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April 8, 2010
2500-13, L-28886

Ms. Sheree Swanson
Lawrence Berkeley National Laboratory
One Cyclotron Road, MS 56W0100
Berkeley, CA 94720

RE: Geotechnical Investigation Report
Solar Energy Research Center
Lawrence Berkeley National Laboratory
Berkeley, California
(UCB Project 12501A)

Dear Ms. Swanson:

The attached report presents the results of our geotechnical investigation for the proposed Solar Energy Research Center at the Lawrence Berkeley National Laboratory. Our authorization from the University of California, Berkeley refers to this project as Project 12501A.

The accompanying report presents information regarding geologic and geotechnical conditions at the site and presents conclusions and recommendations pertaining to the design of the project. The interpretations, conclusions and recommendations presented in this report were developed in accordance with generally accepted professional principles and practices at the time that the report was prepared. Should you have questions or comments concerning our findings, the geotechnical design concepts discussed, or our recommendations, please do not hesitate to call.

Very truly yours,

ALAN KROPP & ASSOCIATES, INC.

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1.00 INTRODUCTION

This report presents the results of a design-level geotechnical study by Alan Kropp and Associates, Inc. (AKA) for the Solar Energy Research Center (SERC) project at the Lawrence Berkeley National Laboratory (LBNL). The SERC will be situated on the south side of McMillan Road near the eastern limit of LBNL's "Old Town" area; this approximate location is shown on the Vicinity Map (Figure 1).

We obtained information about the proposed SERC project primarily from: (1) Smith Group, the project architects; and (2) discussions with David Bleiman of Rutherford and Chekene (R&C), the project structural engineers. We provided our services in general accordance with our January 29, 2010 proposal under an authorization from the University of California, Berkeley (UCB). Our authorization refers to this project as UCB Project 12501A.

1.01 Project Description

The SERC will be located at the site currently occupied by LBNL Buildings 25A, 44, 44A and 44B, which will be demolished prior to the construction of the project. As currently envisioned, the SERC will be a three-story building with one level (Level 1) mostly below-grade. The approximate footprint of Level 1 is shown on the Site Plan (Figure 2). Levels 2 and 3 each measure about 60 feet by 150 feet, in plan. Where Levels 2 and 3 are not present, the roof of Level 1 will be covered with patio areas.

The topographic contours on Figure 2 generally indicate that surface elevations at the site vary from about +940 feet in the direct vicinity of Building 25A to +930 feet along the site's western margin (east of Building 5). The information provided by Smith Group indicates that Level 1 of the SERC will be at elevation +924 feet. As currently envisioned by R&C, the SERC will be: (1) supported on conventional spread footing foundations; (2) underlain by a drainage system; and (3) surrounded by perimeter retaining walls that are separate from the building structure. Assuming that no major changes are made in the adjacent surface topography, these permanent retaining walls will be up to about 16 feet high.

The proposed site is located within the developed portion of LBNL and is partially surrounded by existing roads and subsurface utilities. We therefore anticipate that some of the perimeter retaining walls will be constructed using top-down methods (i.e., using tiebacks or soil nails) and will also serve to shore the perimeter of the building excavation. In this case, the effective height of the walls during construction will be somewhat greater than the permanent retained height so as to accomplish the additional excavation needed to excavate for the perimeter spread footings and drainage system.

The project will include grading and paving to construct/improve existing adjacent driveways and parking areas, as well as installing subsurface utilities. We further anticipate that various other site retaining walls may be constructed that are not adjacent to the SERC building.

1.02 Geologic Considerations

Geologic conditions in the general area of the SERC site were investigated and characterized in three previous phases of study, which were conducted by AKA in association with William Lettis & Associates, Inc. (WLA), engineering geologists. Our previous phases of study were performed for an adjacent LBNL project known as the General Purpose Laboratory (GPL). As currently envisioned, the GPL will be constructed directly south of the SERC at the location currently occupied by Building 25. These phases included:

- One exploratory boring (AKA-1) located within the footprint of the SERC site (Figure 2);
- Two exploratory borings (AKA-2 and AKA-3) located south of Building 25;
- Two geologic trenches (T-1 and T-2) southwest and southeast of Building 25; and
- Geologic and geotechnical reviews of logs of nearby borings drilled by others.

A stand-alone geologic report was prepared by WLA (WLA 2009), which concluded that the general area of the GPL and SERC sites is suitable for development from a geologic stability (i.e., landsliding) perspective. Some of the geologic descriptions within WLA's 2009 report (prepared under subcontract to AKA) have been included within this geotechnical investigation report. AKA also prepared a design-level geotechnical investigation report for the GPL, which we issued as a draft in December 2009.

The site is not within a previous or current State of California Earthquake Fault Zone, as delineated under the Alquist-Priolo Earthquake Fault Zoning Act (AP Act). The closest active fault is the Hayward fault, which is located about 1,700 feet west of the site as shown on the attached AP Fault Zone Map (Figure 3). The California Geological Survey (CGS) Special Publication 42 (SP42, CGS 1999) provides guidelines for evaluating and mitigating fault rupture hazards within designated Earthquake Fault Zones. Although our overall geologic characterization of the site includes information on various faults in the project vicinity, it was not our objective to investigate or document these features in accordance with the rigorous SP42 guidelines, as the faults in the direct vicinity of the site are not zoned as active.

1.03 Environmental Considerations

In the 1990s, LBNL geologists participated in the preparation of a Resource Conservation and Recovery Act (RCRA) Facility Investigation report, pursuant to the permitting requirements of the California Environmental Protection Agency (Cal-EPA) and Department of Toxic Substances Control (DTSC). The resulting report (LBNL/Parsons 2000), commonly referred to as the "RFI Report", is a public document that includes detailed information pertaining to the geology and hydrogeology of LBNL.

The RFI Report includes maps and data indicating that the SERC site is within an area that is the subject of environmental study. We reviewed portions of the RFI Report to obtain information on geotechnical and geologic conditions in the vicinity of the site; it was *not* our purpose nor was it within our scope to develop opinions, conclusions or recommendations relating to the environmental aspects of the project. However, we judge that the SERC project may include activities that could potentially be affected by environmental constraints (e.g., excavation, dewatering, and offsite soil disposal).

1.04 Purpose and Scope of Services

The primary purpose of our design-level geotechnical study was to evaluate and document the geotechnical conditions present at the proposed SERC site and provide geotechnical conclusions and recommendations for the project. The scope of our services included:

- Using existing subsurface data to characterize subsurface conditions at the proposed site;
- Conducting project-specific geotechnical engineering analyses;
- Developing geotechnical engineering conclusions and recommendations for the project; and
- Preparing this design-level geotechnical investigation report.

As previously noted, the scope of this study did not include evaluating chemical constituents in the onsite soil or groundwater, or providing recommendations pertaining to soil handling/disposal, or other environmental aspects of the proposed work.

2.00 METHODS OF INVESTIGATION

2.01 Review of Geologic References

We reviewed geologic maps, literature, consulting reports and web-based information pertaining to geologic and seismic conditions in the general SERC site area. These references were used primarily to: (1) obtain information relating to the geologic setting of the site; (2) determine site-to-source distances to principal active faults; (3) obtain estimates of earthquake ground shaking; and (4) check for previously mapped fault traces and landslides in the vicinity of the site. A list of selected references is presented in Section 10.00.

2.02 Review of Site-Specific Subsurface Data

We compiled and reviewed site-specific subsurface data from a variety of sources, which were used to evaluate and characterize subsurface conditions at the proposed SERC site. Specific sources of data are introduced in the following subsections. The original source materials used to characterize local geotechnical conditions are attached in Appendices A through C.

2.02.1 Geotechnical and Geologic Data by AKA/WLA

Our previous phases of study included subsurface explorations in the vicinity of the proposed SERC site; the site-specific data and interpretations previously developed were originally presented in the following documents:

- Alan Kropp & Associates, Inc., 2009a, "Summary of Preliminary Investigation and Analyses, Slide Investigation Building 25 Area Project," May 29, 2009 memorandum (Borings AKA-1 through AKA-3).

- William Lettis & Associates, Inc. (WLA), 2009, "Paleolandslide Investigation, Building 25," August 31, 2009 report (Trenches T-1 and T-2).

Borings AKA-1 through AKA-3 were drilled to explore geologic relationships and contacts for the purposes of evaluating geologic stability. All of the borings were drilled in May 2009 by Pitcher Drilling Company of East Palo Alto, California, using rotary wash drilling methods. A WLA geologist supervised the drilling of the three borings, logged the soils and bedrock encountered, and obtained samples of the subsurface materials for subsequent evaluation. In general, samples were obtained using: (1) a 1½-inch inside diameter Standard Penetration Test (SPT) split barrel drive sampler; (2) a 2½-inch inside diameter Modified California split barrel drive sampler equipped with brass liners; and (3) a "101-type" wireline core barrel sampler. Drive samplers were advanced using a standard 140-pound automatic hammer falling 30 inches. The upper portions of the borings were sampled at approximately 5-foot vertical intervals. Zones of geologic interest within the borings were sampled continuously. The field logs and soil/rock samples were reviewed by John Baldwin, C.E.G., WLA's lead engineering geologist for the project, The soils were logged in general accordance with the Unified Soil Classification System (ASTM D-2487-06).

Of these previous AKA/WLA explorations, only Boring AKA-1 is located within the direct vicinity of the SERC site (Figure 2). Boring AKA-1 extended to a depth of approximately 46 feet; core samples from the boring were reexamined by Mr. Baldwin in March 2010 specifically for the purpose of identifying the probable top-of-bedrock elevation at this location. The log of Boring AKA-1 is attached in Appendix A.

2.02.2 Geotechnical and Geologic Reports by Others

We reviewed the following geotechnical and geologic reports by others, which include data from previous subsurface explorations in the direct vicinity of the site:

- Harding Associates (HA), 1963, "Foundation Investigation, Proposed Building 26," (Borings 1 and 2).

- Harding Miller Lawson & Associates (HMLA), 1966, "Progress Report, Soil Investigation, Omnitron Site," (Borings 10 and 11).

- Harding Lawson Associates (HLA), 1983, "Geotechnical Investigation, Building 17 to 25A Road Realignment," March 4, 1983 report (Boring 4.169).

The locations of the above-referenced borings are shown on the Site Plan (Figure 2); the boring logs are presented in Appendix B.

2.02.3 Environmental Boring/Well Logs

As part of our design-level study for the GPL (AKA 2009b), WLA geologists met with Mr. Preston Jordan of LBNL's EH&S Division and obtained copies of environmental boring logs from monitoring well installations in the general vicinity of Buildings 25 and 25A. WLA geologists were also provided access to the EH&S Division's "core library" containing archived samples from the well boreholes. The stated purpose of WLA's review was to: (1) evaluate the quality of the previous boring logs; (2) identify depths to bedrock; and (3) provide a general characterization of any overlying soils that may be present. The results of WLA's core review are documented in a December 10, 2009 letter to AKA, which is attached in Appendix C, together with the environmental boring logs.

According to WLA's December 10, 2009 core review letter, archived samples were reviewed for 11 of the 26 borings for which logs were available. Eight of the 11 borings for which samples were reviewed are in the direct vicinity of the SERC site. The locations of these environmental borings/wells are shown on the Site Plan (Figure 2); the logs of the borings are included in Appendix B following WLA's core review letter.

3.00 GEOLOGIC AND SEISMIC SETTING

3.01 Berkeley Hills Geology

The SERC site is situated on the west side of the northwest-trending Berkeley Hills. Bedrock geology in the Berkeley Hills is complex and includes a variety of moderately to highly deformed (faulted and folded) sedimentary, volcanic and metamorphic rock units.

The Regional Geologic Map presented on Figure 4 is based on a published geologic map of this area from the United States Geological Survey (Graymer 2000). Bedrock units mapped in the general area of the GPL site area include (from oldest to youngest): (1) Great Valley Complex (map symbol Ku); (2) the Orinda Formation (map symbol Tor); and (3) Moraga Volcanics (map symbol Tmb). Information on the relative ages of these units is presented below (Ma = millions of years ago):

Ages of Geologic Units (Jones and Curtis 1991)

Stratigraphic Unit	Period/Epoch	Age
Great Valley Sequence	Upper Jurassic – Lower Cretaceous	159 – 99 Ma
Orinda Formation	Tertiary/Middle to Late Miocene	13.5 – 10.5 Ma
Moraga Formation	Tertiary/Late Miocene	10.2 – 8.4 Ma

During the late Miocene and early Pliocene (11.2 to 3.6 million years ago) an extended period of compression occurred that resulted in the folding, faulting and uplifting of the Berkeley Hills.

3.02 Bay Area Active Faults

The Berkeley Hills are located within the San Francisco Bay Area, a region that is seismically active. This seismic regime is associated with a broad region of deformation near the boundary between the North American and Pacific tectonic plates. Within the region are a series of major active northwest-trending faults, which include the San Andreas, Hayward, Rodgers Creek, Calaveras, San Gregorio, Concord-Green Valley, West Napa, and Greenville faults. These major faults are near-vertical and generally exhibit right-lateral strike-slip movement (which means that the movement is predominantly horizontal and when viewed from one side of the fault, the opposite side of the fault is observed as being displaced to the right).

Various researchers and government agencies have mapped the locations of active and potentially active faults in California. Faults that are defined as active exhibit one or more of the following: (1) evidence of Holocene-age (within about the past 11,000 years) displacement, (2) measurable aseismic fault creep, (3) close proximity to linear concentrations or trends of earthquake epicenters, and (4) prominent tectonic-related aseismic geomorphology. Potentially active faults are defined as those that are not known to be active, but have evidence of Quaternary-age displacement (within about the past 2 million years). The closest known active fault to the project site is the Hayward fault, which has been mapped by the California Division of Mines and Geology (CDMG 1982) approximately 2,000 feet southwest of the site on the UCB Campus (Figure 3). The CDMG is now known as the California Geological Survey (CGS).

The CGS has also developed Active Fault Near-Source Zone Maps to be used in conjunction with previous (pre-2008) versions of the Uniform Building Code (UBC). The Active Fault Near-Source Zone Maps were developed to be used with the UBC to characterize earthquake ground shaking for code-based structural design. The following table shows approximate distances and direction from the site to major Bay Area active faults, based on the CGS/UBC mapping.

Principal Bay Area Active Faults, Site to Seismic Source Distances

Seismic Source	Approximate Distance from Site	Approximate Direction from Site
San Andreas	31 km	Southwest
San Gregorio	33 km	Southwest
Hayward/Rodgers Creek	~ 0.5 km	Southwest
Northern Calaveras	21 km	East
Concord-Green Valley	23 km	Northeast
West Napa	34 km	North

3.03 Regional Seismicity

The San Francisco Bay region is seismically active. Since 1800, five earthquakes of M≥6.5 have occurred in the Bay Area (Bakun 1999). These include the: 1836 M6.5 event east of Monterey Bay; 1838 M6.8 event on the Peninsula section of the San Andreas fault; 1868 M6.8-7.0 Hayward event on the southern Hayward fault; 1906 M7.9 San Francisco earthquake on the San Andreas fault; and 1989 M6.9 Loma Prieta event in the Santa Cruz Mountains.

In 2003, The Working Group on California Earthquake Probabilities (WGCEP 2003), in conjunction with the United States Geological Survey (USGS), published an updated report evaluating the probabilities of significant earthquakes occurring in the Bay Area over the next three decades (2002-2031), which has since been updated on a state-wide scale in 2008 for the time span of 2007-2036. The WGCEP (2008) report indicates that there is a 0.63 (63 percent) probability that at least one magnitude 6.7 or greater earthquake will occur in the San Francisco Bay region before 2037. This probability is an aggregate value that considers seven principal Bay Area fault systems and unknown faults (background values - WGCEP 2003). The findings of the WGCEP (2008) report are summarized in the following table:

WGCEP (2008) Probabilities

Fault System	Probability of At Least One Magnitude 6.7 or Larger Earthquake in 2007-2036
Hayward/Rodgers Creek	0.31
San Andreas	0.21
Calaveras	0.07
San Gregorio	0.07
Concord-Green Valley	0.03
Greenville	0.03
Mount Diablo Thrust	0.01
Background (2002-2031)	0.14*

The published background values are not explicitly stated in the WGCEP (2008) and thus the WGCEP (2003) values were used. The background values indicate that between 2002 and 2031 there is a 14 percent chance that an earthquake with a magnitude of greater than 6.7 may occur in the Bay Area on a fault system not characterized in the study. It should be noted differences between the 2008 and 2003 WGCEP generally fall within the magnitude of error, and major differences in background values are not expected.

