial
PL — Plastic Limit (in %)	TxCU	320 (2600)	— Consolidated Undrained Triaxial
G <sub>s</sub> — Specific Gravity	DS	2750 (2000)	— Consolidated Drained Direct Shear
SA — Sieve Analysis	FVS	470	— Field Vane Shear
■ — "Undisturbed" Sample	*UC	2000	— Unconfined Compression
⊠ — Bulk Sample	LVS	700	— Laboratory Vane Shear

### KEY TO TEST DATA



**Harding Lawson Associates**  
Engineers, Geologists  
& Geophysicists

**Soil Classification Chart  
and Key to Test Data**  
Building 50F  
Lawrence Berkeley Laboratory  
Berkeley, California

PLATE

**6**

DRAWN  
J. Weitzel

JOB NUMBER  
2000,165.01

APPROVED  
L.L.

DATE  
1/83

REVISED

DATE

**I CONSOLIDATION OF SEDIMENTARY ROCKS;** usually determined from unweathered samples. Largely dependent on cementation.

- U = unconsolidated
- P = poorly consolidated
- M = moderately consolidated
- W = well consolidated

**II BEDDING OF SEDIMENTARY ROCKS**

Splitting Property	Thickness	Stratification
Massive	Greater than 4.0 ft.	very thick bedded
Blocky	2.0 to 4.0 ft.	thick-bedded
Slabby	0.2 to 2.0 ft.	thin-bedded
Flaggy	0.05 to 0.2 ft.	very thin-bedded
Shaly or platy	0.01 to 0.05 ft.	laminated
Papery	less than 0.01 ft.	thinly laminated

**III FRACTURING**

Intensity	Size of Pieces in Feet
Very little fractured	Greater than 4.0
Occasionally fractured	1.0 to 4.0
Moderately fractured	0.5 to 1.0
Closely fractured	0.1 to 0.5
Intensely fractured	0.05 to 0.1
Crushed	Less than 0.05

**IV HARDNESS**

1. **Soft** — Reserved for plastic material alone
2. **Low hardness** — can be gouged deeply or carved easily with a knife blade
3. **Moderately hard** — can be readily scratched by a knife blade; scratch leaves a heavy trace of dust and is readily visible after the powder has been blown away.
4. **Hard** — can be scratched with difficulty; scratch produces little powder and is often faintly visible.
5. **Very hard** — cannot be scratched with knife blade; leaves a metallic streak.

**V STRENGTH**

1. **Plastic** or very low strength
2. **Friable** — crumbles easily by rubbing with fingers
3. **Weak** — An unfractured specimen of such material will crumble under light hammer blows.
4. **Moderately strong** — Specimen will withstand a few heavy hammer blows before breaking.
5. **Strong** — Specimen will withstand a few heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.
6. **Very strong** — Specimen will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments.

**VI WEATHERING** — The physical and chemical disintegration and decomposition of rocks and minerals by natural processes such as oxidation, reduction, hydration, solution, carbonation, and freezing and thawing.

- D. Deep** — Moderate to complete mineral decomposition; extensive disintegration; deep and thorough discoloration; many fractures; all extensively coated or filled with oxides, carbonates and/or clay or silt.
- M. Moderate** — Slight change or partial decomposition of minerals; little disintegration; cementation little to unaffected. Moderate to occasionally intense discoloration. Moderately coated fractures.
- L. Little** — No megascopic decomposition of minerals; little or no effect on normal cementation. Slight and intermittent, or localized discoloration. Few stains on fracture surfaces.
- F. Fresh** — Unaffected by weathering agents. No disintegration or discoloration. Fractures usually less numerous than joints.



**Harding Lawson Associates**  
 Engineers, Geologists  
 & Geophysicists

**Physical Properties Criteria  
 for Rock Descriptions**  
 Building 50F  
 Lawrence Berkeley Laboratory  
 Berkeley, California

PLATE

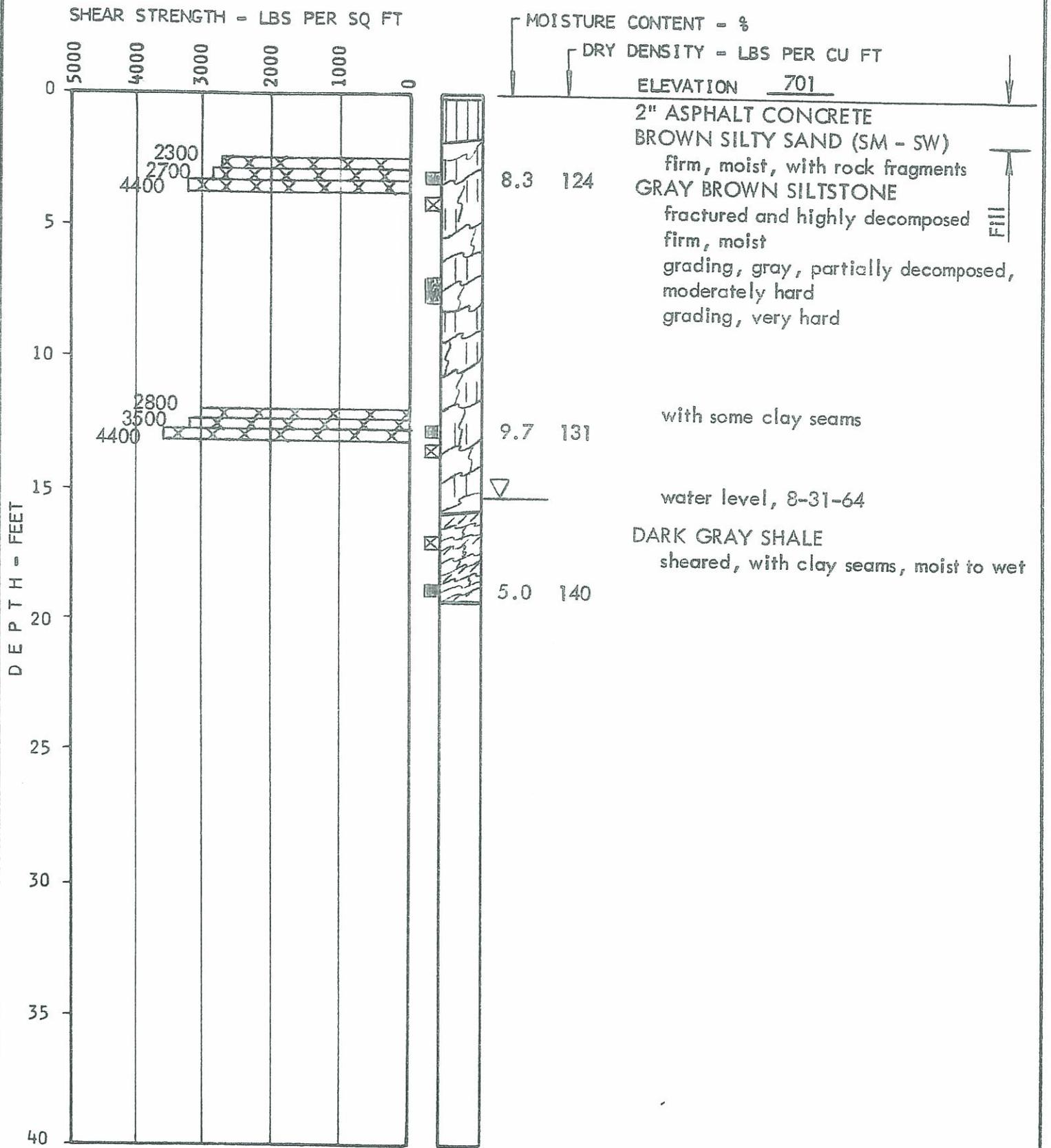
**7**

**Hardin Associates, 1965  
Test Borings 1 through 4**

REFERENCE:

Harding Associates, Planned Building 50-B, Lawrence Radiation Laboratory, Berkeley, California (HA 2000.23) plates, cross sections and boring logs dated January 20, 1965.

BORING 1



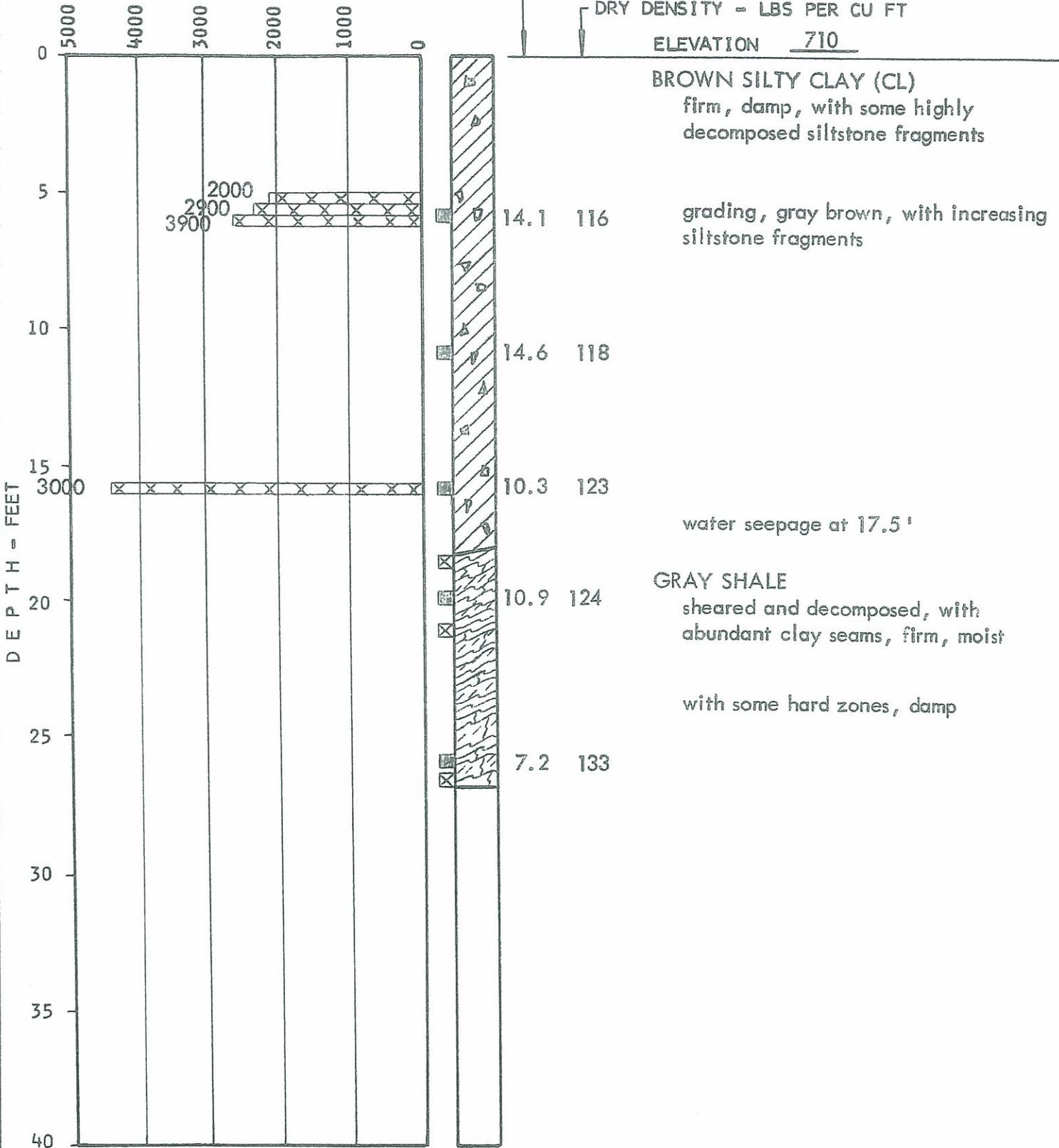
BORING 2

SHEAR STRENGTH - LBS PER SQ FT

MOISTURE CONTENT - %

DRY DENSITY - LBS PER CU FT

ELEVATION 710



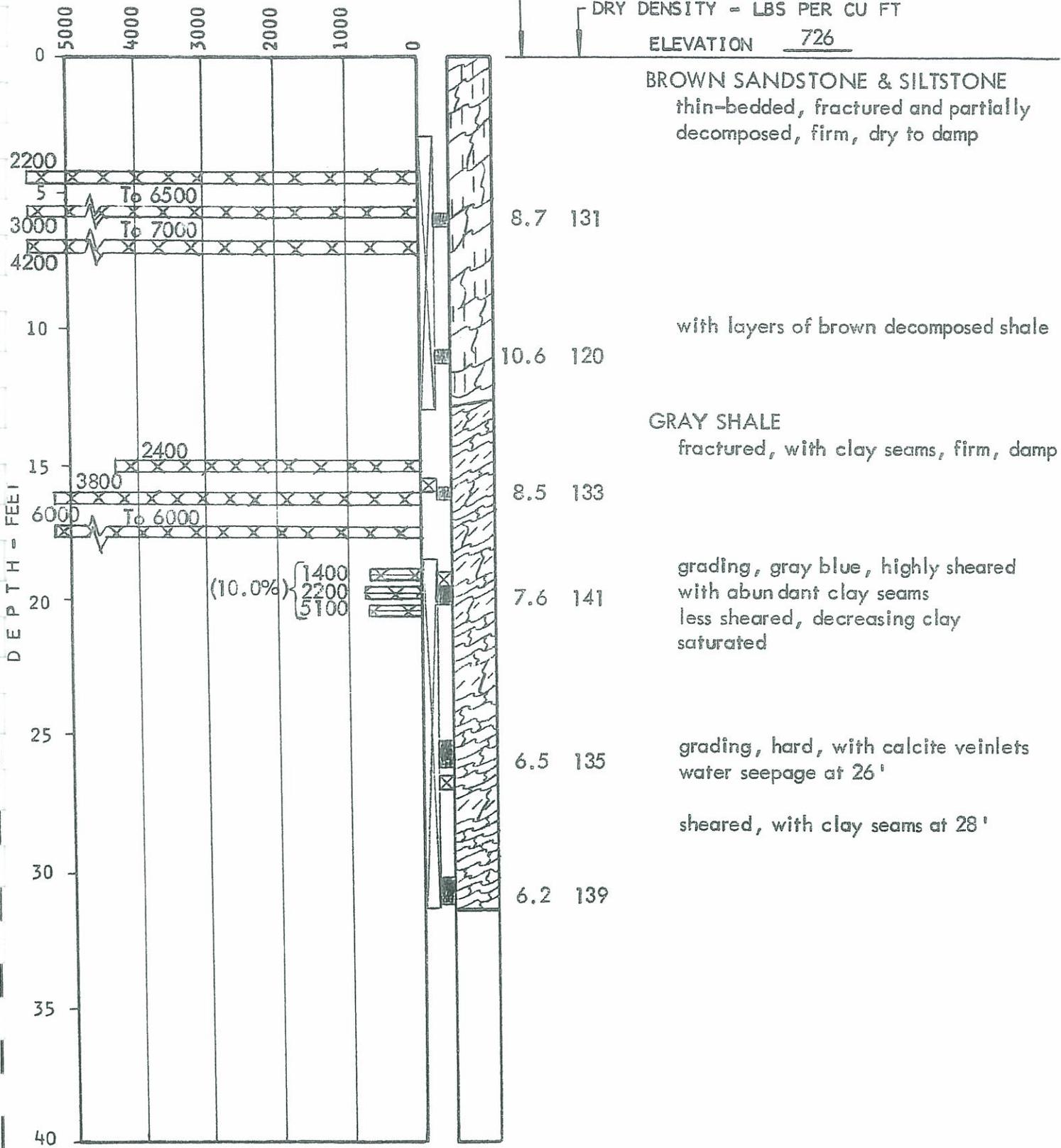
BORING 3

SHEAR STRENGTH - LBS PER SQ FT

MOISTURE CONTENT - %

DRY DENSITY - LBS PER CU FT

ELEVATION 726



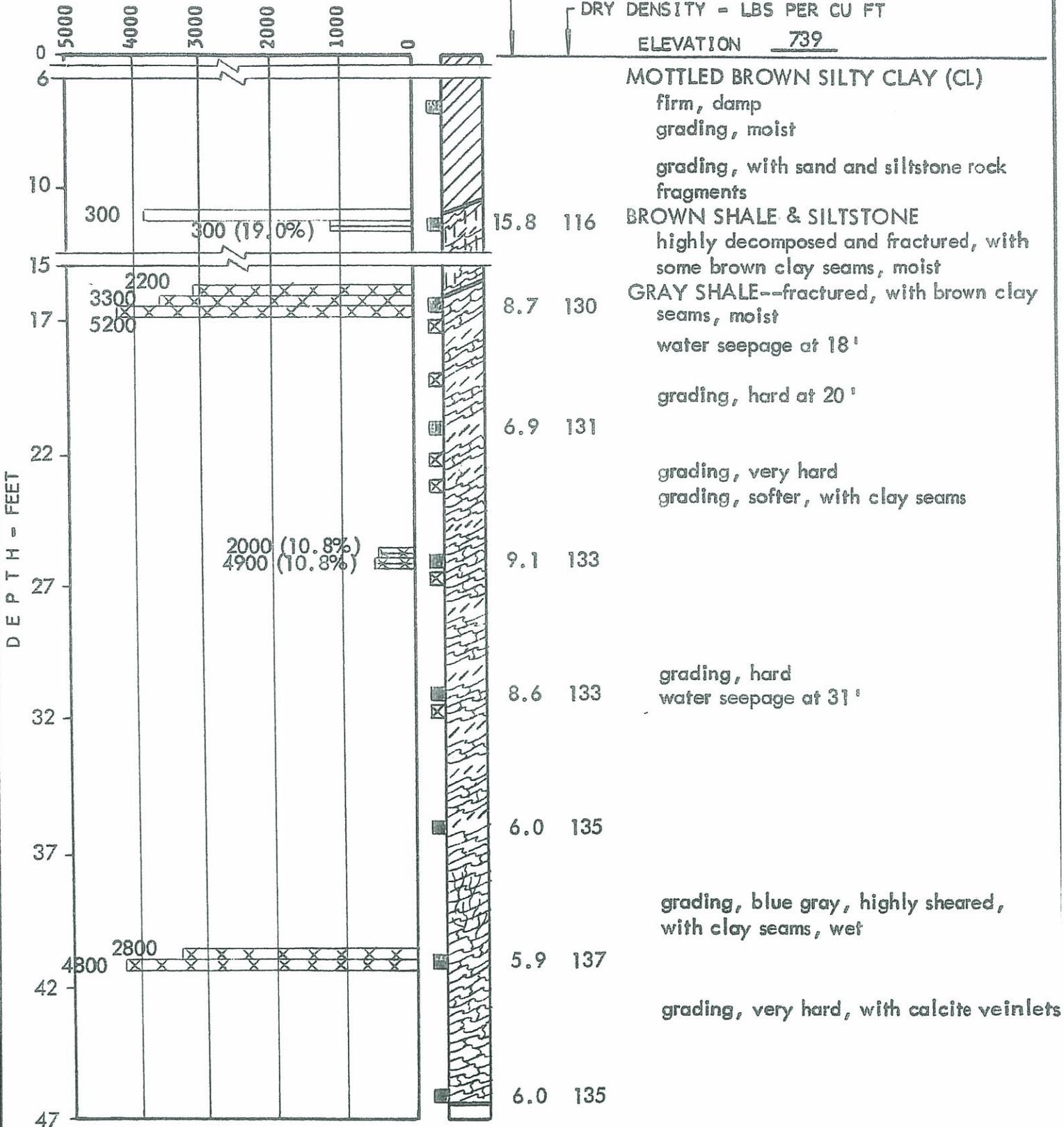
BORING 4

SHEAR STRENGTH - LBS PER SQ FT

MOISTURE CONTENT - %

DRY DENSITY - LBS PER CU FT

ELEVATION 739



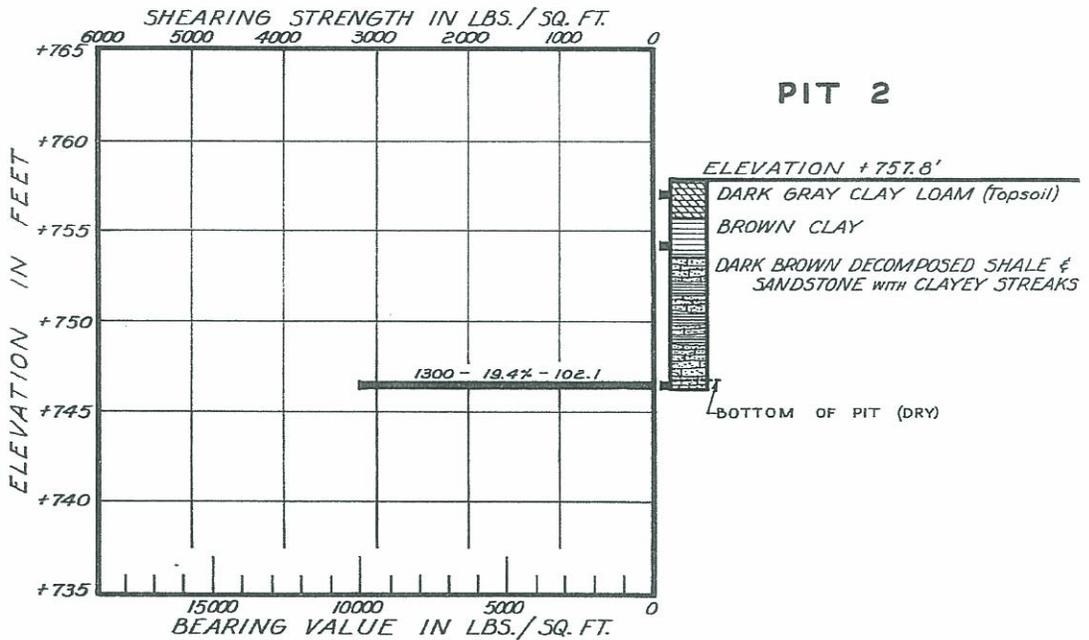
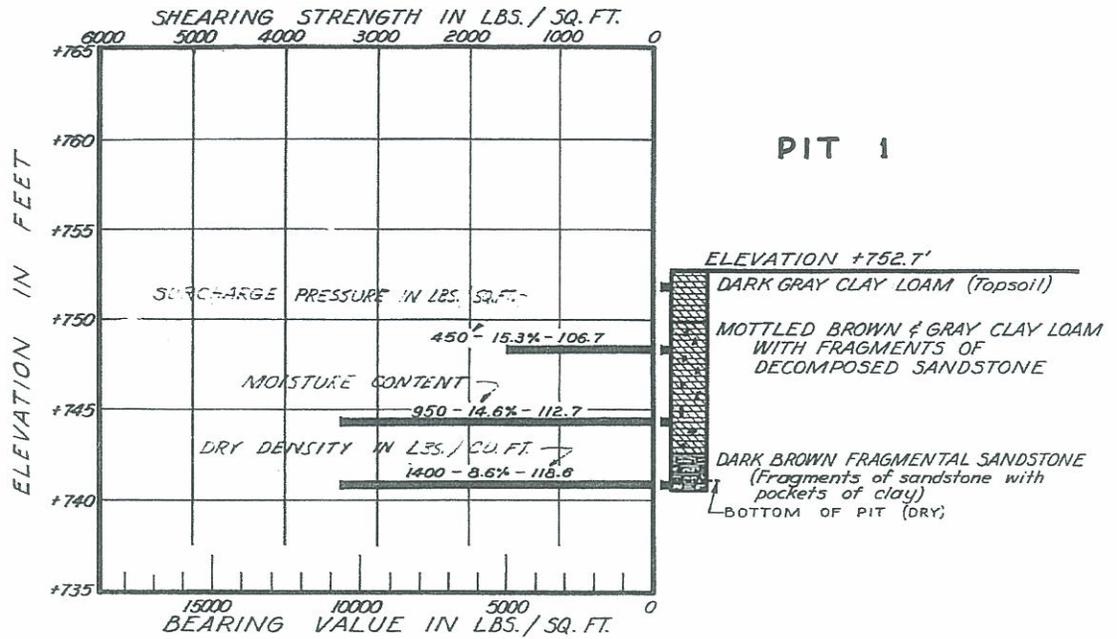
**D&M 1960**  
**Test Borings 2 through 4**

REFERENCE:

Dames & Moore, *Earth Pressure Recommendations, Proposed Retaining Wall, Proposed Addition to Building 50, Lawrence Radiation Laboratory, Berkeley, California* (no reference number), Report dated August 19, 1960

DRAWN BY: KEMPER DATE: 11/22/78  
 DATA CHECK BY: \_\_\_\_\_ DATE: \_\_\_\_\_  
 DRAFTING CHECK BY: \_\_\_\_\_ DATE: \_\_\_\_\_  
 REPORT DICTATED BY: \_\_\_\_\_ DATE: \_\_\_\_\_

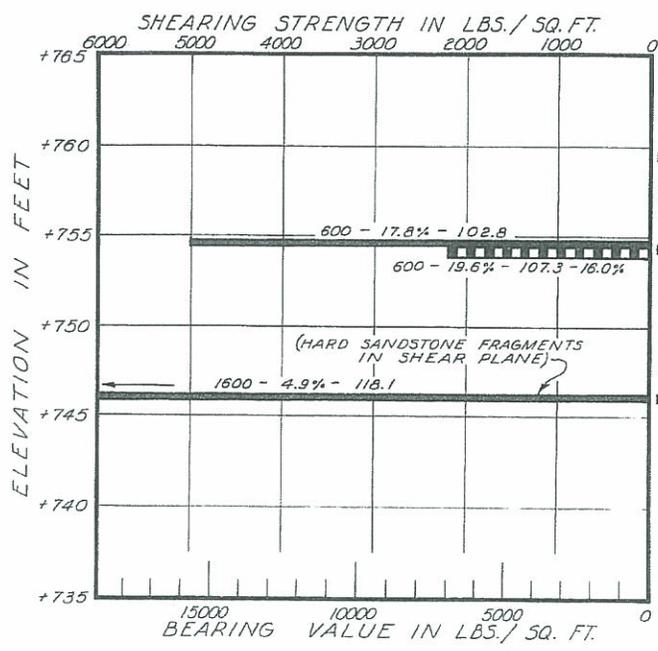
DAMES & MOORE  
 JOB TITLE: FOUNDATION ENGINEERING  
 LOCATION: 11th & I St. S.W.  
 CLIENT: ...  
 DATE JOB STARTED: \_\_\_\_\_ DATE PRINTED: \_\_\_\_\_  
 THIS DRAWING IS ONE OF A \_\_\_\_\_  
 SERIES OF \_\_\_\_\_ DRAWINGS