4.00 LOCAL GEOLOGIC CONDITIONS

4.01 Topography and Geomorphology

The proposed SERC site is located along the west side of a north/south-trending ridge that divides two natural watersheds. Early maps (Lawson and Palache 1901) and historical photographs (1935 and 1939) of the site area depict pre-development morphology, and show what appears to be a gently sloping and rounded north/south-trending bedrock ridge with southwest-trending and southeast-trending bedrock spurs. These early depictions show the ridge continuing northward and steepening toward the present-day Lawrence Hall of Science. Highly resistant andesitic and basalt units of the Moraga Formation form the steep ridges and cliff faces upslope of the site (north of McMillan Road).

Areas east of the site are within the upper reaches of Chicken Creek Canyon and generally drain south towards Strawberry Creek. West of the site is a broad surface that drains northwest toward Blackberry Creek. The pre-existing natural topography within this area of LBNL has been altered by grading; most notably by cutting performed to construct the pads for Buildings 25 and 25A and to realign McMillan Road.

4.02 Local Bedrock Units

4.02.1 Great Valley Group

Great Valley Group rocks are not present near the ground surface within the direct vicinity of the SERC site. Regional mapping (Figure 4) shows the closest near-surface Great Valley Group rocks about 600 feet south and southwest of the site. The contact between the Great Valley Group and overlying Orinda Formation generally trends northwest-southeast and dips back into the slope (i.e., down towards the northeast). The curvilinear geometry of this contact (Figure 4) generally reflects the intersection of this approximately planar northeast-dipping fault surface with the surface topography. Although not encountered in boreholes in the Building 25 area, the Great Valley Group is present beneath the site at considerable depth and would lie below the Orinda Formation.

4.02.2 Orinda Formation

The Orinda Formation is a non-marine sedimentary formation composed of well- to poorly-bedded siltstone, claystone, fine lithic sandstone, and pebble conglomerate. Regionally, the Orinda Formation can be up to about 2,400 feet thick. The conglomerate contains a high percentage of detritus derived from the Franciscan Assemblage (Graymer, et al. 1996). The coarse-grained conglomerate was deposited under alluvial fan conditions, while the sandstone, mudstone and finer-grained conglomerate were deposited as flood plain and channel material (Jones and Curtis 1992). In the vicinity of the GPL site, Lawson and Palache (1901) and Lawson (1900) interpret the sedimentary lacustrine deposits as interfingering with volcanic deposits of the Moraga Formation at or near the interplay between eruptive and fluvial deposition.

Locally, the Orinda Formation consists of well-bedded to massive siltstone and claystone and occasional lenses of coarse-grained sandstone and conglomerate. The top of bedrock commonly consists of fluid-stained, yellowish/reddish brown, weathered, silty sandstone to siltstone. With increasing depth, the Orinda Formation gradually becomes less weathered and maintains coloration more indicative of the deposit, such as, greenish-gray, gray, olive gray and grayish-blue colors with discontinuous lenses of sand and gravel lenses.

4.02.3 Moraga Formation

The Miocene Moraga Formation consists of as many as five distinct flows typically defined by basaltic and andesitic composition (Wahrhaftig and Sloan 1989). The maximum thickness of the Moraga Formation in the vicinity of LBNL is estimated at about 800 to 1000 feet (Lawson and Palache 1901). The source vent is believed to be the Round Top volcanic complex, which is located several miles southeast of LBNL. Early studies by Lawson and Palache (1901) refer to the volcanic deposits in the project vicinity as the Campan series and are described as fresh-water deposits interbedded with lavas and tuffs. Others also describe similar clastic deposits at or near the base of eruptive sequences (Lawson 1901; Wahrhaftig and Sloan 1989; and Clements 1963). The Moraga Formation is exposed in multiple cut slopes along Centennial Drive and Grizzly Peak Boulevard north of the site and in several abandoned quarries at LBNL (e.g., northwest of Building 78 and southwest of Building 83). Moraga Formation also is exposed in the Lawrence Road cut south and downslope of Building 25.

In the site area, the Moraga Formation consists of andesitic and basaltic flows, tuff, and volcaniclastics (locally described as agglomerate or volcanic breccia). Potassium-argon ages of the volcanic flows vary from 10.2 million years (Ma) to 8.4 Ma (Curtis 1989). The basal member of the volcanics, defined as an amygdaloidal andesite is interpreted to have been deposited over a broad alluvial flood plain with later flows and tuffs being confined to narrow channels, ravines and valleys (Lawson and Palache 1901; Wahrhaftig and Sloan 1989). Locally, the Moraga Formation rests depositionally on the Orinda Formation, or is present as intact translated slide bodies (LBNL/Parsons 2000). At the site, the Moraga Formation typically consists of highly fractured and weathered, subangular to subrounded cobble to large, boulder-size agglomerate to andesite with a predominantly silty to fine gravelly matrix. Minor amounts of weathered and altered ash and tuff also are present.

4.03 <u>Local Faults</u>

The map presented on Figure 4 (Graymer 2000) interprets faulting in this general area of LBNL as complex, with: (1) the Orinda Formation in fault contact with the older Great Valley Group south and southwest of the site; (2) two north-striking faults in Chicken Creek Canyon east of the site; and (3) several northwest-striking faults in the vicinity of Old Town, including one that is proximate to the site.

The location of the Orinda-Great Valley contact is reasonably well-established within the lower Chicken Creek Canyon area (in the vicinity of LBNL Building 31) and west of the Advanced Light Source (LBNL Building 6) based on borehole data. Great Valley rocks near the fault zone are described as a dark gray to black, sheared and brecciated fine-grained shale and siltstone (Jordan & Javandel 2007). Orinda-Great Valley contact juxtaposes rocks of vastly different ages (younger than 13.5 million years versus older than about 100 million years, respectively). From a geologic perspective, the Orinda-Great Valley contact is of structural significance, but it is not considered an active fault.

Various consultants and researchers (e.g., HLA 1982, Graymer 2000, LBNL/Parsons 2000, WLA 2009) have interpreted faults in the vicinity of the SERC site; however, none of the faults that have been mapped locally are zoned as active (Figure 3). To our knowledge, no site-specific fault investigations (i.e., trenching studies) have been performed in the vicinity of the site to identify and constrain the locations of any faults that may be present. Notably, the locations of faults shown on previous published and unpublished maps vary considerably. Based on this observation, we consider it likely that some of the faults do not exist at the locations shown on previous maps.

4.04 Landslides

Maps prepared by Nilsen (1975) and HLA (1982) show no landslides in the general vicinity of the SERC site. The landslide inventory map prepared by the CGS (2003) generally shows landslides east and west of the site, but no landslides within the ridge area occupied by Buildings 25 and 25A. In general, the maps prepared by researchers and consultants prior to 2000 generally show the SERC site area as underlain by bedrock unaffected by large-scale landsliding.

The RFI report (LBNL/Parsons 2000) postulates the existence of a number of large-scale "paleolandslides" at LBNL, all of which are mapped as lying topographically below the belt of in-place Moraga Formation rock that generally caps the Berkeley Hills. In the RFI Report, the Moraga Formation rock that underlies B25 is interpreted to be a paleolandslide overlying in-place Orinda Formation bedrock. However, more recent data developed by WLA (2009) indicates that: (1) the B25 ridge area has been stable for perhaps tens of thousands of years; and (2) Moraga formation rocks are in depositional contact with underlying Orinda Formation rocks southeast of Building 25. These new data have been used by WLA (2009) to generally refute the paleolandslide model of LBNL/Parsons (2000), as discussed further in Section 6.03.

5.00 GEOTECHNICAL SITE CONDITIONS

5.01 Surface Conditions

The SERC site is located in an area of LBNL that was originally developed during the 1940s and 1950s. As shown on Figure 2, the project site is currently occupied by Building 25A, a two-story structure that measures approximately 50 feet by 120 feet, in plan. Three smaller single-story buildings are situated within the site west and northwest of Building 25A. Buildings 44A and B share a common parking lot with Building 25A and have ground floors near the same elevation (about +940 feet). Building 44 is located directly west of Building 44B at the approximate elevation of the access road that bounds the western side of the SERC building site (near Elevation +930 feet). All four buildings will be demolished prior to the construction of the planned SERC improvements.

Building 25 currently occupies the site directly south of Building 25A and, as currently planned, will be demolished in order to construct the adjacent GPL project. As noted earlier, previous investigations have generally encountered bedrock near the ground surface near the north end of Building 25. Based on this observation, we consider it likely that all of the existing buildings at the site are supported on conventional shallow foundations. As currently planned, the future GPL will also be supported on conventional shallow foundations (spread footings), near the level of the existing grade.

The existing access road that bounds the western side of the SERC building site is paved with asphalt concrete (AC) and is only about 12 feet wide where it passes west of Building 44. Portions of the road exhibit longitudinal cracking roughly parallel to the outboard (western) edge of the road. Directly west of the road, the ground surface slopes steeply downward towards Building 5; within this sloping area landscaping, retaining walls, and various utilities were observed.

5.02 Subsurface Conditions

5.02.1 Generalized Description of Subsurface Conditions

The planned SERC building site is underlain by a relatively thin (0 to about 10-foot-thick) mantle of soil over bedrock. The surface of bedrock is highest in the general area of Building 25A, where it was encountered in environmental borings within a foot or two of the ground surface (i.e., above +938 feet). Bedrock is generally deeper towards the northwest corner of the site where HMLA's Boring 10 encountered bedrock at a depth of about 5 feet (Elevation +922 feet). In Boring AKA-1 the bedrock surface is interpreted to be shallower than 10.5 feet (above Elevation +929 feet), the depth at which continuous coring and geologic logging was initiated. Two adjacent environmental borings for which samples were not available (MW25A-95-4 and MW25A-98-3) reportedly encountered Moraga Formation bedrock within about a foot of the ground surface (near Elevation +939 feet). These differing interpretations could not be resolved due to the absence of samples. However, it is possible that: (1) the bedrock surface at the site could include a near-vertical step (or steps) caused by erosional or tectonic features; and/or (2) the natural soils at the site and the bedrock materials from which they were derived are at times indistinguishable. Figure 5 presents an interpretive map of bedrock surface contours, which we developed based on the subsurface data from AKA-1 (Appendix A), previous geotechnical investigations by others (Appendix B) and the WLA-reviewed environmental borings (Appendix C).

The subsurface conditions encountered in the borings are described briefly below.

Bedrock – The bedrock encountered in previous borings drilled in the vicinity of the site consists of both Orinda Formation (including siltstone, sandstone, claystone and conglomerate) and Moraga Volcanics (including andesite and tuff or agglomerate breccias). The bedrock layers that exist within the Moraga Formation are variable and include some materials that are harder and more resistant than the Orinda Formation. The logs of borings presented in Appendices A through C frequently show Standard Penetration Test (SPT) blow counts of 50 blows for 6 inches or less of penetration within Moraga Formation materials. SPT blow counts between about 20 and 50 blows per foot are commonly reported within Orinda Formation materials.

Colluvium – Colluvium is a geologic term that refers to soil that has moved downslope by gravity. The data we reviewed indicates that colluvium is thickest in the northwestern portion of the site. The colluvium encountered in HMLA Boring 10 (1966) and HLA Boring 4.169 (1983) extends to elevations of 921 and 917 feet, respectively, at which point Orinda Formation bedrock was encountered.

Fill – The borings drilled in the SERC site area did not encounter appreciable amounts of fill. However, any fill materials present at the site are considered *undocumented* in that no records have been found to show that the fill was placed in accordance with geotechnical engineering recommendations.

Groundwater – The boring logs presented in Appendices B and C provide limited data on groundwater conditions in the area of the planned SERC site. On March 9, 1992, a high groundwater elevation measurement of about +923 feet was obtained during the drilling of an environmental monitoring well east of the SERC building site (MW26-92-11). The RFI Report includes: (1) an interpreted water level elevation contour map (RFI Report Figure B2.4-2) showing groundwater at the site between Elevation +900 and +920 feet; and (2) an interpretive cross section (RFI Report Figure B2.3-4) showing groundwater levels measured in wells at the site in September 1999 ranging from about +905 to +915 feet.

Summaries of the data and related information used to develop the above generalized descriptions follow.

5.02.2 Summary of Bedrock Depth/Elevation Data

The bedrock depth/elevation data used to develop Figure 5 are summarized in the tables that follow. The original source materials (boring logs) on which the tabular summaries are based are presented in Appendices A through C.

Bedrock Depth/Elevation Data from Geotechnical and Geologic Reports

Source of Data	Boring No.	Surface Elevation (feet)	Depth to Bedrock (feet)	Elevation of Bedrock (feet)	Top of Bedrock Description
AKA (2009)	AKA-1	939.7	<10.5	>929	Orinda (siltstone)
HLA (1983)	4.169	945	28	917	Moraga (andesite)
HMLA (1966)	10	927	6	921	Orinda (siltstone)
HMLA (1966)	11	930	1.5	928.5	Orinda (siltstone)
HA (1963)	1	937.6	17.5	920.1	Orinda (siltstone)
HA (1963)	2	936.5	18	918.5	Orinda (siltstone)

Bedrock Depth/Elevation Data from WLA-Reviewed Environmental Boring Logs

Boring No.	Surface Elevation (feet)	Depth to Bedrock (feet)	Elevation of Bedrock (feet)	Top of Bedrock Description
MW25A-98-1	934.70	~2	932.7	Orinda (silty sandstone)
MW25A-98-6	940.70	~2	938.7	Moraga (tuff breccia)
MW25A-98-7	940.10	~2.5	937.6	Orinda (siltstone)
MW25A-99-2	940.69	~1.5	939.2	Orinda (siltstone)
MW25A-99-5	940.60	~1	939.6	Orinda (silty sandstone)
SB25A-96-2	937.29	~1.5	935.8	Orinda (silty sandstone)
SB25A-96-3	939	~2	937	Orinda (tuffaceous siltstone)
MW26-92-11	941	~8.5	932.5	Orinda (sandy claystone)

5.02.3 Summary of Groundwater Data

Borings AKA-1 and AKA-2 were drilled using rotary wash drilling methods in which borehole stability is maintained through the use of drilling fluids; this technique generally precludes the accurate measurement of natural groundwater levels either during or immediately after drilling. Information on the depth and elevation of groundwater reported in previous onsite borings by others is presented in the following table.

Groundwater Depth/Elevation Data

Source of Data	Surface Elevation (feet)	Depth to Groundwater (feet)	Elevation of Groundwater (feet)	Date of Groundwater Measurement
HA (1966) 11	930	23	907	8/30/66
MW26-92-11	~941	~18	~923	3/9/92

5.02.4 Summary of Plasticity Test Data

The reports that we reviewed included the results of plasticity (Atterberg Limits) tests performed on samples of soil and bedrock materials from the general vicinity of the SERC site. The results of laboratory Atterberg Limits determinations performed previously by others are summarized in the following table.

Atterberg Limits Test Results

Source of Data	Boring No.	Approximate Depth of Sample (feet)	Liquid Limit	Plasticity Index	Material (Soil Classification)
HLA (1988)	1	2.5	37	14	(ML)
HMLA (1966)	12	30.4	31	10	Siltstone (CL-ML)
HMLA (1966)	12	47.6	33	13	Siltstone (CL-ML)
HMLA (1966)	13	31.9	43	13	Basalt (ML)

Onsite soils having a PI of 15 or less are generally considered to have a sufficiently low expansion potential to be used as non-expansive fill. The borings referenced in the above table are not shown on Figure 2 as they fall outside of the immediate SERC Study Area.

6.00 GEOLOGIC AND GEOTECHNICAL HAZARDS

We conclude that the proposed project is generally feasible from a geologic and geotechnical hazard standpoint provided that the geotechnical recommendations outlined in this report are appropriately addressed in the project design. The principal geologic and geotechnical hazard considerations for the project are discussed in the sections that follow.

6.01 Ground Shaking

The San Francisco Bay Area is seismically active and strong ground shaking is likely to occur at the site within the life of the project as a result of future earthquakes. For this reason, structures at the site should be designed to resist strong ground shaking in accordance with the requirements of the California Building Code (CBC) and local design practice.

6.01.1 CBC Site Class

The seismic design provisions of the 2007 CBC include a methodology by which sites are classified as A through F in order to quantify site-specific ground shaking effects. As previously discussed, the proposed SERC building will have one level (Level 1) that is mostly below-grade with a floor elevation of +924 feet. The building's foundations will be below this elevation and will be founded mostly on bedrock. Based on this interpretation, we judge a Class C designation (very dense soil and soft rock) to be appropriate for the design of the SERC building.

The current version of LBNL Lateral Force Design Criteria RD 3.22 (Revision 12, dated 6/09/2009) states that "seismic analyses will utilize the static lateral force procedures of the CBC unless a dynamic analysis is necessary." Location-specific seismic design parameters for use with the 2007 California Building Code (CBC) are presented in Section 8.02.1 of this report.