## LOG OF PITS

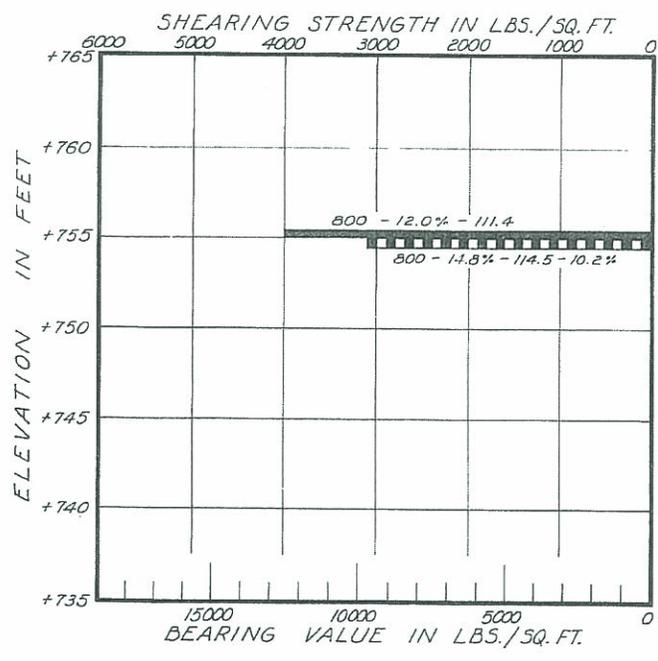
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 THIS DRAWING IS ONE OF A \_\_\_\_\_ CLIENT \_\_\_\_\_ DATE JOB STARTED \_\_\_\_\_  
 SERIES OF \_\_\_\_\_ DRAWINGS \_\_\_\_\_



**PIT 3**

ELEVATION +760.3'  
 DARK GRAY CLAY LOAM (Topsoil)  
 BROWN CLAY  
 MOTTLED BROWN & GRAY DECOMPOSED  
 SHALE & SANDSTONE WITH  
 CLAYEY STREAKS  
 BOTTOM OF PIT (DRY)



**PIT 4**

ELEVATION +762.1'  
 DARK GRAY CLAY LOAM WITH FRAGMENTS  
 OF DECOMPOSED SANDSTONE (Topsoil)  
 BROWN CLAY LOAM WITH  
 FEW ROOTS  
 BROWN & GRAY DECOMPOSED SHALE &  
 SANDSTONE WITH CLAYEY STREAKS  
 BOTTOM OF PIT (DRY)

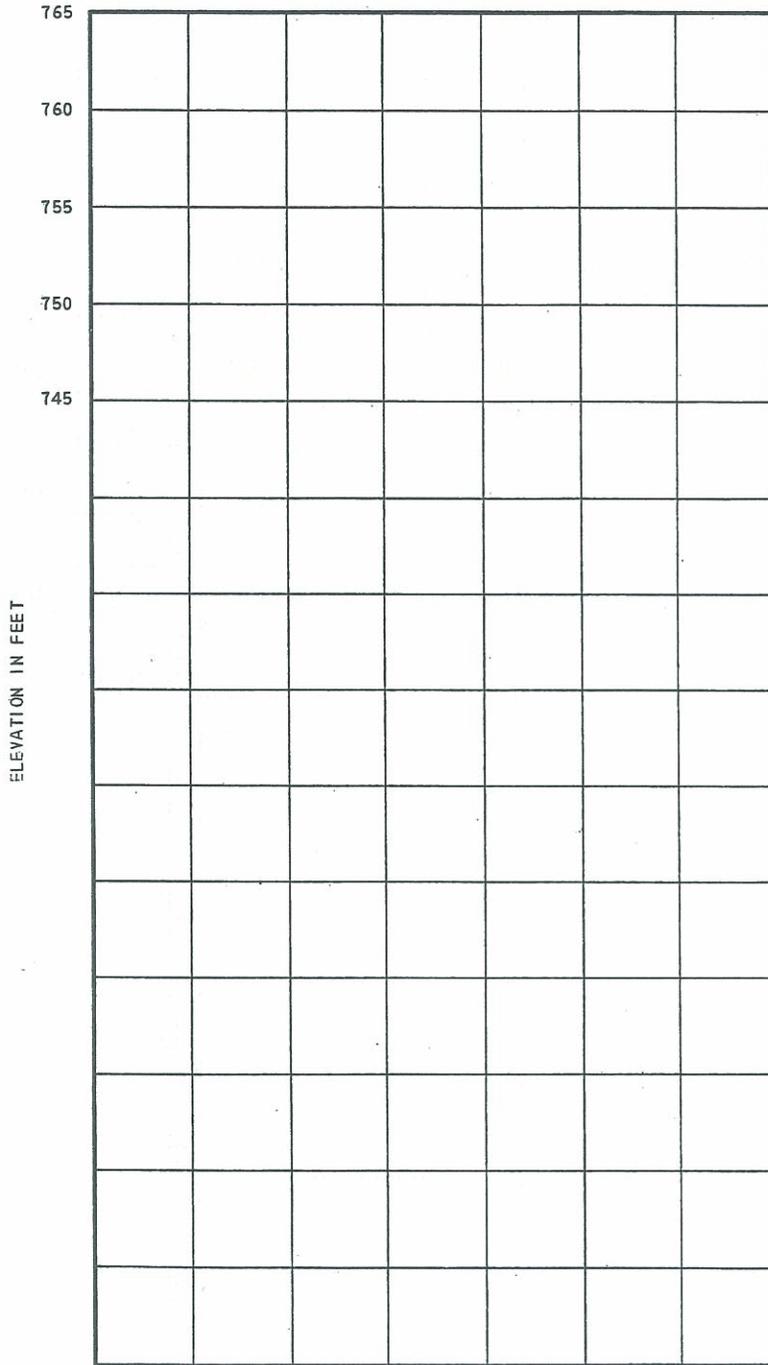
**LOG OF PITS**

**D&M 1959**  
**Test Borings 1a through 6**

REFERENCE:

Dames & Moore, Foundation Investigation, Proposed Building 70 Addition, Lawrence Radiation Laboratory, Berkeley, California (no reference number), Report dated April 2, 1959

**BORING I**  
DRILLED 3-6-59



ELEVATION 762.5'



LIGHT BROWN CLAY LOAM WITH SOME SMALL  
ROCK FRAGMENTS & WOOD (FILL)  
LIGHT BROWN DECOMPOSED SANDSTONE &  
SHALES INTERBEDDED  
(GRADING LESS DECOMPOSED)

BLUE-GRAY SANDSTONE, FRACTURED, SOME  
ALTERED ZONES, SEEPAGE

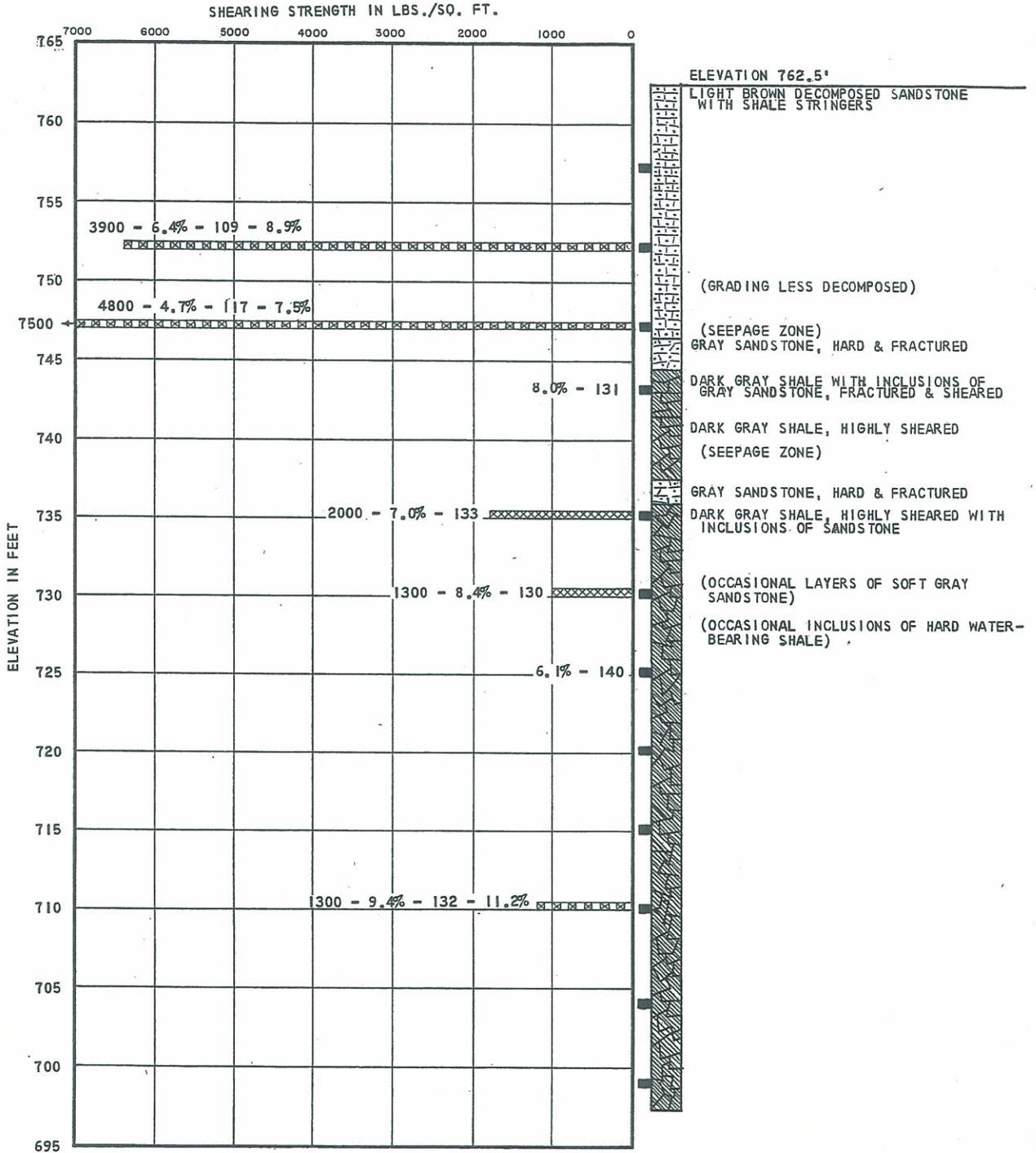
ELEVATION IN FEET

**LOG OF BORING**

BY \_\_\_\_\_ DATE \_\_\_\_\_  
PLATE \_\_\_\_\_ OF \_\_\_\_\_

BY RC DATE 4/17  
CHECKED BY DEAN'S DATE \_\_\_\_\_

**BORING I-A**  
 DRILLED 3-11-59

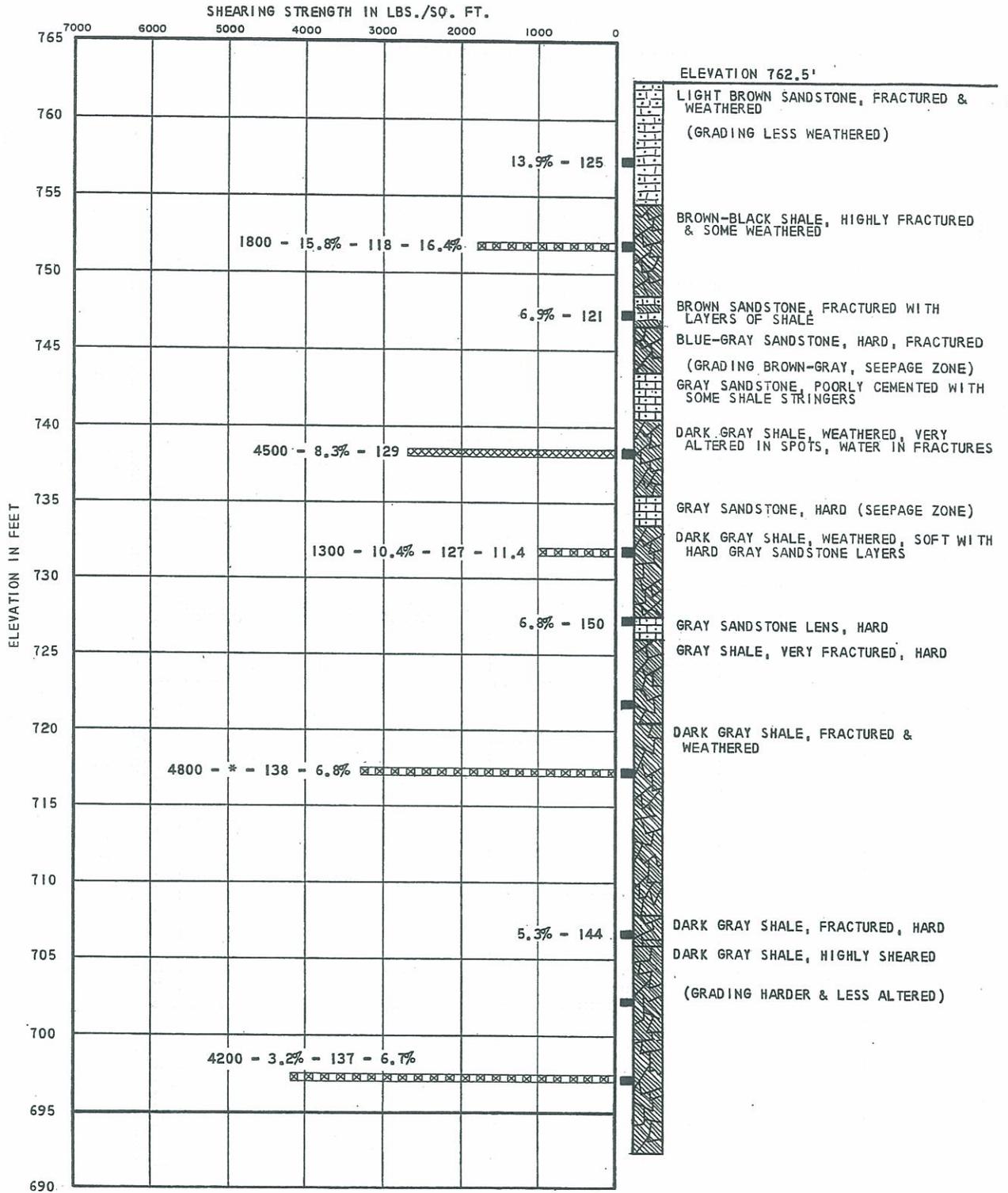


BY: \_\_\_\_\_ DATE: \_\_\_\_\_  
 BY: \_\_\_\_\_ DATE: \_\_\_\_\_  
 PLATE OF \_\_\_\_\_

BY: 26 DATE: \_\_\_\_\_  
 CHECKED BY: 223 DATE: 4/1/57

**LOG OF BORING**

**BORING 2**  
 DRILLED 3-6-59 & 3-9-59

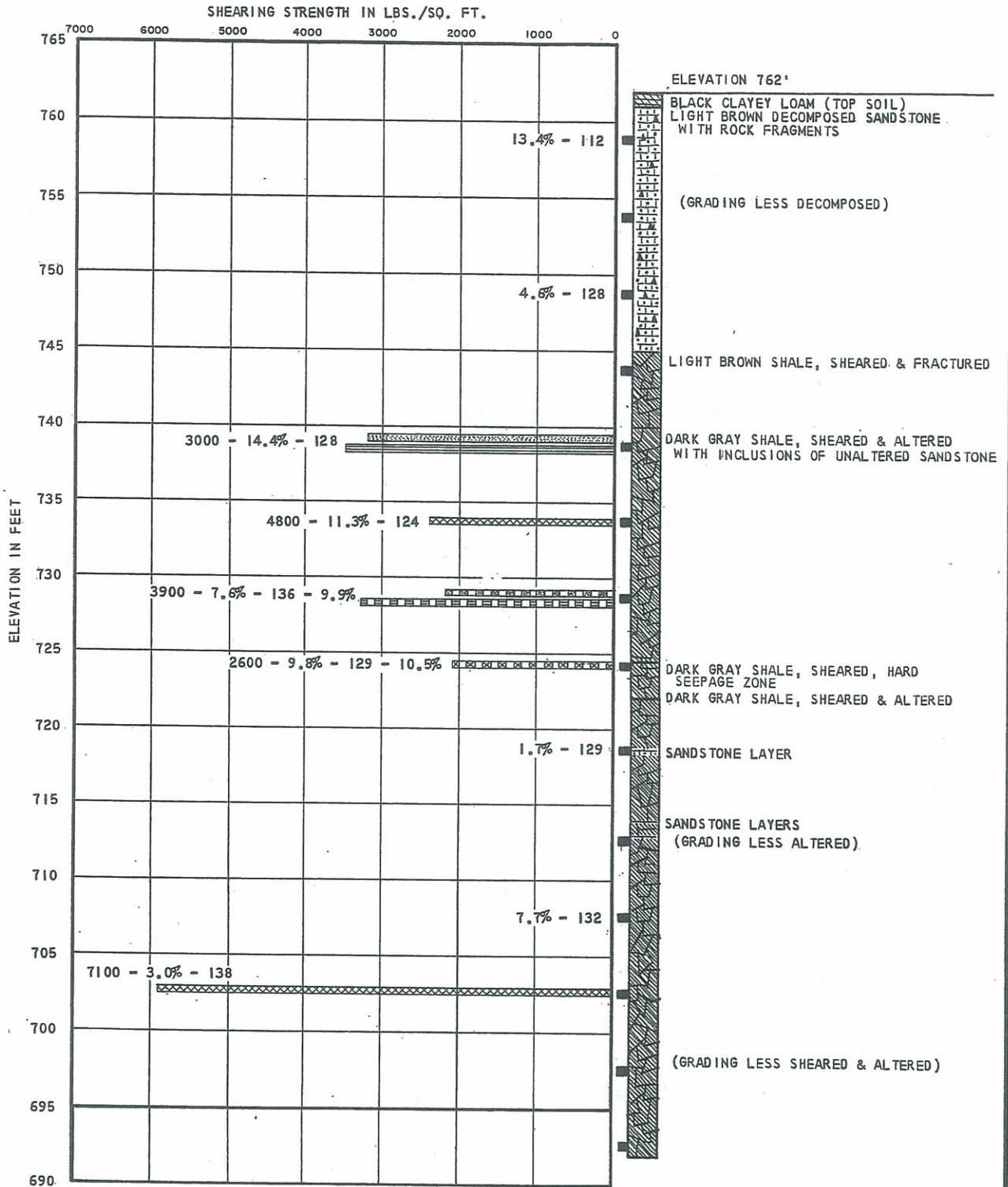


**LOG OF BORING**

BY \_\_\_\_\_ DATE \_\_\_\_\_  
 BY \_\_\_\_\_ DATE \_\_\_\_\_  