6.01.2 Campus Ground Motions

The seismic design provisions of the 2007 CBC also allow the use of earthquake ground motions developed through a site-specific probabilistic seismic hazard assessment (PSHA). PSHA-derived design ground motions (response spectra and time histories) have been developed for the LBNL campus by URS Corporation (URS). The latest ground motions developed by URS utilize "next generation" attenuation models (NGA models), which can predict lower levels of ground shaking than previous PSHA models. Use of the URS-derived earthquake ground motions (time histories) is mandated by LBNL Lateral Force Design Criteria RD 3.22 in instances where a dynamic analysis is necessary. Recommendations concerning the application of the campus ground motions developed by URS are included in Section 8.02.2.

6.02 Surface Fault Rupture

Historically, earthquake fault rupture most often occurs along pre-existing active faults. The SERC site is not located within an AP Zone (Figure 3), and the references that we reviewed indicate the closest mapped active fault is about 2,000 feet away. Based on this information, we consider the overall risk of surface fault rupture at the site to be low.

6.03 Geologic Stability

Our previous phases of geologic study (AKA 2009 and WLA 2009) were performed to check the paleolandslide hypothesis of LBNL/Parsons (2000), as it relates to this site. Based on a detailed review of existing information, geologic field reconnaissance and mapping, three test borings (AKA-1 through AKA-3) and two exploratory trenches (T-1 and T-2), WLA concludes that: (1) the trench and geomorphic data provide evidence for long-term (thousands of years) stability of the bedrock ridge upon which the site is located; and (2) if a paleolandslide exists beneath the site it is geologically stable.

6.04 Inundation

The site is located in the hills upslope of the city of Berkeley above Elevation +900 feet; inundation by tsunami or seiche is therefore not a concern. To our knowledge, there are no dams or large reservoirs upslope of the site that could pose an inundation hazard to the SERC facility. In our opinion, there is essentially no risk of significant inundation at the planned SERC site.

6.05 Liquefaction and Densification

The soils generally considered most susceptible to liquefaction are saturated (i.e., below groundwater) clean sands, silts and gravels having little or no cohesion. Densification can occur where these types of low-cohesion soils are above groundwater. Current and ongoing research has demonstrated that cohesive silts and clays of low plasticity can also exhibit seismic strength degradation behavior that is in some ways similar to liquefaction. The range of conditions over which this behavior occurs is the subject of continuing research. There appears to be general agreement that soils having a PI of 7 or less are susceptible to earthquake-induced strength loss, whereas soils having a PI of 18 or greater are not.

In general, the SERC site is underlain by bedrock and soils of moderate plasticity that are not submerged. Consequently, we judge there to be little to no potential for seismically-induced liquefaction or densification at the planned SERC site.

7.00 GEOTECHNICAL EVALUATIONS AND CONCLUSIONS

7.01 Foundations

Based on the available data, it appears that most of the planned SERC building can be founded on spread footing foundations that bear directly upon bedrock. This conclusion is generally based on: (1) the bedrock surface contour elevations shown on Figure 5; (2) our current understanding that Level 1 of the SERC building will have a floor elevation of Elevation +924 feet; and (3) the assumption that the bedrock materials at the foundation level are non-expansive (expansive materials are discussed further in Section 7.04).

The interpreted bedrock surface contours shown on Figure 5 generally indicate that in the northwest corner of the site, footings at standard depths (i.e., about 18 inches below the floor slab) will not be directly underlain by bedrock. This condition may also occur in other areas of the site, due to natural variations in the bedrock surface. Where suitable bedrock is not exposed at planned footing depths, the footings can be deepened to bear directly upon rock; or can be underlain by lean mix concrete that bears directly on rock.

The borehole data and previous geologic mapping indicates that the bedrock upon which the SERC building will be founded is likely to be variable. For this reason, we recommend in this report that all foundations for the SERC building be interconnected in order to span localized irregularities.

Post-construction static settlement of properly constructed spread footings under allowable loads should be very small; less than about ½ inch. Recommendations for the design and construction of spread footings are presented in Section 8.03. If other types of foundation systems are to be considered (e.g., drilled piers, footings on improved ground, or tiedowns to resist uplift), we should be contacted to provide supplemental geotechnical recommendations.

7.02 Retaining Wall at Building Perimeter

As currently envisioned, a permanent retaining wall will be constructed at the building perimeter prior to the construction of the SERC building. Since the building is located in an area where adjacent surface and subsurface improvements (e.g., utilities) are present, we anticipate that at least a portion of the perimeter retaining wall will be installed using top-down construction methods. This approach is seen as advantageous from a number of perspectives, including: (1) the wall installation methods will minimally disturb the adjacent improvements; (2) the permanent wall can also serve as shoring for the SERC building excavation; and (3) the completed wall will be free to deflect under applied seismic earth loads without affecting the SERC building structure. Top-down construction can be accomplished using either of the following methods:

- Tied-back retaining walls using post-tensioned ground anchors (tiebacks); and
- "Soil nail"-type retaining walls in which the ground anchors are not post-tensioned.

We consider either method to be generally appropriate, from a geotechnical standpoint, provided that there is adequate space and right-of-way behind the wall for permanent tiebacks or soil nails.

Tieback retaining walls are commonly designed by either the project Civil or Structural Engineer, with the tiebacks themselves being designed by a specialty drilling contractor based on: (1) lateral loads and "no-load" zone restrictions provided by the project Geotechnical Engineer; and (2) tieback performance requirements (load-deflection characteristics and corrosion protection) provided by the project Civil or Structural Engineer. Tieback walls may consist of: (1) soldier piles with concrete lagging; (2) soldier piles with reinforced shotcrete lagging; or (3) reinforced shotcrete bearing on-grade.

Soil nail wall design can be provided by either the project Civil or Structural engineer or by a contractor retaining the services of a shoring engineer. Unlike tieback walls, soil nail walls are typically designed based on material properties provided by the Geotechnical Engineer (cohesion, friction angle, unit weight) using specialized software incorporating slope stability-type analysis methods. In our experience with retaining walls supporting excavations, soil nail walls are generally feasible where wall heights exceed about 10 feet.

The load-deformation behavior and capacity of installed tiebacks and soil nails need to be confirmed by load testing.

7.03 Design Considerations Relating to Groundwater

As discussed in Section 5.02.1, a high groundwater elevation measurement of about +923 feet was obtained during the drilling of an environmental monitoring well east of the SERC building site (MW26-92-11) in March 1992. Interpretive cross sections presented in the RFI Report show that groundwater levels measured at the site in September 1999 varied from about +905 to +915 feet. As currently envisioned, the top of the SERC building Level 1 floor slab will be at +924 feet.

The available groundwater measurements generally indicate that water levels at the site vary significantly on a seasonal basis, with "high" groundwater levels approaching the level of the proposed Level 1 floor slab. However, it should be anticipated that: (1) natural groundwater levels at the site may periodically rise above the planned elevation of the Level 1 floor slab; and (2) water may also be present at higher elevations under localized "perched" conditions.

As currently planned, the SERC building Level 1 floor slab and the perimeter retaining wall that surrounds the building will be fully drained in order to prevent the build-up of hydrostatic pressures. If this is the case, it will not be necessary to design the Level 1 floor slab to resist hydrostatic uplift. However, it will be essential that retaining wall backdrainage and slab underdrainage systems: (1) are appropriately designed, detailed, and installed; and (2) are capable of functioning continuously, when needed, over the design life of the SERC building.

In this report, we recommend that all below-grade drainage systems be readily maintainable and be designed to flow *by gravity* to an appropriate discharge and that a waterproofing consultant be retained to provide any additional recommendations needed pertaining to the waterproofing of retaining walls or below-grade portions of the SERC building. Geotechnical recommendations for the design of retaining wall backdrainage and slab underdrainage systems are presented in Section 8.05 and Section 8.10.1, respectively.

7.04 Onsite Expansive Materials

Based on the plasticity test data from nearby sites, it appears that appreciable quantities of expansive soil and/or bedrock may not be present at the SERC site (expansive materials tend to shrink and swell with changes in moisture content and therefore have the potential to damage improvements that are supported directly upon them unless appropriately mitigated). However, as a precaution, it will be necessary for AKA to observe and test, as appropriate, the following materials to check their expansion potential: (1) soils and bedrock exposed in near-surface site excavations; (2) onsite materials proposed to be re-used as engineered fill; and (3) proposed import fill materials. Geotechnical recommendations pertaining to earthwork and expansive soils are presented in Section 8.09.

7.05 Construction Considerations

7.05.1 Excavation and Shoring

We anticipate that most of the near-surface soil and bedrock at the site can be excavated with conventional earth-moving equipment. However, it is possible that locally harder bedrock could be encountered that would require jack-hammering or hoe-ramming to excavate, particularly in confined areas such as footing excavations. In addition, the existing materials at the site may include subsurface obstructions. Removal of buried obstructions could require equipment capable of breaking concrete.

As currently envisioned, the SERC building will be surrounded by permanent retaining walls that will obviate the need for temporary shoring at the building excavation perimeter. All excavations deeper than 4 feet that will be entered by workers will need to be shored or sloped for safety in accordance with the applicable: (1) California Occupational Safety and Health Administration (Cal-OSHA) standards; and (2) site-specific health and safety protocols and procedures required by LBNL.

7.05.2 Dewatering and Soil Moisture

The available data suggest that groundwater at the site is deeper than anticipated site excavations over a portion of the year. However, groundwater may at times rise to above the level of the excavation bottom and may also be locally present at higher elevations.

The control of groundwater during construction is the responsibility of the contractor. The contractor should anticipate that site excavations may need to be dewatered and depending upon the water source (i.e., surface versus subsurface), there may be environmental aspects to the appropriate collection, storage and disposal of onsite water. The design, permitting, installation, monitoring, and abandonment of site dewatering and discharge systems are the contractor's responsibility.

The onsite soils may include materials that are wet of optimum, from an earthwork compaction standpoint. Deeper excavations at the site could encounter soils that are saturated. The contractor should anticipate that soils obtained from site excavations will include clayey soil and clayey rock materials that will need to be processed (e.g., by air drying) prior to being placed as engineered fill.

7.05.3 Construction Monitoring

The contractor's responsibilities should include: (1) documenting the condition of the adjacent improvements prior to the commencement of site demolition and excavation activities; (2) designing demolition, excavation and construction programs that will keep surface settlements and vibrations within acceptable limits set by LBNL; and (3) coordinating with LBNL and providing settlement and vibration monitoring, as needed, to assure that adjacent facilities are not adversely affected during the geotechnical aspects of construction.

7.05.4 Wet Weather Construction

Although it is possible for excavation and/or construction to proceed during or immediately following the wet winter months, a number of geotechnical problems may occur which may increase costs and cause project delays. Dewatering requirements will potentially increase due to rainfall, surface runoff, seepage and rises in groundwater level. The water content of onsite soils may increase during the winter and rise significantly above optimum moisture content for compaction of subgrade or backfill materials. If this occurs, the contractor may be unable to achieve the specified levels of compaction. The stability of temporary slopes will decrease, potentially increasing the lateral extent of excavation required. If utility or footing trenches are open during winter rains, caving of the trench walls may occur. Subgrade preparation beneath footings, mat foundations, slabs-on-grade and pavement sections may prove difficult or infeasible. In general, we note that it has also been our experience that increased clean-up costs may be incurred, and greater safety hazards may exist, if the work proceeds during the wet winter months.

8.00 GEOTECHNICAL RECOMMENDATIONS

8.01 General

Contractors responsible for the geotechnical aspects of the project (e.g., general, shoring, grading, foundation) should become familiar with the contents of this report and acknowledge:

- The site conditions, as described in this report and the attached Appendices;
- The construction considerations discussed in Section 7.05 of this report; and
- LBNL requirements for excavations, safety and vibrations.

We recommend that these and all other contractor responsibilities be clearly defined in the project plans and specifications.

8.02 Seismic Design

8.02.1 2007 California Building Code Seismic Parameters

Structures at the site should be designed to resist strong ground shaking in accordance with the applicable building codes and local design practice. This section provides location-specific seismic design parameters for use with the 2007 California Building Code (CBC 2007). Based on our review of the subsurface conditions and the 2007 California Building Code (CBC), we judge that a "C" Site Class is applicable.

Site Location
Latitude = 37.8760 degrees
Longitude = -122.2466 degrees

Mapped Spectral Accelerations
Short Period, (S_S, Site Class B) = 2.008g
1-Second Period, (S_1, Site Class B) = 0.783g

Maximum Considered Earthquake Spectral Response Accelerations
Short Period, (SM_S, Site Class C) = 2.008g
1-Second Period (SM_1, Site Class C) = 1.018g

Design Spectral Response Accelerations
Short Period (SD_S, Site Class C) = 1.339g
1-Second Period (SD_1, Site Class C) = 0.679g

The acceleration parameters presented above were obtained using the United States Geological Survey's Java-based website application (http://earthquake.usgs.gov/research/hazmaps/design/), which we accessed on December 3, 2009. The Maximum Considered Earthquake Spectral Response Accelerations are associated with 2 percent probability of exceedence in 50 years level-of-hazard. The Design Spectral Response Accelerations are two-thirds of the Maximum Considered Earthquake values.

8.02.2 LBNL/UCB Campus Probabilistic Ground Motions

In 2000, a suite of probabilistically-derived ground motions were developed for the UCB campus as a whole. This standardized suite of ground motions (response spectra and acceleration time histories) was updated by URS Corporation (URS) in 2003 and again in 2008. The 2008 update (URS 2008) included ground motions for LBNL for four return periods (72, 475, 949 and 2,475 years). The results for a 475-year return period (10 percent probability of exceedence in 50 years) are summarized in the following table.

LBNL PSHA Ground Motions

475-year Return Period Spectral Accelerations		
0.01-Second Period	**0.2-Second Period**	**1-Second Period**
0.86g	2.02g	0.85g

If use of the URS ground motions is being considered, we recommend that the project design team obtain the most current version of the ground motion report directly from LBNL. AKA would be pleased to consult with the design team pertaining to the applicability of the ground motions to the SERC site; however, we would also recommend that URS be consulted to verify that any assumptions and/or restrictions pertaining to the application of the ground motions were appropriately considered.

8.03 Foundation Design

8.03.1 General

All footings should be designed to bear at least 18 inches below lowest adjacent firm finished grade. Continuous and isolated spread footings should have minimum widths of 18 inches and 24 inches, respectively.

- New spread footings supporting site retaining walls can be designed to bear upon natural non-expansive soil, bedrock or appropriately engineered fill.

- Footings for the SERC building should be interconnected and designed to extend at least 12 inches into bedrock, or should bear upon lean mix (or structural) concrete that extends at least 12 inches into bedrock.

The bedrock surface contour map presented on Figure 5 can be used to evaluate the approximate elevation of the top of bedrock at the SERC site. All footing excavations should be checked by AKA for proper depth, bearing, and cleanout prior to the placement of reinforcing steel. Footing excavations should be kept moist and free of loose material and standing water prior to concrete placement.

8.03.2 Footing Bearing Pressures

Shallow spread footings bearing on non-expansive natural firm soil or engineered fill can be designed using the following bearing pressures:

Bearing Pressures for Spread Footings on Soil

Load Case	Bearing Pressure (psf)	Factor of Safety
Dead Load (DL) Allowable	2000	3.0
Dead Plus Live Load (DL+LL) Allowable	3000	2.0
Total (DL+LL+wind or seismic) Allowable	4000	1.5
Ultimate	6000	1.0

Footings that extend at least 12 inches into bedrock or bear upon lean mix or structural concrete that extends at least 12 inches into bedrock can be evaluated using the higher bearing values presented in the following table.

Bearing Pressures for Spread Footings in Rock

Load Case	Bearing Pressure (psf)	Factor of Safety
Dead Load (DL) Allowable	3000	3.0
Dead Plus Live Load (DL+LL) Allowable	4500	2.0
Total (DL+LL+wind or seismic) Allowable	6000	1.5
Ultimate	9000	1.0

8.03.3 Lateral Resistance

Resistance to lateral loads can be provided by friction along the base of foundations and by passive pressures developing on the sides of below-grade structural elements. Passive resistance in soil can be estimated using an equivalent fluid weight of 350 pounds per cubic foot (pcf). Passive resistance in bedrock can be estimated using an equivalent fluid weight of 450 pcf. These values can be increased by one-third for dynamic loading. The bedrock surface contour map presented on Figure 5 can be used to evaluate the approximate elevation of the top of bedrock at the SERC site.

Where pavements cover the adjacent ground surface or floor slabs, passive resistance can be assumed to begin at the ground surface. In areas not confined by slabs or pavements, passive resistance should be neglected within 1 foot of the ground surface.

A friction coefficient of 0.35 can be used to evaluate frictional resistance along the bottoms of spread footing foundations. The above passive and frictional resistance values include a factor of safety of at least 1.5 and can be fully mobilized with deformations of less than 1/2- and 1/4-inch, respectively.

8.04 Retaining Wall Lateral Earth Pressures

8.04.1 Applicability of Lateral Earth Pressure Distributions

This section presents lateral earth pressure distributions for: (1) cantilever walls that are free to rotate; (2) fixed "basement-type" walls; and (3) tied-back or internally braced flexible walls. The static lateral earth forces in this section are unfactored; normal factors of safety for long-term sustained loading should be used when utilizing these load distributions for wall design. The applicability of each lateral earth pressure distribution is as follows:

- Rigid walls that are free to rotate at their tops can be designed using an "active" triangular earth pressure distribution.

- Rigid walls that are fixed and unable to rotate can be designed using an "at rest" triangular earth pressure distribution.

- Temporary tied-back or braced retaining walls (i.e., shoring) should be designed using an "apparent" trapezoidal lateral force distribution. This force distribution is appropriate for flexible walls restrained by tiebacks or internal bracing under temporary loading conditions (i.e., during and immediately after construction).

- Permanent tied-back retaining walls should be designed using a lateral pressure distribution that envelops both: (1) the temporary trapezoidal "apparent" earth pressure distribution; and (2) the appropriate triangular earth pressure distribution for the permanent condition, depending upon whether the wall is free to rotate or is fixed.

The lateral earth pressures presented in this section are appropriate only for retaining walls that are fully-drained to prevent the build-up of hydrostatic pressures. Recommendations for retaining wall backdrainage are presented in Section 8.05.

8.04.2 Static Lateral Pressures on Walls Considered Free-to-Rotate

This load case applies to any site retaining walls that are free-to-rotate and are therefore unrestrained by tiebacks, other structural elements or wall geometry. The recommended lateral pressure distribution for this case is based on active soil pressures and increases uniformly with depth (triangular distribution).

Static Lateral Pressures for Free-to-Rotate Site Retaining Walls

Slope Behind Wall	Horizontal Lateral Pressure (psf per foot of depth)	Increase over Level Backslope
Level	45	1.00
3:1	50	1.11
2:1	60	1.33

8.04.3 Static Lateral Pressures on Fixed "Basement-Type" Walls

The recommended lateral pressure distribution for this case is based on "at rest" earth pressures and increases uniformly with depth (triangular distribution).

Static Lateral Pressures for Fixed Retaining Walls

Slope Behind Wall	Horizontal Lateral Pressure (psf per foot of depth)	Increase over Level Backslope
Level	60	1.00
3:1	67	1.11
2:1	80	1.33

8.04.4 Static Lateral Pressures for Tied-Back or Braced Flexible Walls

Our recommended lateral pressure distribution diagram for static lateral earth pressures on flexible tied-back walls or internally braced walls is presented on Figure 6. The recommended static lateral pressure distribution is based on active earth pressures redistributed into a trapezoidal "apparent" earth pressure diagram. The maximum (uniform) lateral pressures shown on Figure 6 are also presented below.

Static Lateral Pressures for Tied-back or Braced Flexible Walls

Slope Behind Wall	Uniform Horizontal Lateral Pressure (psf for wall height in feet)	Increase over Level Backslope
Level	25H	1.00
3:1	28H	1.12
2:1	33H	1.32

For the temporary (i.e., during construction) load case, H should be taken as the vertical distance from the bottom of the adjacent excavation to the top of the retaining wall. For the long-term (i.e., after construction) load case: (1) H can be taken as the vertical distance from the adjacent finished grade to the top of the retaining wall; (2) the "design" earth pressure should also envelop the appropriate triangular earth pressure distribution for the permanent condition, depending upon whether the wall is free to rotate or is fixed.

8.04.5 Increases in Lateral Wall Pressures

Retaining walls should be designed to resist increases in lateral pressure caused by vehicle loadings and/or other surcharges that may be applied at the ground surface. The following lateral pressure distributions can be used for the design of retaining walls for a level backfill condition under normal surcharge conditions.

Increases in Lateral Wall Pressures Caused by Surcharges

Load Condition	Lateral Pressure
Surcharge (vehicles)	100 psf (uniform) – applied over the upper 10 feet of the wall height
Surcharge (general)	0.5 times anticipated surcharge load (uniform) – applied over the full height of the wall

Unusually heavy and/or concentrated surcharge loads should be evaluated on an individual basis.

Lateral load increases caused by earthquake shaking can be estimated using the earthquake surcharge pressures presented below.

Increases in Lateral Wall Pressures Caused by Earthquake Shaking

Slope Behind Wall	Uniform Horizontal Lateral Pressure (psf for wall height, H, in feet)
Level	18H
3:1	20H
2:1	24H

We note that the selection of appropriate lateral pressures for seismic design involves considerable judgment and, in our opinion, would be best evaluated in consultation with the design engineer for the retaining wall. In evaluating seismic lateral pressures, static earth loads and earthquake surcharge loads should be considered in combination using appropriate load factors. It is commonly accepted that seismic lateral pressure increases can be neglected for elevator pits, sumps or other "balanced" conditions where the ground surface on both sides of the structure is nearly level.

8.05 Wall Backdrainage

8.05.1 General

The lateral forces and pressures presented in the previous section (Section 8.04) are only appropriate for retaining walls that are fully drained to prevent the build-up of hydrostatic pressure. Wall drainage may consist of either: (1) holes, slots or gaps in the wall that allow water to freely drain through the wall face; or (2) a wall backdrainage system that collects water from behind the wall and drains it, by gravity, to an appropriate discharge location. Backdrainage should consist of either prefabricated drainage material (Miradrain or an approved alternative) installed in accordance with the manufacturer's recommendations, or a vertical gravel blanket at least 12 inches thick. Additional drainage provisions may be required if seepage conditions are exposed during wall construction. We recommend that a waterproofing consultant be retained to provide any additional recommendations needed pertaining to the waterproofing of retaining walls or below-grade portions of the SERC building.

The upper foot of retained soil behind the wall should be backfilled with low permeability soil to limit surface water infiltration into the wall backdrainage system. Concrete paving or a lined V-ditch/gutter should be installed behind the wall above the low-permeability soil that directs water away from the back of the wall and toward a suitable gravity discharge.

8.05.2 Prefabricated Drainage Material

Prefabricated drainage material should be in direct contact with the retained soil/rock materials behind the wall and should be designed to drain through weepholes or into a perforated plastic pipe or other approved prefabricated drainage conduit. If prefabricated drainage material is used, the elements comprising the wall backdrainage system should be specified and detailed in accordance with the manufacturer's recommendations. Drainage material should have sufficient crushing strength to support the expected lateral earth pressures.

We recommend full slope face coverage with prefabricated drainage panels unless soldier piles are used, in which case minimum 50% slope face coverage is acceptable. Additional drainage provisions may be required if seepage conditions are exposed during wall construction.

8.05.3 Vertical Gravel Blanket

The drain rock used to construct the vertical gravel blanket should conform to Caltrans specifications for Class 2 permeable material. Alternatively, locally available, clean, ½- to ¾-inch maximum size crushed rock or gravel could be used, provided it is encapsulated in a non-woven geotextile filter fabric, such as Mirafi 140N or an approved alternative. The gravel blanket should drain into a perforated plastic pipe installed (with perforations down) along the base of the walls on a 2-inch-thick bed of drain rock. Plastic pipe should be sloped to drain by gravity to a sump, relief wells or other suitable discharge and a cleanout should be provided at the pipe's upslope end. Perforated and non-perforated plastic pipe used in the drainage system should consist of 4-inch diameter Schedule 40 PVC or an approved equivalent.

8.06 <u>Tieback Design and Testing</u>

8.06.1 Tieback Design

Tieback holes should be inclined downward at an angle of at least 10 degrees below the horizontal. Tiebacks should be anchored in an anchorage zone located behind an imaginary plane inclined 60 degrees from the horizontal passing through a line 5 feet behind the base of the excavation (Figure 6). Portions of the tieback that are not within the anchorage zone should be designed to be unbonded. A minimum unbonded length of 15 feet should be provided. All permanent tiebacks should be equipped with double corrosion protection. Appropriate corrosion protection should also be provided surrounding the tieback/wall anchorage connection.

The allowable capacities of tiebacks will depend upon a variety of factors including the installation method, hole diameter, grout pressure, and workmanship. Consequently, we recommend that the contractor be responsible for providing tiebacks of the required design capacity that meet specified deflection limits. The following allowable skin friction values can be used to preliminarily estimate the lengths of tiebacks post-grouted under pressure:

Estimated Skin Friction Values for Post-Grouted Tiebacks

Anchorage Material	**Allowable Skin Friction with Post-Grouting (psf)**	
	Long-Term DL+LL	**Total Loads**
Soil	900	1200
Rock	2400	3200

The allowable skin friction values for static earth loads and total loads presented above include safety factors of approximately 2.0 and 1.5, respectively.

8.06.2 Tieback Testing

The required anchor bond length or each tieback should be confirmed by a load test program conducted under the observation of the Geotechnical Engineer. All tiebacks should be proof-tested to at least 1.25 times the design load. In addition, the first two production tiebacks as well as 2 percent of the remaining tiebacks should be performance-tested to 1.5 times the design load. Tiebacks should be sized such that the magnitude of the test load does not exceed 80 percent of the guaranteed ultimate strength of the tendons or bars. Replacement tiebacks should be provided for tiebacks that fail the load test. The load at which tiebacks should be permanently locked off should be determined by the project Structural Engineer. Approximately 10 percent of the tiebacks should be checked 24 hours after the initial lock-off to check that stress relaxation has not occurred. AKA should review and approve all submittals from the contractor pertaining to the design, installation and testing of tiebacks.

8.07 <u>Soil Nail Design and Testing</u>

8.07.1 Soil Nail Design

Soil nail walls should be designed in general conformance to the Federal Highway Administration (FHWA) procedures, as outlined in the following publication:

> FHWA, 2003, "Geotechnical Engineering Circular no. 7–Soil Nail Walls," FHWA0-IF-03-017, March 2003.

The Caltrans computer program SNAILZ or other proprietary programs can be used to assist the designer. Generally, nail lengths of 0.8 to 1.0 times the wall height are used. The following geotechnical design parameters can be used for design:

Soil Parameters for Soil Nail Wall Design

	Material Type	
Design Parameter	**Soil**	**Rock**
Effective cohesion, c'	500 psf	2500 psf
Effective friction, ø'	25°	0°
Soil total unit weight	120 pcf	120 pcf
Ultimate soil-grout bound	500 psf	1200 psf

For soil nail design, we recommend assuming that groundwater levels may temporarily rise to Elevation +930 feet during and/or following periods of prolonged heavy rainfall.

The design of the soil nail wall should confirm adequate factors of safety with respect to internal, external and combined failure modes in accordance with generally accepted procedures for soil nail walls. A ground acceleration of 0.57g can be used for seismic design (i.e, two-thirds of the probabilistically-derived PGA for LBNL). Drilled holes for the soil nails should slope downward at a minimum angle of 15 degrees below the horizontal to facilitate the gravity flow of grout into the annular space between the soil nail and the soil/rock interface. All soil nails should be equipped with centralizers.

8.07.2 Soil Nail Testing

A testing program should be implemented during construction to include the following:

1. Pre-production verification of nail pull-out capacity should be performed. This will entail installing nails in undisturbed ground and loading them to at least 200% of the design load and, preferably, to failure. Where failure loading is intended, the reinforcing must be sized to the anticipated failure load, which could be as great as 400% of the design load.
2. Creep load test(s) should be performed during the verification program.
3. During production installation, about 5% of the nails should be proof-loaded to at least 150% of the design load.

Soil nail testing and inspection should conform to the general requirements presented in the Caltrans Standard Special Provisions for Soil Nail Walls and, where applicable, guidelines in the FHWA Soil Nail Manual. AKA and the project Structural Engineer should review and approve all submittals from the contractor pertaining to the design, installation and testing of soil nails.

8.08 <u>Retaining Wall Foundations</u>

8.08.1 Retaining Wall Footings

Retaining wall footings can be designed in accordance with the recommendations previously presented in Section 8.03, "Foundation Design."

8.08.2 Drilled Pier and Soldier Pile Design

The axial capacity of drilled piers and soldier piles can be evaluated using an allowable skin friction value of 500 psf in soil and 1200 psf in bedrock, which can be increased one-third for total compressive loads (including wind or seismic) but should not be increased for uplift loads. The recommended allowable skin friction values can be considered over the full pile/pier diameter for the embedded portion of the soldier piles and over one-half of the pier diameter for the upper portion of soldier piles that are retained by tiebacks, ignoring any skin friction from above the uppermost tieback row.

Resistance to lateral loads can be provided by passive resistance acting on the embedded length of the soldier pile. For firm, in-place soil and engineered fill, passive resistance can be evaluated using a triangular distribution with a uniform increase of 300 psf per foot of depth. In bedrock, passive resistance can be assumed to increase at the higher rate of 450 psf per foot of depth starting at a minimum value of 1,200 pounds per square foot (psf) at the bedrock surface. Passive resistance can be applied over two (horizontal) pier diameters. The above passive resistance values can be considered "allowable" values under long-term loading conditions and include a factor of safety of at least 2.0.

8.08.3 Drilled Pier and Soldier Pile Installation

In general, holes for drilled piers and soldier piles should be drilled straight and plumb (within 1 percent of vertical). All piers should be cleaned of loose soil and rock fragments prior to concreting. We judge that the holes can likely be drilled using heavy auger drilling equipment; however, zones of relatively hard rock could be encountered. The contractor should be prepared to utilize suitable hard rock drilling techniques, if necessary. If water accumulates in the holes, it should be removed by pumping or bailing prior to concrete placement unless tremie methods are used.

Concrete placement should start as soon as possible after the drilling and cleanout is complete. In all cases, holes for piers and soldier piles should be concreted on the day they are drilled. Following placement of the reinforcing steel, H-section or welded channel sections, holes should be concreted from the bottom up in a single operation. If water is present in the hole, the tremie pipe should be constantly maintained at least 5 feet below the surface of the concrete during casting of the pier. As the concrete is placed, the casing used to stabilize the hole should be withdrawn. The bottom of the casing should be maintained not more than 5 feet or less than 1 foot below the surface of the concrete.

Drilled piers and soldier piles should be installed by a qualified drilling contractor. We recommend that AKA observe drilled pier and soldier pile installation to confirm that subsurface conditions are as anticipated and that the piers/piles are constructed in accordance with the recommendations presented in this report.

8.09 Earthwork

8.09.1 Site Preparation

The site limits, as defined on the plans, should be clearly marked in the field. Prior to demolition and site clearing, all active subsurface utilities in and immediately surrounding the site limits should be located, marked and protected or relocated. Areas within the site limits should be cleared of structures, foundations, pavements, aggregate base, slabs-on-grade, catch basins, storm drains, sewers, utilities and all other near-surface improvements. Any soils containing vegetation and/or organic matter should be stripped. Cleared materials should be removed from the site unless they are specifically identified as suitable for re-use by LBNL and AKA. Stripped materials are not suitable for re-use as engineered fill and should be removed from the site or stockpiled for later use as landscape material (at LBNL's discretion).

The contractor should document the condition of existing improvements located outside of the site limits, and should perform any and all monitoring activities required by LBNL (which may include monitoring construction vibrations). We recommend that AKA and LBNL review the contractor's proposed construction monitoring plans prior to the start of demolition and site preparation activities.

8.09.2 Excavation

Any existing fill materials within and surrounding planned improvements should be excavated to expose firm natural materials (colluvium or bedrock). Where fill materials are encountered that extend below planned near-surface improvements (e.g., pavements, exterior slabs and site retaining walls), all of the existing fill materials should be removed within a zone extended down from the external edge of the improvements at an inclination of ½:1 (horizontal to vertical), or flatter.

Excavation for the SERC building should be observed intermittently by AKA to check that subsurface conditions are as anticipated and to determine whether additional observation is necessary to evaluate geologic stability. AKA should also observe the condition of exposed subgrades following excavation to check that suitable materials are exposed.

Due to the inherent variability of the soil and rock materials at the site, it should be anticipated that it may be necessary to overexcavate weak and/or expansive materials, where present, in order to replace these unsuitable materials with engineered fill. Localized overexcavation may also be needed to ensure that the footings that support the SERC building are not underlain by soil.

8.09.3 Fill Materials

Fill materials should conform to the requirements presented below:

General Fill – General fill material should have an organic content of less than 3 percent by volume and should not contain rocks or lumps larger than 6 inches in greatest dimension.