 CHECKED BY \_\_\_\_\_ DATE \_\_\_\_\_

BY RC DATE 1/17  
 CHECKED BY CS DATE 1/17

**BORING 3**  
 DRILLED 3-10-59

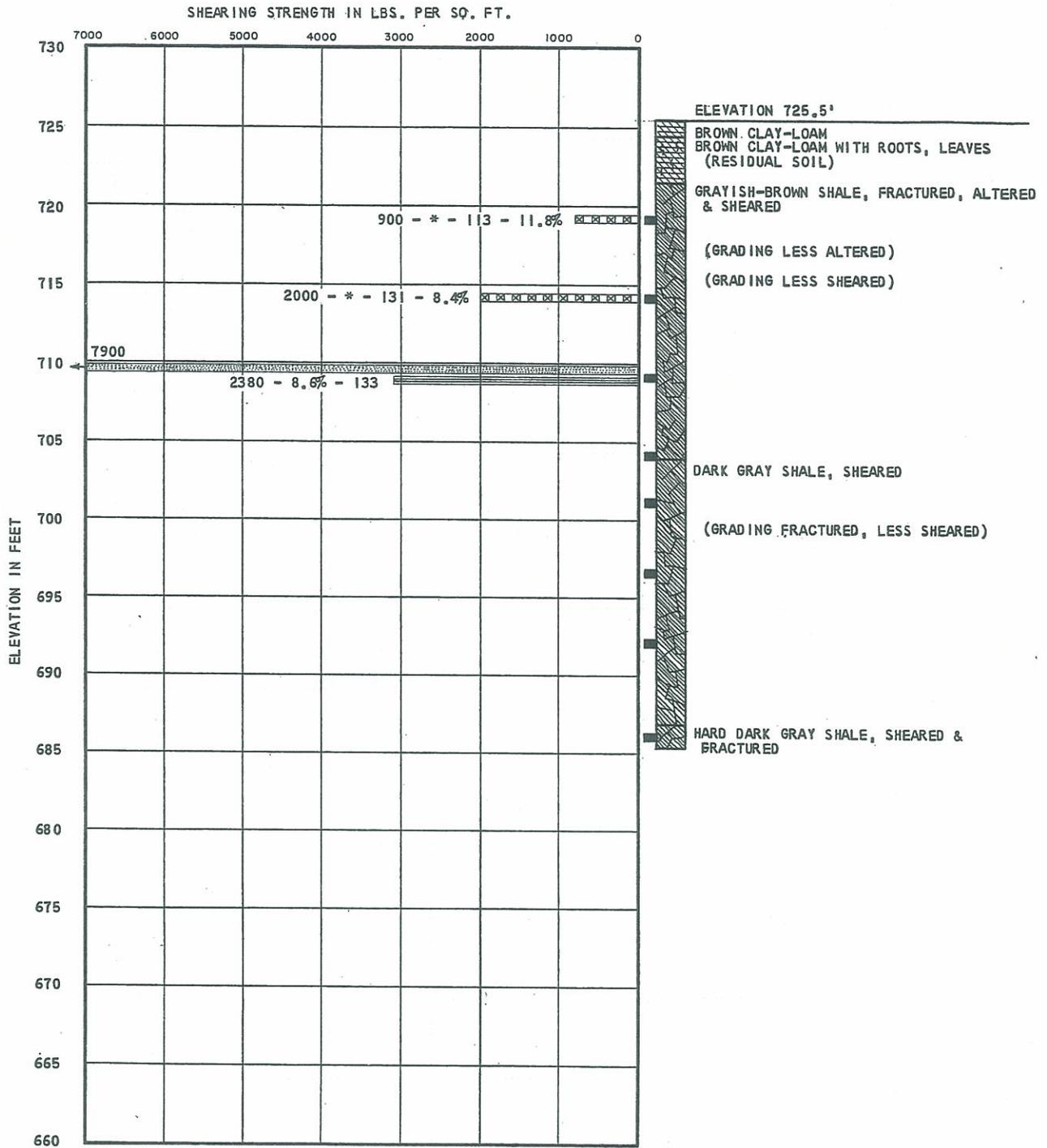


**LOG OF BORING**

BY: \_\_\_\_\_ DATE: \_\_\_\_\_  
 BY: \_\_\_\_\_ DATE: \_\_\_\_\_  
 PLATE: \_\_\_\_\_ OF \_\_\_\_\_

BY: *RG* DATE: *9/1/59*  
 CHECKED BY: *MS* DATE: *9/1/59*

**BORING 4**  
 DRILLED 3-12-59



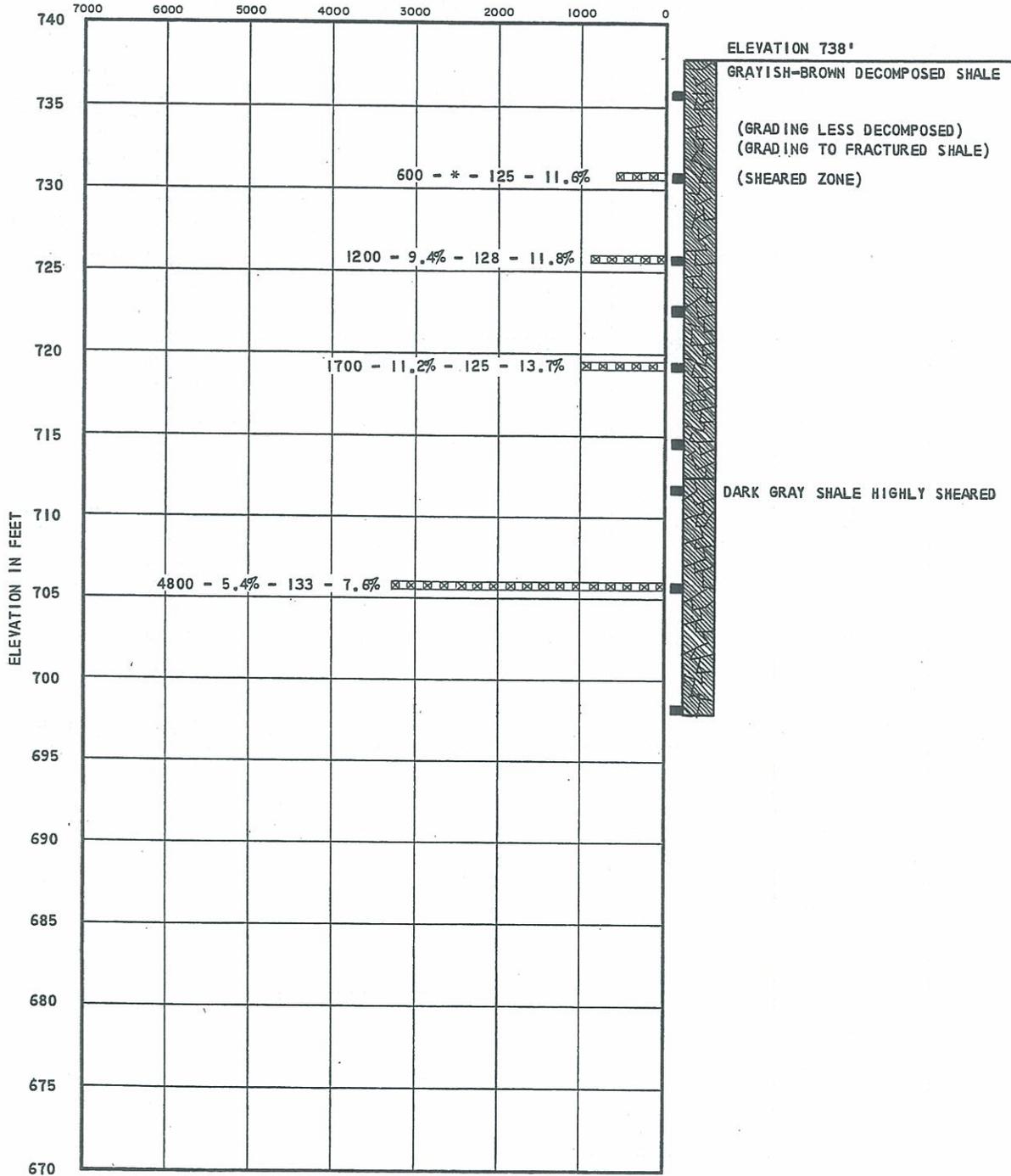
BY \_\_\_\_\_ DATE \_\_\_\_\_  
 BY \_\_\_\_\_ DATE \_\_\_\_\_  
 OF \_\_\_\_\_  
 PLATE \_\_\_\_\_

City of Calif. - Berkeley  
 BY K1 DATE 3-19-59  
 CHECKED BY [Signature] DATE 4/1/59

**LOG OF BORING**

**BORING 5**  
 DRILLED 3-12-59

SHEARING STRENGTH IN LBS. PER SQ. FT.

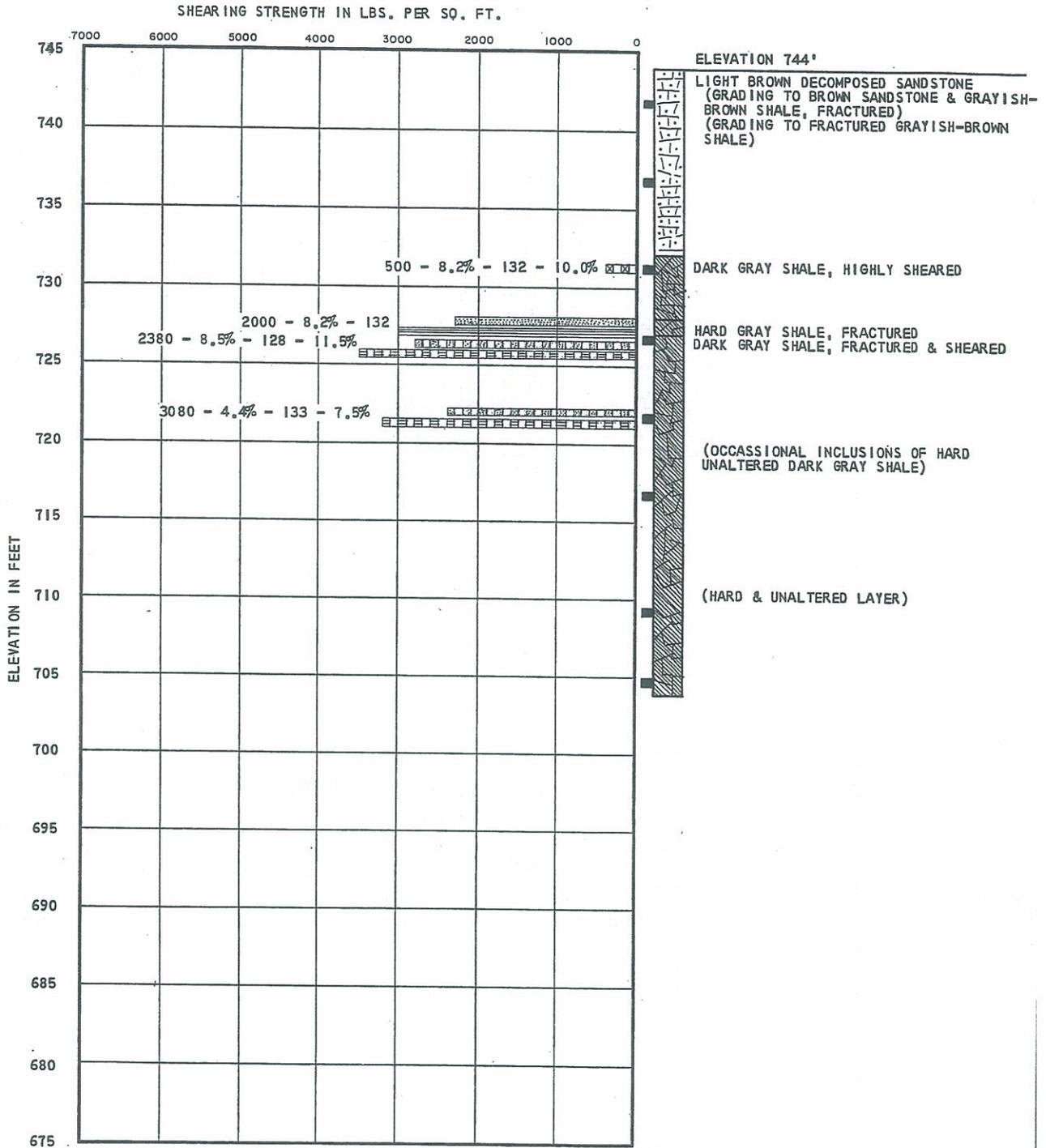


BY \_\_\_\_\_ DATE \_\_\_\_\_  
 BY \_\_\_\_\_ DATE \_\_\_\_\_  
 PLATE \_\_\_\_\_ OF \_\_\_\_\_

BY kl DATE 3-19-59  
 CHECKED BY WCS DATE 1/1/57

**LOG OF BORING**

**BORING 6**  
DRILLED 3-13-59

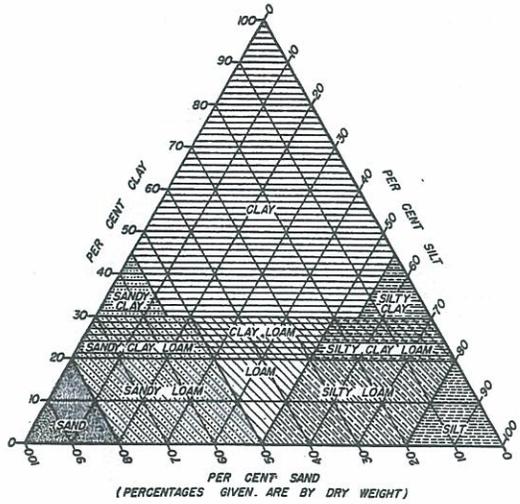


BY \_\_\_\_\_ DATE \_\_\_\_\_  
BY \_\_\_\_\_ DATE \_\_\_\_\_  
OF \_\_\_\_\_  
PLATE \_\_\_\_\_

UNIV. of Calif. - Berkeley  
BY KJ DATE 3-19-59  
CHECKED BY [Signature] DATE 4/1/59

**LOG OF BORING**

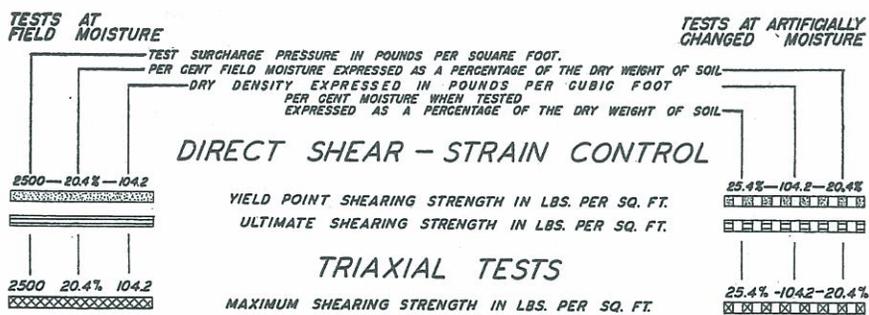
REVISIONS  
 BY \_\_\_\_\_ DATE \_\_\_\_\_  
 BY \_\_\_\_\_ DATE \_\_\_\_\_  
 PLATE \_\_\_\_\_ OF \_\_\_\_\_



SOIL FRACTION	PARTICLE SIZE			
	LOWER LIMIT		UPPER LIMIT	
	MM.	INCHES	MM.	INCHES
CLAY			.005	.0002
SILT	.005	.0002	.05	.002
SAND	FINE	.05	.002	.25
	MEDIUM	.25	.01	.5
	COARSE	.5	.02	2.0
ROCK FRACTION	THESE FRACTIONS ARE NOT CONSIDERED IN DETERMINING SOIL CLASSIFICATION.			
GRAVEL	2.0	.08	64	2.5
COBBLES	64	2.5	256	10
BOULDERS	256	10		