Non-Expansive Fill – Non-expansive fill material should:

- Be free of 6-inch plus material with no more than 15 percent of material larger than 2.5 inches;
- Be free of organic material, debris and environmental contaminants;
- Have a Plasticity Index of 15 or less; and
- Have a Liquid Limit of 40 or less.

Import Fill – We recommend that import fill conform to the requirements for non-expansive fill and have a Plasticity Index of 12 or less.

All proposed fill materials should be approved by AKA prior to their use. Much of the material cleared or excavated from the site may be suitable for re-use as non-expansive fill, from a geotechnical standpoint; although some materials may need to be processed (i.e., by crushing and/or blending) to meet the above requirements. LBNL should review and approve proposed fill materials to be used beneath or adjacent to the SERC building to check that they are suitable from an environmental perspective. Any proposed import fill material should be evaluated by our firm prior to its importation to the site.

8.09.4 Fill Placement

AKA should observe the condition of the subgrade upon which fill will be placed to check that it is firm and non-yielding and that no old fill or other unsuitable material is present. Prepared subgrades should be relatively level prior to fill placement. Where fill is to be placed adjacent to areas of sloping ground, it should be appropriately keyed and benched into the slope. We recommend that AKA evaluate whether subdrainage (e.g., permeable material, perforated plastic pipe, non-perforated cleanout and outlet pipes, and a suitable gravity discharge) should be installed prior to filling based on conditions observed following excavation. Any required subdrainage should be installed in accordance with AKA's supplemental recommendations.

Fill materials should be placed in a manner that minimizes lenses, pockets and/or layers of materials differing substantially in texture or gradation from the surrounding fill materials. The soils should be spread in uniform layers not exceeding 8 inches in loose thickness prior to compaction. Each layer should be compacted using mechanical means to at least 90 percent relative compaction (per ASTM D1557). Fill materials that are predominantly granular in nature should be compacted to at least 95 percent relative compaction, per ASTM D1557. The fill should be constructed in layers such that the surface of each layer is nearly level.

AKA should observe fill placement and test compaction, as appropriate, to confirm and document that the work was performed in accordance with the specifications and the intent of our geotechnical recommendations.

8.09.5 Utility Trenches

Utility trenches should be backfilled with fill placed in lifts not exceeding 8 inches in uncompacted thickness. Trenches should be filled by placing a granular shading layer beneath and around the pipe, and then 6 to 12 inches of shading should be carefully placed and tamped above the pipe. The remaining portion of the trench should be backfilled with onsite or import soil. The backfill (above shading layers) should be placed and compacted to at least 90 percent relative compaction (per ASTM D1557). The compaction requirements given above should be considered minimum recommended requirements. If LBNL and/or utility company specifications require different or more stringent backfill requirements, those specifications should be followed.

If imported granular soil is used, sufficient water should be added during the trench backfilling operations to prevent the soil from "bulking" during compaction. All compaction operations should be performed by mechanical means only. We recommend against jetting.

Where granular backfill is used in utility trenches, we recommend an impermeable plug or mastic sealant be used where utilities enter the building to minimize the potential for free water or moisture to enter below the building. Finally, because of the potential for collapse of trench walls, we recommend the contractor carefully evaluate the stability of all trenches and use temporary shoring, where appropriate. The design and installation of the temporary shoring should be wholly the responsibility of the contractor. In addition, all state and local regulations governing safety around such excavations should be carefully followed.

AKA should observe utility trench backfilling and test compaction, as appropriate, to confirm and document that the work was performed in accordance with the specifications and the intent of our geotechnical recommendations.

8.10 Interior Slabs-on-Grade

8.10.1 Slab Underdrainage and Support

An underdrainage system should be installed below the Level 1 concrete floor slab to intercept and drain away seepage that could otherwise become trapped beneath the building. The underdrainage system should consist of:

- An underdrainage layer consisting of at least 8 inches of ¾-inch clean, open-graded, compacted drainrock; and
- A system of perforated and non-perforated minimum 4-inch-diameter SDR 35 or Schedule 40 PVC pipes.

Prior to the construction of the underdrainage layer, the exposed natural subgrade materials should be proof-rolled under our observation and confirmed to be uniform and non-yielding. In all cases, the upper 18 inches of material beneath the bottom of the Level 1 slab-on-grade should consist of non-expansive material (i.e., have a PI of 15 or less). If expansive materials are encountered at subgrade level, it may be necessary to overexcavate and replace these unsuitable materials with non-expansive fill, depending upon the thickness of the underdrainage layer (which is also considered non-expansive).

Perforated collector pipes should be placed near the base of the drainrock layer that are sloped to drain towards non-perforated collector pipes that lead to an appropriate gravity discharge. Perforated collector pipes should be bedded upon a thin layer of drainrock and should be spaced no more than 20 feet apart. Where necessary, the collector pipes can be installed in drainrock-filled trenches that are contiguous with the underdrainage layer. The underdrainage layer should be compacted by mechanical means, with care taken not to damage the collector pipes during the compaction efforts.

8.10.2 Moisture Retarder

A moisture retarder should be installed beneath interior concrete slabs that are cast on-grade; if a waterproof barrier is desired, a waterproofing consultant should be consulted to provide supplemental or alternate recommendations. Either of the following moisture retarders is considered generally acceptable, from a geotechnical standpoint:

- 4 inches (minimum) of free-draining gravel overlain by a vapor retardant membrane (Class A Vapor Retarder [ASTM E1745, latest revision]), covered with 2 inches of sand, or

- 6 inches of compacted aggregate base overlain by a heavy-duty impermeable membrane (Stego® wrap 15-mil or an approved equivalent) installed and taped in accordance with the manufacturer's recommendations.

If a heavy-duty impermeable membrane is used beneath the SERC Building Level 1 slab-on-grade, it may be acceptable to the manufacturer to place the membrane directly on the slab underdrainage layer, provided that it is protected by a 2-inch-thick sand covering. Alternatively, the heavy-duty impermeable membrane should be placed on an aggregate base layer that is compacted to provide a smooth and uniform surface.

Slab reinforcing should be provided in accordance with the anticipated use and loading of the slab. We recommend that interior slabs-on-grade be at least 5 inches thick and be reinforced with steel bar reinforcement. We also recommend that the specifications for slab-on-grade floors require that moisture emission tests be performed on the slab prior to the installation of flooring. No flooring should be installed until safe moisture emission levels are recorded for the type of flooring to be used.

8.11 Exterior Slabs-on-Grade

Subgrades beneath exterior slabs-on-grade should be proof-rolled under our observation and confirmed to be uniform and non-yielding prior to the placement of the slab reinforcement. Concrete slabs that may be subject to vehicle loadings should be designed in accordance with Section 8.12.2, "Rigid Pavements." Where non-expansive materials are present at subgrade level, exterior slabs can be cast directly upon the prepared, proof-rolled and approved subgrade. In all cases, the upper 12 inches of material beneath the bottom of exterior slabs-on-grade should consist of non-expansive material (i.e., have a PI of 15 or less).

Slab reinforcing should be provided in accordance with the anticipated use and loading of the slab. We recommend that exterior slabs-on-grade be at least 4 inches thick and be reinforced with steel bar reinforcement. Exterior slabs should be structurally independent from buildings and be free-floating. Score cuts or construction joints should be provided and minor movement and cracking of the slab should be expected. Steps to the building from exterior slab areas should include a gap between the steps and the building foundations. The recommendations presented above, if properly implemented, should help reduce the magnitude of exterior slab cracking.

8.12 Pavements

8.12.1 Flexible Pavements

Flexible asphalt concrete (AC) pavements may be used for parking areas and driveways. We developed the following recommended pavement sections for various traffic indices using the Caltrans R-value design method for flexible pavements. The sections below are based on an assumed subgrade R-value of 30 for non-expansive soil. The R-value of the soil beneath the aggregate base should be confirmed during construction.

Flexible Pavement Thickness Design for Subgrade R-Value = 30

Traffic Index	Asphalt Concrete (inches)	Caltrans Class 2 Aggregate Base (inches)	Total Thickness (inches)
4	2	6	8
5	3	6	9
6	3	9	12
7	3	12	15

The upper 12 inches of material beneath the bottom of the aggregate base should consist of non-expansive material (i.e., have a PI of 15 or less).

The assumed traffic indices of 4.0 and 5.0 are commonly used for automobile and light truck parking areas and access driveways, respectively. Traffic indices of 6.0 and 7.0 are commonly used for moderate truck access and parking areas. A traffic study has not been conducted by our firm for this project and our opinion regarding the applicability of the assumed traffic indices is experience-based and judgmental. The project civil engineer should choose the appropriate traffic indices for the pavement areas of the site and then use the given section for that traffic index.

The upper 6 inches of subgrade beneath planned pavements should be compacted to at least 95 percent relative compaction per ASTM D-1557. Pavement subgrades should be proof-rolled and confirmed to be uniformly firm and non-yielding prior to the placement of aggregate base. Aggregate base for use in pavements should conform to Caltrans Standard Specifications for Class 2 Aggregate Base. The aggregate base used in pavement sections should be compacted to at least 95 percent relative compaction as determined by ASTM D-1557.

8.12.2 Rigid Pavements

Rigid Portland cement concrete (PCC) pavements may also be used in driveway/loading areas. This section provides recommendations for Caltrans jointed plain concrete pavement (JPCP), which is engineered with longitudinal and transverse joints to control where cracking occurs. JPCPs do not contain steel reinforcement, other than tie bars and dowel bars. The project civil engineer should design and detail the JPCP per Caltrans specifications.

We developed the following pavement thickness design using the Caltrans R-value design method for rigid pavements and an assumed traffic index. The section below is for subgrade soils with an R-value between 10 and 40. We have assumed an R-value between 10 and 40 for the lime-treated soil or non-expansive fill required beneath the aggregate base. Once the non-expansive material has been selected by the contractor, the R-value should be verified.

Portland Cement Concrete Pavement Thickness Design

Traffic Index	Portland Cement Concrete (inches)	Caltrans Class 2 Aggregate Base (inches)	Total Thickness (inches)
< 9	9	12	21

The upper 12 inches of material beneath the bottom of the aggregate base should consist of non-expansive material (i.e., have a PI of 15 or less).

The upper 6 inches of subgrade beneath planned pavements should be compacted to at least 95 percent relative compaction per ASTM D-1557. Pavement subgrades should be proof-rolled and confirmed to be uniformly firm and non-yielding prior to the placement of aggregate base. Aggregate base for use in pavements should conform to Caltrans Standard Specifications for Class 2 Aggregate Base. The aggregate base used in pavement sections should be compacted to at least 95 percent relative compaction as determined by ASTM D-1557.

8.13 Future Geotechnical Services

We recommend we be provided the opportunity to review the project plans and specifications as they are being developed in order to check conformance with the intent of our geotechnical recommendations and to provide timely input, in the event that revisions are needed. We should also perform a general review of the geotechnical aspects of the final plans and specifications, the results of which we should document in a formal plan review letter.

It is critical that we be retained to provide geotechnical engineering services during the construction phases of the work in order to observe compliance with the design concepts, specifications, and recommendations and to allow design changes in the event that subsurface conditions differ from those anticipated prior to the start of construction. The scope of our construction-phase observation and testing services should include (but not necessarily be limited to): site preparation; excavation; subsurface drainage installation; placement and compaction of drainage layers, fill and aggregate base; footing excavation/construction; retaining wall drainage and backfill; installation and testing of tiebacks and/or soil nails (if used); pavement and slab-on-grade subgrade preparation; and utility installation.

9.00 LIMITATIONS

This report has been prepared for the exclusive use of the LBNL and its consultants for specific application to the proposed Solar Energy Research Facility (SERC) project in accordance with generally accepted soil and foundation engineering practices. No other warranty, expressed or implied, is made. Note that the findings presented in this report are based, in part, upon data collected by previous investigators. We cannot accept responsibility for the accuracy of the data obtained from others or (consequently) for interpretations that we have made based on existing available data. In the event that any changes in the nature or design of the building are planned, the conclusions and recommendations contained in this report should not be considered valid unless the changes are reviewed and the conclusions of this report are modified or verified in writing.

The findings of this report are valid as of the present date. However, the passing of time will likely change the conditions of the existing property due to natural processes or the works of man. In addition, due to legislation or the broadening of knowledge, changes in applicable or appropriate standards may occur. Accordingly, the findings of this report may be invalidated, wholly or partly, by changes beyond our control. Therefore, this report should not be relied upon after a period of three years without being reviewed by this office.

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LEGEND

Approximate location of Lawrence Berkeley National Laboratory boundary

Source: U.S. Geological Survey, Topographic Map of the Oakland East, Oakland West, Briones Vally and Richmond Quadrangles, 1993.

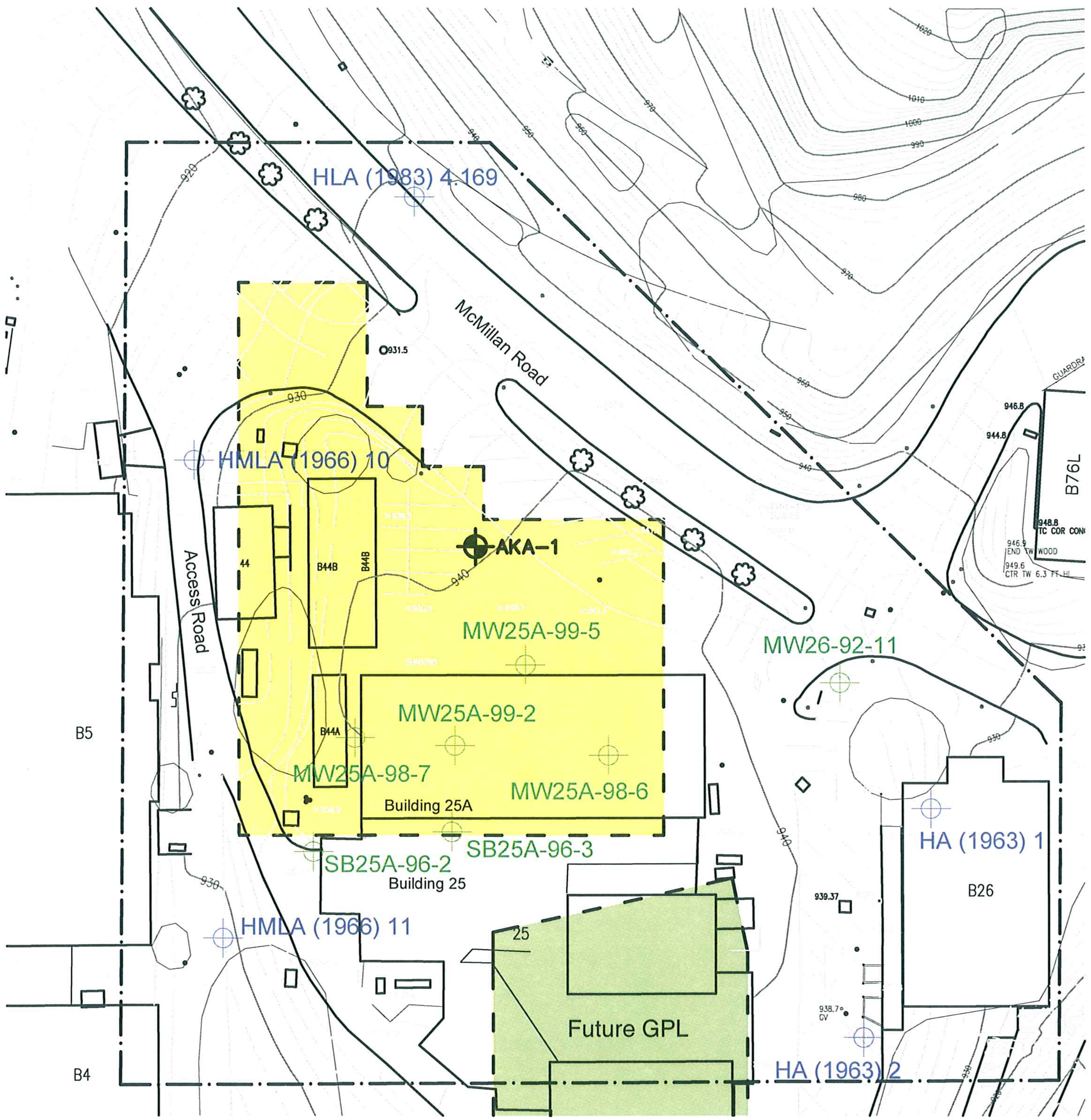
0 1,000 2,000
Feet

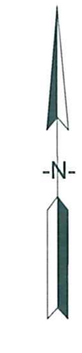
ALAN KROPP
& ASSOCIATES
Geotechnical Consultants

VICINITY MAP

SOLAR ENERGY RESEARCH CENTER (SERC)
Berkeley, California

PROJECT NO.	DATE	FIGURE 1
2500-13	**April 2010**	





LEGEND

Symbol	Description
AKA-1	Approximate location of AKA/WLA boring (AKA-2009) Appendix A
HLA (1983) 4.169	Approximate location of previous geotechnical/geologic boring by others (HA 1963; HMLA 1966; HLA 1988) Appendix B
MW25A-99-5	Approximate location of previous environmental boring/well by LBNL Appendix C
(yellow box)	Approximate footprint of Helios East Building
(green box)	Approximate footprint of future General Purpose Lab (GPL)
(dash-dot line)	Approximate limits of study area (borings and trenches outside study area not shown)

50 0 50 100

APPROXIMATE SCALE FEET

Base: Topographic base received from LBNL May 2009.

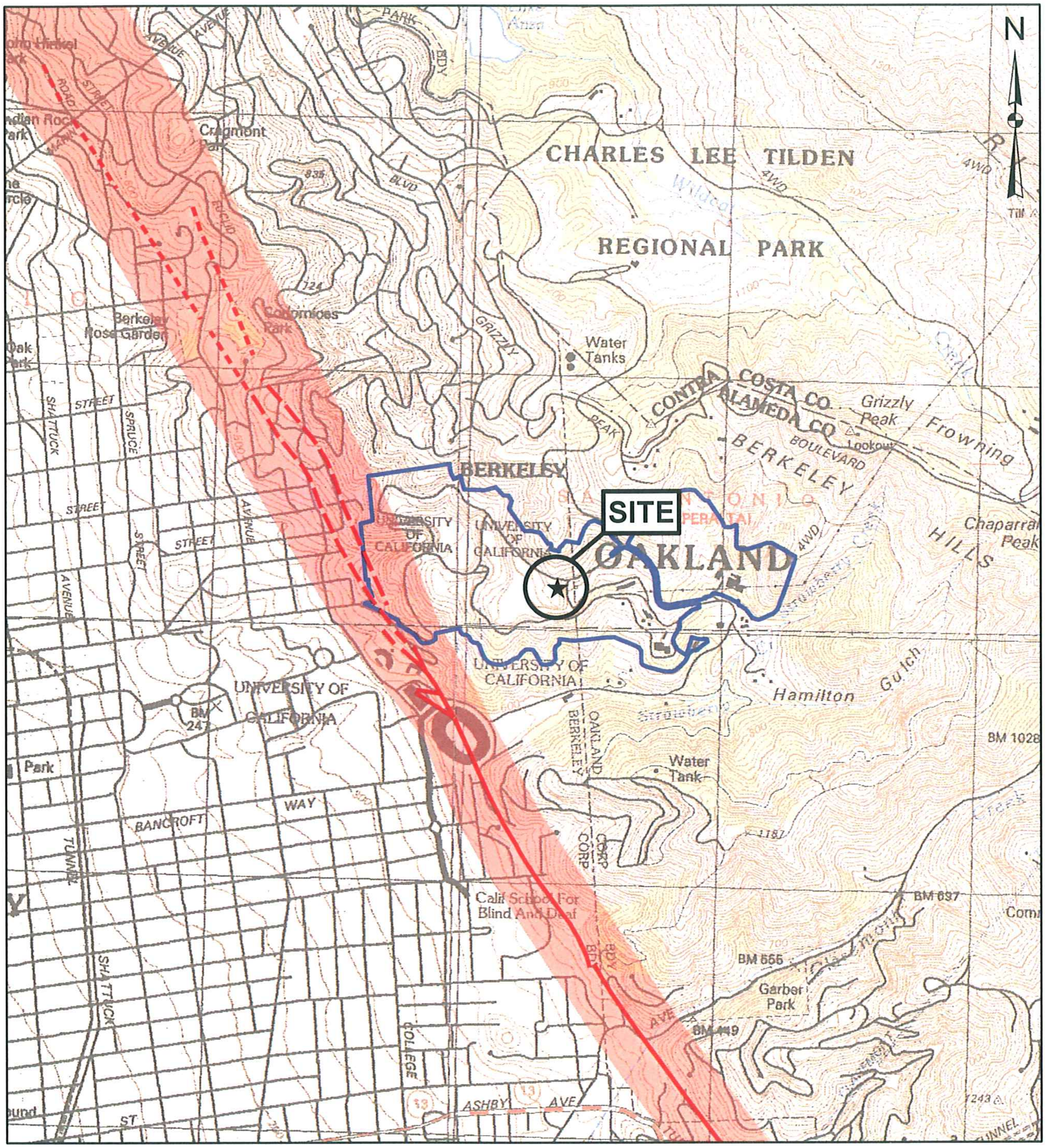
ALAN KROPP & ASSOCIATES

Geotechnical Consultants

SITE PLAN

SOLAR ENERGY RESEARCH CENTER (SERC)
Berkeley, California

PROJECT NO.	DATE	FIGURE 2
2500-13	**April 2010**	



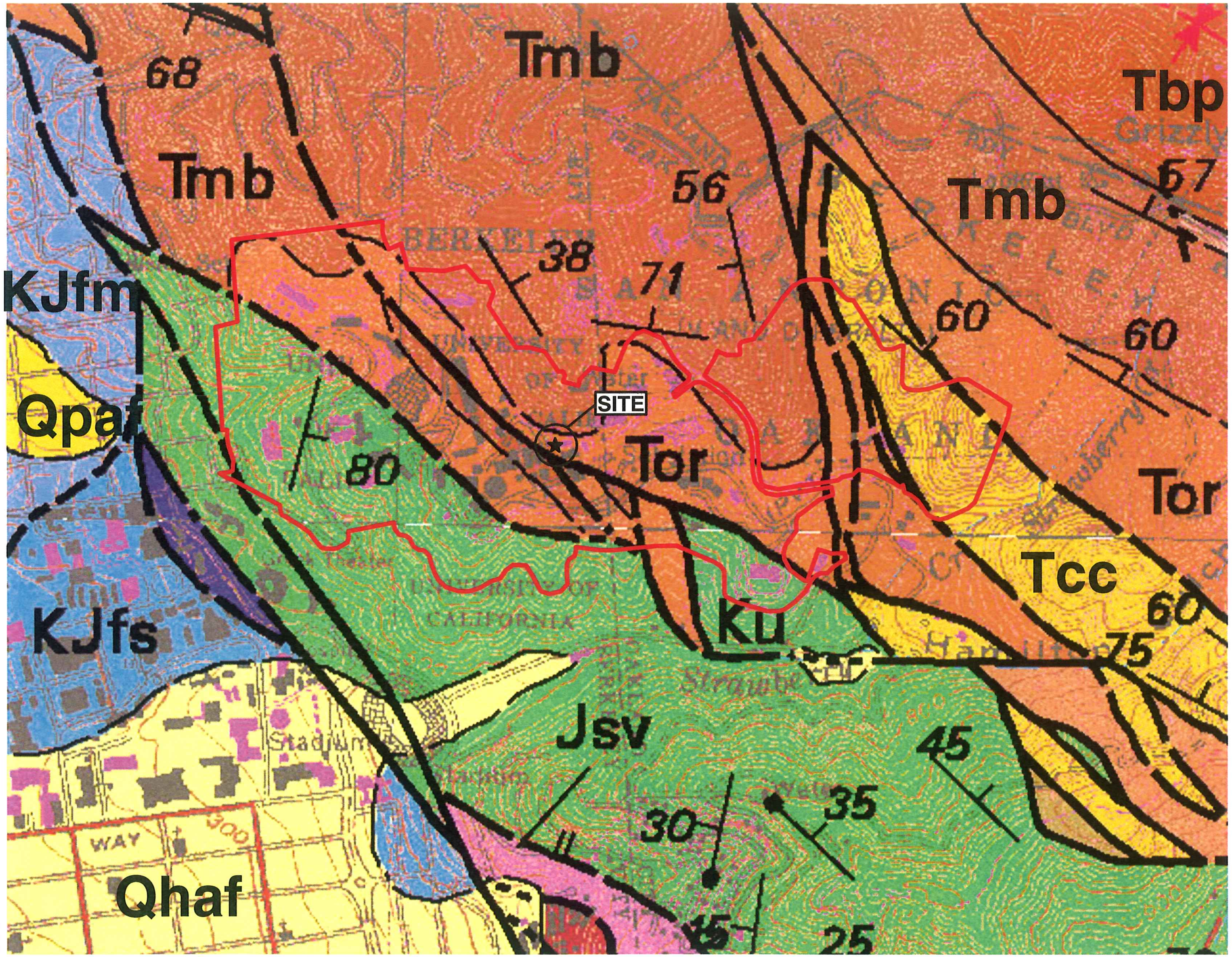
LEGEND

— Approximate location of Lawrence Berkeley National Laboratory boundary

0 2,000 4,000 Feet

Source: The California Geological Survey, 2001, CD's 2001-04, 2001-05, and 2001-06: GIS Files of Official Alquist-Priolo Earthquake Fault Zones (http://www.consrv.ca.gov/CGS/geologic_hazards/regulatory_hazard_zones/ap_cd_htm.htm).

ALAN KROPP & ASSOCIATES *Geotechnical Consultants*	AP FAULT ZONE MAP		
	SOLAR ENERGY RESEARCH CENTER (SERC) Berkeley, California		
	PROJECT NO.	DATE	FIGURE 3
	2500-13	**April 2010**	



-N-

LEGEND

Surficial Deposits

Qhaf	Alluvial fan and fluvial deposits (Holocene)
Qpaf	Alluvial fan and fluvial deposits (Pleistocene)

Assemblage I

Tbp	Bald Peak Basalt (late Miocene)
Tmb	Moraga Formation (late Miocene)
Tor	Orinda Formation (late Miocene)
Tcc	Claremont chert (late to middle Miocene)

Great Valley Complex

Ku	Unnamed sedimentary rocks (Late Cretaceous, Turonian and Cenomanion)
Jsv	Keratophyre and quartz keratophyre (Late Jurassic)

Franciscan Complex

KJfs	Franciscan complex sandstone, undivided (Late Cretaceous to Late Jurassic)
KJfm	Franciscan complex, mélange (Cretaceous Late Jurassic), includes mapped locally:
fs	Graywacke and meta-graywacke blocks

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

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

35 Strike and dip of bedding

48 Strike and dip of bedding, top indicator observed

Approximate boundary of Lawrence Berkeley National Laboratory

400 0 800 1600

APPROXIMATE SCALE FEET

Base: "Geologic Map and Map Database of the Oakland Metropolitan Area, Alameda, Contra Costa and San Francisco Counties, California" by Graymer, R.W., U.S. Geological Survey, Miscellaneous Field Studies MF-2342, 2000.

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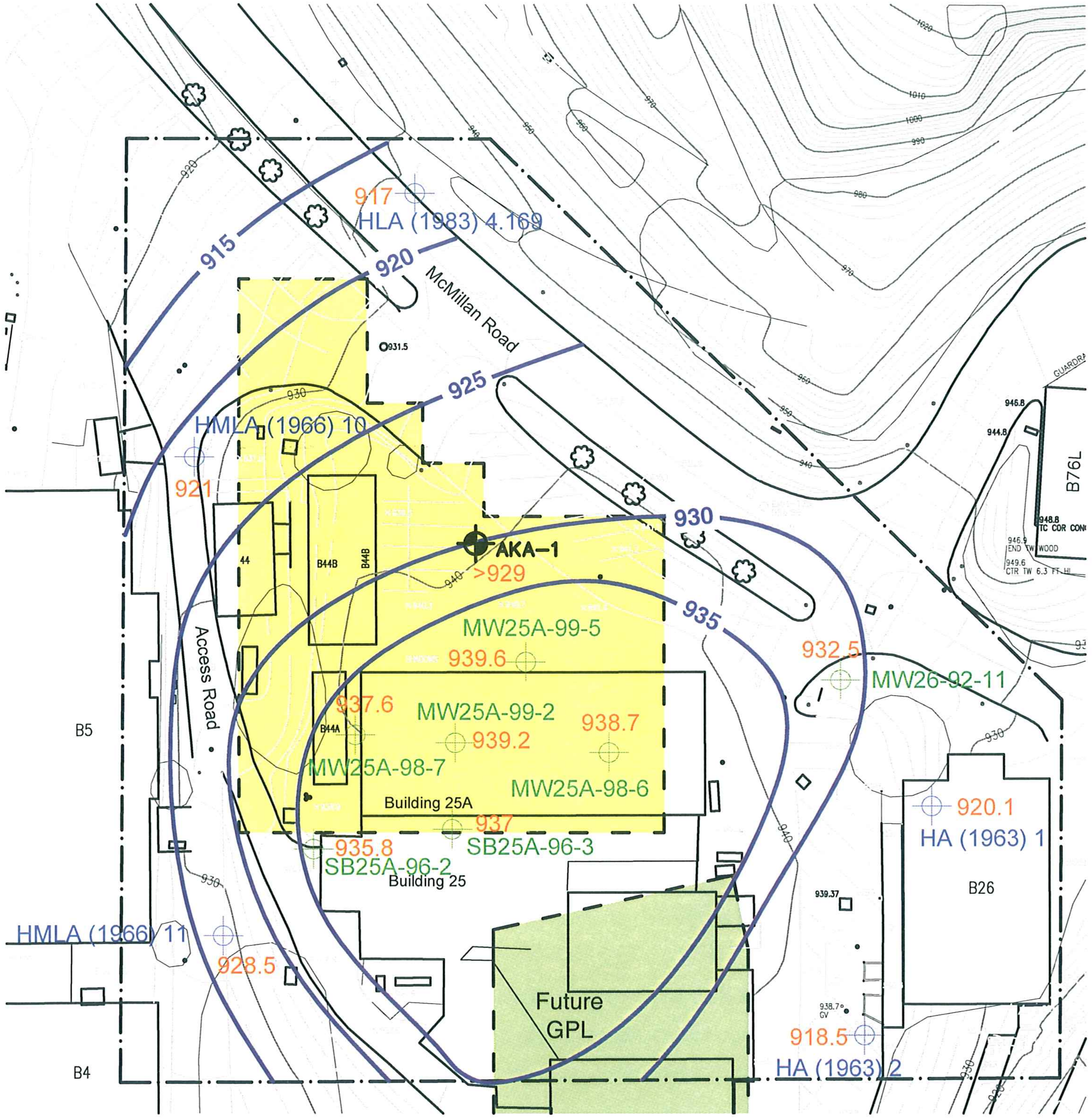
SIMPLIFIED REGIONAL GEOLOGIC MAP		
SOLAR ENERGY RESEARCH CENTER (SERC) Berkeley, California		
PROJECT NO. 2500-13	DATE April 2010	FIGURE 4



-N-

LEGEND

AKA-1	Approximate location of AKA/WLA boring (AKA-2009) Appendix A
HLA (1983) 4.169	Approximate location of previous geotechnical/geologic boring by others (HA 1963; HMLA 1966; HLA 1988) Appendix B
MW25A-99-5	Approximate location of previous environmental boring/well by LBNL Appendix C
932.5	Interpreted bedrock surface elevation (feet)
915	Interpreted bedrock surface contour (feet)
	Approximate footprint of Helios East Building
	Approximate footprint of future General Purpose Lab (GPL)
	Approximate limits of study area (borings and trenches outside study area not shown)

NOTE: Based on reviewed environmental logs only. Unconfirmed logs suggest that bedrock at some locations may be higher than shown.

50 0 50 100
APPROXIMATE SCALE FEET

Base: Topographic base received from LBNL May 2009.