■ INDICATES DEPTH AT WHICH UNDISTURBED SAMPLE WAS EXTRACTED

FILE 234-AE  
 NAME OF CLIENT - BERKELEY  
 BY AT DATE 1-2-50  
 CHECKED BY \_\_\_\_\_ DATE \_\_\_\_\_



SOIL CLASSIFICATION CHART  
 AND  
 KEY TO TEST DATA

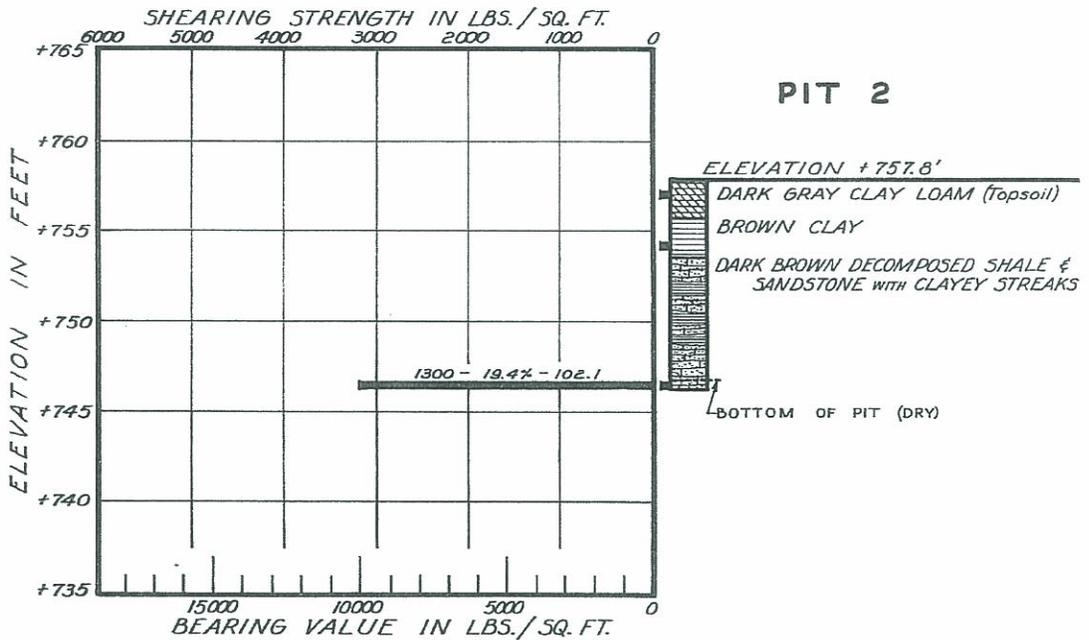
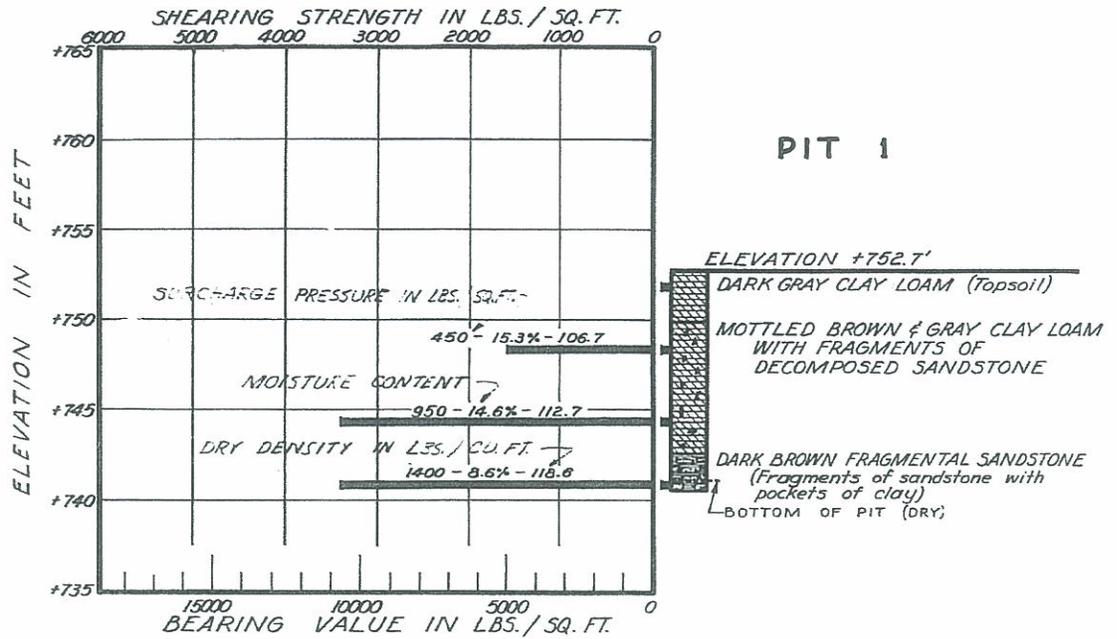
**D&M 1948**  
**Test Pits 1 through 4**

REFERENCE:

Dames & Moore, Site Plan and Test Pit Logs Building 50, dated July 28, 1948

DRAWN BY: KEMPER DATE: 11/22/78  
 DATA CHECK BY: \_\_\_\_\_ DATE: \_\_\_\_\_  
 DRAFTING CHECK BY: \_\_\_\_\_ DATE: \_\_\_\_\_  
 REPORT DICTATED BY: \_\_\_\_\_ DATE: \_\_\_\_\_

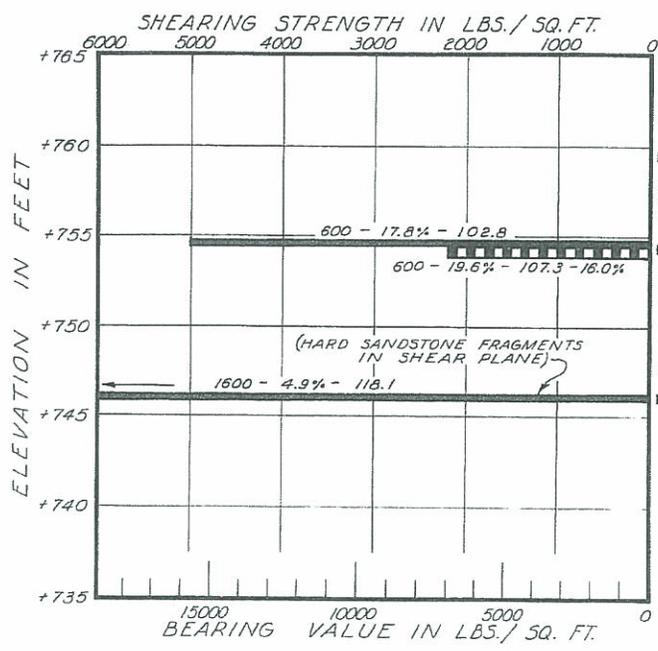
DAMES & MOORE  
 JOB TITLE: FOUNDATION ENGINEERING  
 LOCATION: 1111 W. 11th St. - Tulsa, Okla.  
 CLIENT: OKLAHOMA STATE UNIVERSITY  
 DATE JOB STARTED: \_\_\_\_\_ DATE PRINTED: \_\_\_\_\_  
 THIS DRAWING IS ONE OF A \_\_\_\_\_  
 SERIES OF \_\_\_\_\_ DRAWINGS



## LOG OF PITS

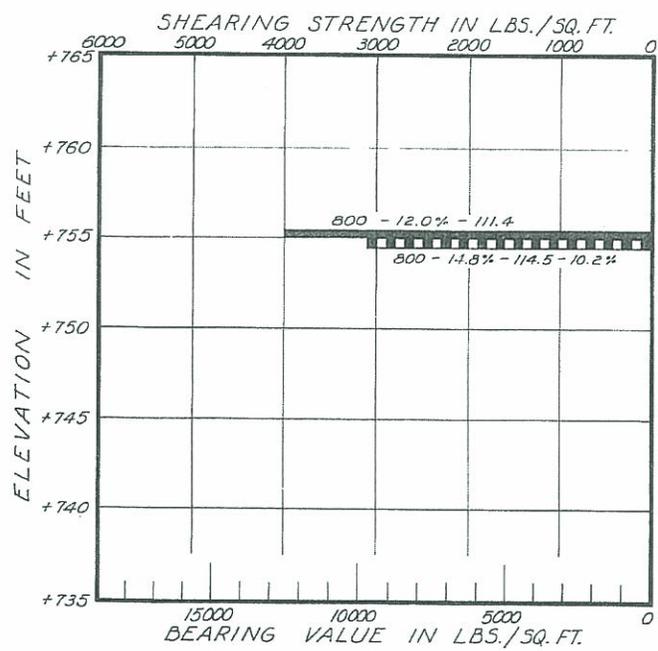
DRAWN BY \_\_\_\_\_ DATE \_\_\_\_\_  
 DATA CHECK BY \_\_\_\_\_ DATE \_\_\_\_\_  
 DRAFTING CHECK BY \_\_\_\_\_ DATE \_\_\_\_\_  
 REPORT DICTATED BY \_\_\_\_\_ DATE \_\_\_\_\_

JOB NO. \_\_\_\_\_ LOCATION \_\_\_\_\_  
 THIS DRAWING IS ONE OF A \_\_\_\_\_ CLIENT \_\_\_\_\_  
 SERIES OF \_\_\_\_\_ DRAWINGS DATE JOB STARTED \_\_\_\_\_ DATE PRINTED \_\_\_\_\_



**PIT 3**

ELEVATION +760.3'  
 DARK GRAY CLAY LOAM (Topsoil)  
 BROWN CLAY  
 MOTTLED BROWN & GRAY DECOMPOSED  
 SHALE & SANDSTONE WITH  
 CLAYEY STREAKS  
 BOTTOM OF PIT (DRY)



**PIT 4**

ELEVATION +762.1'  
 DARK GRAY CLAY LOAM WITH FRAGMENTS  
 OF DECOMPOSED SANDSTONE (Topsoil)  
 BROWN CLAY LOAM WITH  
 FEW ROOTS  
 BROWN & GRAY DECOMPOSED SHALE &  
 SANDSTONE WITH CLAYEY STREAKS  
 BOTTOM OF PIT (DRY)

**LOG OF PITS**