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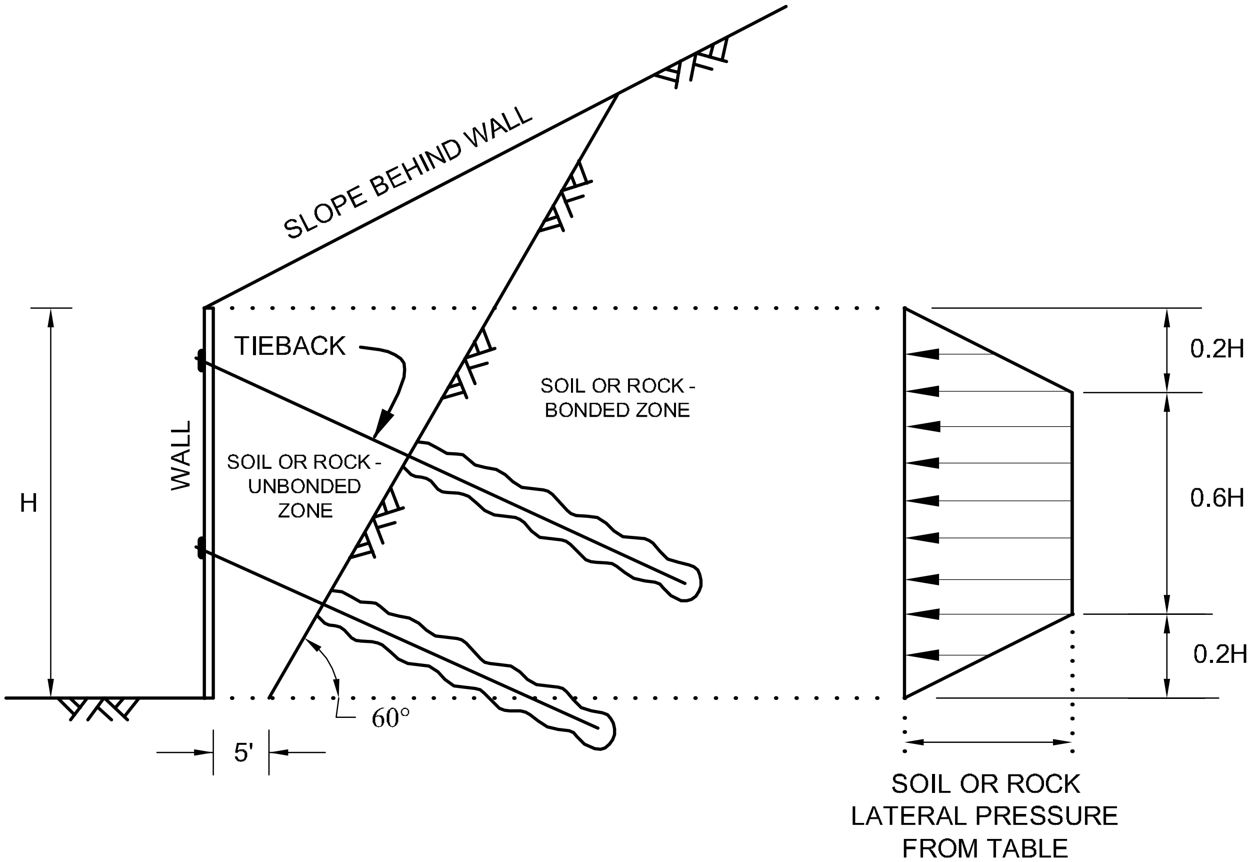
BEDROCK SURFACE CONTOUR MAP
SOLAR ENERGY RESEARCH CENTER (SERC)
Berkeley, California

PROJECT NO.	DATE	FIGURE 5
2500-13	**April 2010**	



SLOPE BEHIND WALL	LATERAL PRESSURE (psf for wall height in feet)
LEVEL	25H
3:1	28H
2:1	33H

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LATERAL PRESSURE DIAGRAM - TIEBACK WALLS

SOLAR ENERGY RESEARCH CENTER (SERC)
Berkeley, California

PROJECT NO.	DATE	FIGURE 6
2500-13	April 2010	

APPENDIX A

Log of Boring AKA-1

SOIL CLASSIFICATION CHART

PRIMARY DIVISIONS			SECONDARY DIVISIONS: CRITERIA *	GROUP SYMBOL	GROUP NAME
COARSE-GRAINED SOILS MORE THAN 50% RETAINED ON NO.200 SIEVE	GRAVELS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO.4 SIEVE	CLEAN GRAVELS LESS THAN 5% FINES	Cu ≥ 4 AND 1 ≤ Cc ≤ 3 *A*	GW	Well-graded gravel
			Cu < 4 AND/OR 1 > Cc > 3	GP	Poorly-graded gravel
		GRAVELS WITH FINES - MORE THAN 12% FINES	FINES CLASSIFY AS ML OR MH	GM	Silty gravel
			FINES CLASSIFY AS CL OR CH	GC	Clayey gravel
	SANDS 50% OR MORE OF COARSE FRACTION PASSES NO. 4 SIEVE	CLEAN SANDS LESS THAN 5% FINES	Cu ≥ 6 AND 1 ≤ Cc ≤ 3	SW	Well-graded sand
			Cu < 6 AND/OR 1 > Cc > 3	SP	Poorly-graded sand
		SANDS WITH FINES - MORE THAN 12% FINES	FINES CLASSIFY AS ML OR MH	SM	Silty sand
			FINES CLASSIFY AS CL OR CH	SC	Clayey sand
FINE-GRAINED SOILS 50% OR MORE PASSES THE NO.200 SIEVE	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50%	INORGANIC	PI > 7 AND PLOTS ON OR ABOVE "A" LINE	CL	Lean clay
			PI < 4 OR PLOTS BELOW "A" LINE	ML	Silt
		ORGANIC	LIQUID LIMIT - OVEN DRIED / LIQUID LIMIT - NOT DRIED < 0.75	OL	Organic Clay & Organic Silt
	SILTS AND CLAYS LIQUID LIMIT 50% OR MORE	INORGANIC	PI PLOTS ON OR ABOVE "A" LINE	CH	Fat clay
			PI PLOTS BELOW "A" LINE	MH	Elastic silt
		ORGANIC	LIQUID LIMIT - OVEN DRIED / LIQUID LIMIT - NOT DRIED < 0.75	OH	Organic Clay & Organic Silt
	HIGHLY ORGANIC SOILS		PRIMARILY ORGANIC MATTER, DARK IN COLOR, AND ORGANIC ODOR	PT	Peat

REFERENCE: Unified Soil Classification System (ASTM D 2487-06)

* Criteria may be done on visual basis, not necessarily based on lab testing

A – $C_u = D_{60}/D_{100}$ & $C_c = (D_{30})^2 / (D_{10} \times D_{60})$

GRAIN SIZES

U. S. STANDARD SERIES SIEVE — CLEAR SQUARE SIEVE OPENINGS

200 — 40 — 10 — 4 — 3/4" — 3" — 12"

SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

ABBREVIATIONS

<u>INDEX TESTS</u>

LL - Liquid Limit (%) (ASTM D 4318-05)
PI - Plasticity Index (%) (ASTM D 4318-05)
-200 - Passing No. 200 Sieve (%) (ASTM D 1140-00)

<u>STRENGTH TESTS</u>

PP - Field Pocket Penetrometer test of unconfined compressive strength (tsf)
TV - Field Torvane test of shear strength (psf)
UC - Laboratory unconfined compressive strength (psf) (ASTM D 2166-06)
TXUU - Laboratory unconsolidated, undrained triaxial test of undrained shear strength (psf) (ASTM D 2850-03a)

<u>MISCELLANEOUS</u>

ATOD - At time of drilling
psf/tsf - pounds per square foot / tons per square foot
psi - pounds per square inch (indicates relative force required to advance Shelby tube sampler)

SYMBOLS

Standard Penetration Test Split Spoon (2-inch O.D.)

Modified California Sampler (3-inch O.D.)

Thin-walled Sampler Tube (either Pitcher or Shelby) (3-inch O.D.)

Rock Core

Bag Sample

Groundwater Level

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KEY TO EXPLORATORY BORING LOGS

LBNL B25 AREA PROJECT
Berkeley, California

PROJECT NO.	DATE	FIGURE
2335-12	**May 2009**	**A1**

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

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

BEDDING OF SEDIMENTARY ROCK

Splitting Property	Thickness	Stratification
Massive	Greater than 4.0 feet	Very thick-bedded
Blocky	2.0 to 4.0 feet	Thick-bedded
Slabby	0.2 to 2.0 feet	Thin-bedded
Flaggy	0.05 to 0.2 feet	Very thin-bedded
Shaly or platy	0.01 to 0.05 feet	Laminated
Papery	Less than 0.01 feet	Thinly laminated

FRACTURING

Intensity	Size of Pieces in Feet
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 feet
Intensely fractured	0.05 to 0.1 feet
Crushed	Less than 0.05 feet

HARDNESS

1. **Soft** - Reserved for plastic material alone.
2. **Low Hardness** - Can be gouged deeply or carved easily by 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 by a knife blade with difficulty; scratch produces little powder and is often faintly visible.
5. **Very Hard** - Cannot be scratched by a knife blade; leaves a metallic streak

STRENGTH

1. **Plastic** - 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.

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.

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PHYSICAL PROPERTIES CRITERIA FOR ROCK DESCRIPTIONS		
LBNL B25 AREA PROJECT Berkeley, California		
PROJECT NO.	DATE	FIGURE **A2**
2335-12	**May 2009**	

DRILL RIG: Fraste Multidrill XL, Rotary Wash	SURFACE ELEVATION: 939.7 feet (see notes)	LOGGED BY: SS
DEPTH TO GROUNDWATER: (see notes)	BORING DIAMETER: 4 7/8 inches	DATE DRILLED: 5/18/09

DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	DEPTH (ft)	SAMPLER TYPE	SAMPLER BLOW COUNTS	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
ASPHALT CONCRETE - 2-inches									
GRAVEL, Well-Graded - sandy, dry	Light Brown	Medium Dense	GW	1					
				2					
				3					
-increasing sand, decreasing gravel at 4 feet				4					
(Interval Not Sampled)									
SAND, Gravelly to GRAVEL, Sandy - well graded, fine to medium grained sand with trace silt, dry to moist	Yellowish Brown	Medium Dense	SW-GW	5					
				6		[31]			
				7					
CLAY, Lean - 70% clay matrix, medium to low plasticity, moist (Colluvium or Possible Bedrock)	Reddish Gray	Stiff to Very Stiff	CL	8					
				9					
				10					
SILTSTONE/CLAYSTONE - finely laminated, friable, with thin grayish laminations, low hardness (start of continuous geologic logging at 10.5 feet)	Gray to Reddish Gray		BED ROCK	11		[28]			PP = 1.5 tsf
				12					
				13					PP = 3.5 tsf
				14					PP = >4.5 tsf
-with very fine sandstone gravel, hard clasts	Gray to Brown			15					PP = 4.0 tsf
-with calcite and quartzite angular clasts at 16 feet				16					
				17					
-with volcanic clasts at 18 feet				18					
	Reddish Gray			19					

(Continued on Next Page)

(Continued from Previous Page)

DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	DEPTH (ft)	SAMPLER TYPE	SAMPLER BLOW COUNTS	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
-very weak mixed zone of clayey gravels (volcanics and siltstone) from 19.5 to 21 feet	Brown		BED ROCK						
				21					
				22					
				23					
				24					
-fractured and crushed rock at 25.2 feet				25					PP = 1.5 tsf
-clay rich from 26 to 27 feet	Bluish Gray			26					
				27					
CONGLOMERATE - with up to 70% clasts, with clay and sand matrix, with volcanic clasts	Greenish Gray		BED ROCK	28					
SILTSTONE - with vertical veins of re-crystalized calcite, moderately hard to hard	Red to Gray		BED ROCK	29					
				30					
-becomes coarser with depth and more conglomerate				31					
-becomes hard and cemented with depth				32					
				33					
				34					
-block structure at 35 feet				35					
-clayey at 36 to 36.5 feet				36					PP = 1.5 tsf
				37					
				38					
-massive	Green			39					PP = 4.0 tsf
				40					PP = 2.0 tsf
				41					PP = 3.5 tsf
				42					

(Continued on Next Page)

DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	DEPTH (ft)	SAMPLER TYPE	SAMPLER BLOW COUNTS	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
(Continued from Previous Page)									
SILTSTONE	Red to Gray		BED ROCK	43					
				44					PP = 2.0 tsf
				45					
				46					

Bottom of boring at 46.5 feet.

NOTES:

1. Groundwater levels were obscured due to rotary wash drilling method. (See report for discussion.)

2. Stratification lines represent the approximate boundaries between material types and the transitions may be gradual.

3. Penetration resistance values (blow counts) enclosed in brackets ([ ]) were recorded with a 3.0-inch O.D. Modified California sampler; these are not standard penetration resistance values.

4. Elevations were determined from survey performed by LBNL subcontractor.

5. Approximate unconfined compressive strength values were recorded in the field using a pocket penetrometer. These values are shown on the logs and are preceded by the symbol "PP".

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EXPLORATORY BORING LOG
LBNL B25 AREA PROJECT
Berkeley, California

PROJECT NO.	DATE	SHEET	BORING
2335-12	**May 2009**	**3 of 3**	**AKA-1**

APPENDIX B

Logs of Borings from Previous Geotechnical Reports by Others

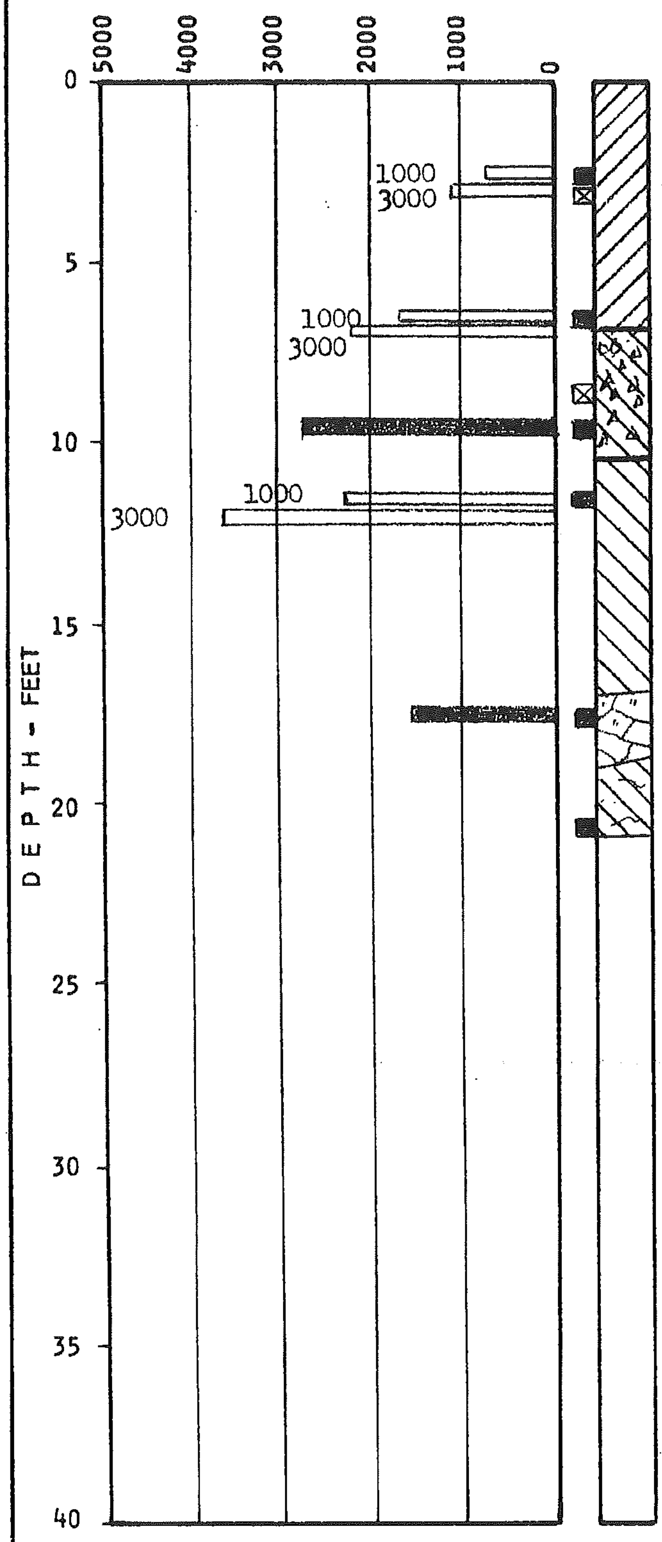
Boring HA (1963) 1
Boring HA (1963) 2

Harding Associates (HA), 1963, "Foundation Investigation, Proposed Building 26, Lawrence Radiation Laboratory, Berkeley 4, California," report dated July 1, 1963 (LBNL File #075).

JOB RADIATION LABORATORY - BLDG. 26 NO 2000.3 BY wea DATE 5-28-63

BORING 26-1

SHEAR STRENGTH - LBS PER SQ FT



MOISTURE CONTENT - %

DRY DENSITY - LBS PER CU FT

ELEVATION 937.6

MOISTURE CONTENT - %	DRY DENSITY - LBS PER CU FT	
		DARK BROWN SILTY CLAY (CH) moderately firm, wet
30.2	83	
		mottled orange- red
28.1	100	
		REDDISH-BROWN CLAY (CL) with many sandstone fragments moderately firm, wet
8.0	128	
16.7	109	ORANGE RED SILTY CLAY (CL) mottled gray - moderaely firm. moist
14.0	117	REDDISH BROWN SILTSTONE with green intrusions - hard, moist
		GREEN SILTY CLAY (CL) with brown intrusions - firm, damp weathered siltstone
14.3	118	
		no free water encountered

FILL

HARDING ASSOCIATES
Soil Mechanics Engineers

LOG OF BORING

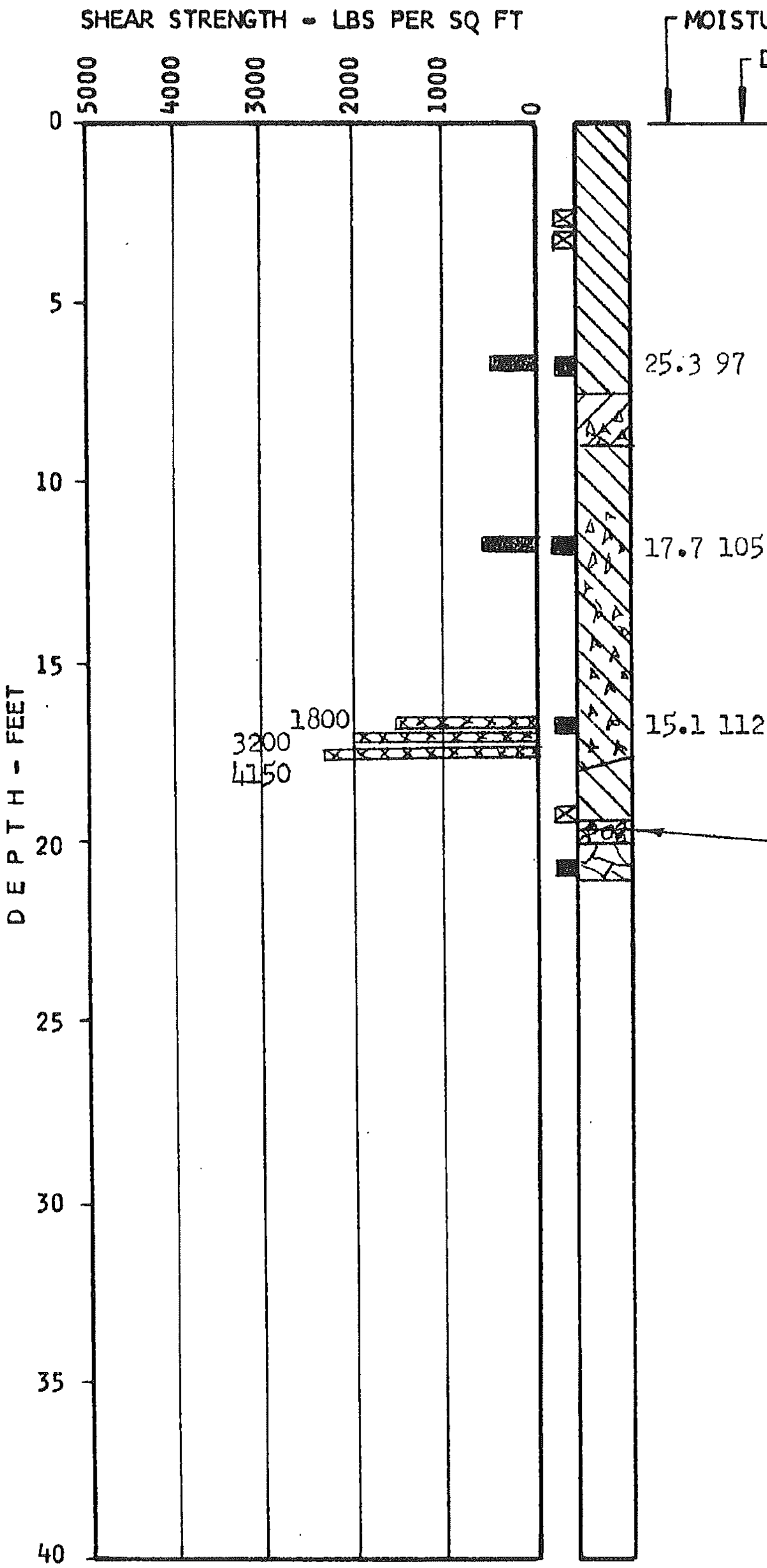
EQUIPMENT 20" Bucket DRILLED 5-16-63

PLATE A 1

075_00020

JOB RADIATION LABORATORY BLDG. 26 NO 2000.3 BY wea DATE 5-29-63

BORING 26-2



REDDISH BROWN SILTY CLAY (CL)
moderately firm, wet
mottled red-gray and green

FILL

DARK BROWN CLAY (CH-CL)
firm, wet
many rock fragments 8.5-9.0'

GRAY-BROWN-GREEN SANDY CLAY (CL)
firm, wet

greenish brown with many angular
rock fragments

GREEN SILTY CLAY (CL)
firm, moist

GREEN CONGLOMERATE LENSE - hard

GREEN SILTSTONE - hard

no free water encountered

HARDING ASSOCIATES
Soil Mechanics Engineers

LOG OF BORING

EQUIPMENT 20" Bucket

DRILLED 5-17-63

PLATE A-2

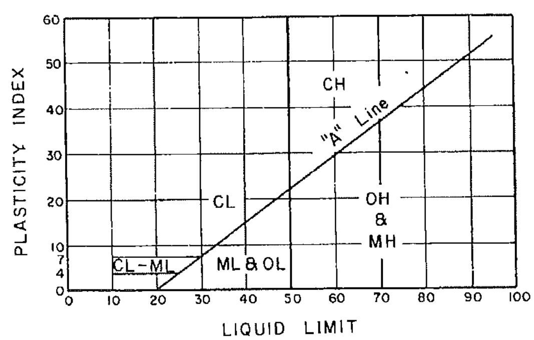
075_00021

MAJOR DIVISIONS		SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS (More than 1/2 of soil > no. 200 sieve size)	GRAVELS (More than 1/2 of coarse fraction > no. 4 sieve size)	GW	Well graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly graded gravels or gravel-sand mixtures, little or no fines
		GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	SANDS (More than 1/2 of coarse fraction < no. 4 sieve size)	SW	Well graded sands or gravelly sands, little or no fines
		SP	Poorly graded sands or gravelly sands, little or no fines
		SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
FINE GRAINED SOILS (More than 1/2 of soil < no. 200 sieve size)	SILTS & CLAYS LL < 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 silts and organic silty clays of low plasticity
	SILTS & CLAYS LL > 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 silty clays, organic silts
HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils

CLASSIFICATION CHART

CLASSIFICATION	RANGE OF GRAIN SIZES	
	U.S. Standard Sieve Size	Grain Size in Millimeters
BOULDERS	Above 12"	Above 305
COBBLES	12" to 3"	305 to 76.2
GRAVEL	3" to No. 4	76.2 to 4.76
coarse	3" to 3/4"	76.2 to 19.1
fine	3/4" to No. 4	19.1 to 4.76
SAND	No. 4 to No. 200	4.76 to 0.074
coarse	No. 4 to No. 10	4.76 to 2.00
medium	No. 10 to No. 40	2.00 to 0.420
fine	No. 40 to No. 200	0.420 to 0.074
SILT & CLAY	Below No. 200	Below 0.074

GRAIN SIZE CHART



PLASTICITY CHART

HARDING ASSOCIATES Soil Mechanics Engineers	UNIFIED SOIL CLASSIFICATION SYSTEM

PLATE A-8

SAMPLE DESIGNATION

UNDISTURBED CORE SAMPLE

REMOLDED SAMPLE OR BULK SAMPLE

STRAIN CONTROLLED TESTS AT FIELD MOISTURE CONTENT

UNCONFINED COMPRESSION TEST

1000 DIRECT SHEAR TEST

1000 TRIAXIAL COMPRESSION TEST

VANE SHEAR TEST

STRAIN CONTROLLED TESTS AT INCREASED MOISTURE CONTENT

1000 (14.5) DIRECT SHEAR TEST

1000 (14.5) TRIAXIAL COMPRESSION TEST

BAR LENGTH REPRESENTS SHEAR STRENGTH

MOISTURE CONTENT WHEN TESTED (PER CENT)

STRESS NORMAL TO THE SHEAR PLANE (LBS. PER SQ. FT.)

∇ INDICATES OBSERVED WATER LEVEL

HARDING ASSOCIATES Soil Mechanics Engineers	KEY TO TEST DATA

PLATE A-5

Boring HMLA (1966) 10
Boring HMLA (1966) 11

Harding Miller Lawson & Associates (HMLA), 1966, "Progress Report, Soil Investigation, Omnitron Site, Lawrence Radiation Laboratory, Berkeley 4, California" report dated August 20, 1966 (LBNL File #008).

JOB U.C. Rad. Lab. - Omnitron Site NO 2000.37 BY AC/gr DATE 8/10/66

BORING 10

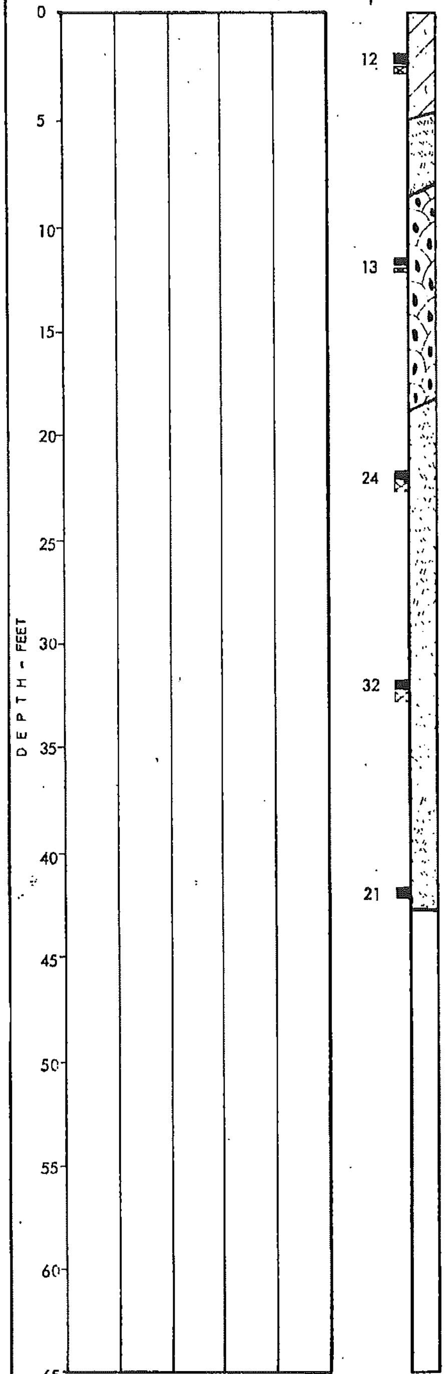
SHEAR STRENGTH - LBS PER SQ FT

STANDARD PENETRATION BLOWS/FOOT

MOISTURE CONTENT - %

DRY DENSITY - LBS PER CU FT

ELEVATION 927



Blows/Foot	Moisture Content %	Dry Density	Description
12	24.1	92	DARK BROWN SANDY CLAY (CL) stiff, wet, with small roots
			GREEN-GRAY SILTSTONE hard, damp, with clay seams
13	17.6	116	GREEN-GRAY CONGLOMERATE hard, wet, with siltstone matrix
24			GREEN-GRAY SILTSTONE with some sand sizes (poorly cemented, scratches and breaks easily)
32	14.3	122	
21	10.4	130	locally very hard and well cemented (no free water encountered)

HARDING ASSOCIATES
SOIL MECHANICS ENGINEERS

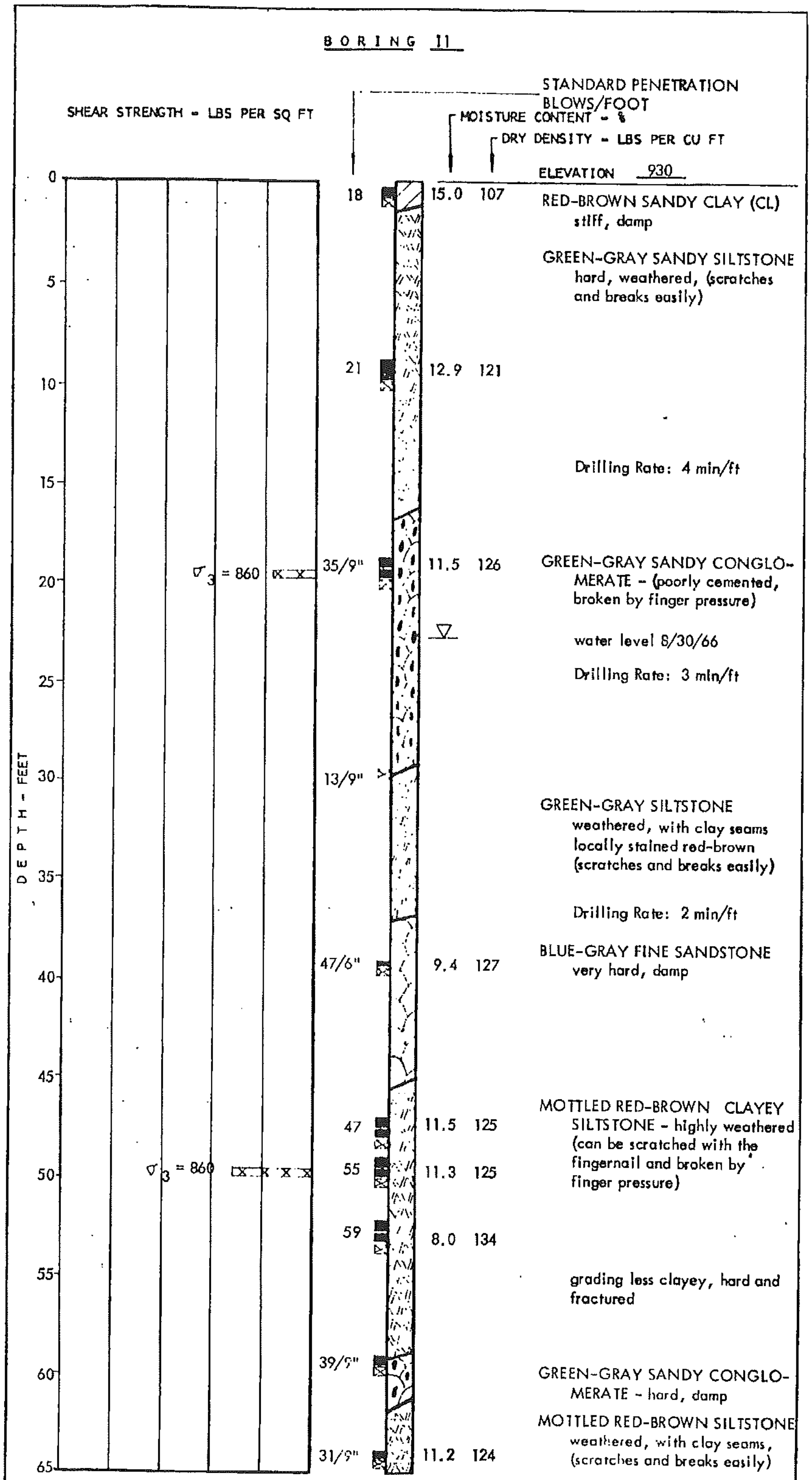
LOG OF BORING

EQUIPMENT 4 5/8" Rotary Wash DRILLED 8/9/66

PLATE A-11

MUB-13805

JOB U.C. Rad. Lab. - Omnitron Site NO 2000.37 BY AC/gr DATE 8/16/66



HARDING ASSOCIATES
SOIL MECHANICS ENGINEERS

LOG OF BORING

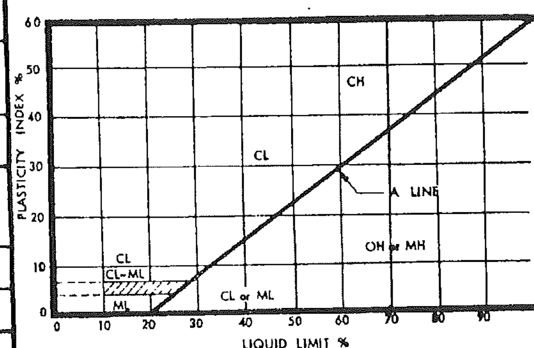
EQUIPMENT 4 3/4" Rotary Wash DRILLED 8/15/66

PLATE A-12

MUB-13806

MAJOR DIVISIONS				TYPICAL NAMES	LABORATORY CLASSIFICATION CRITERIA
COARSE GRAINED SOILS (MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE**)	GRAVELS (MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE SIZE*)	CLEAN GRAVELS (little or no fines)	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	$C_u = \frac{D_{60}}{D_{10}}$ GREATER THAN 4 $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ BETWEEN 1 & 3
			GP	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES, LITTLE OR NO FINES	NOT MEETING ALL GRADATION REQUIREMENTS FOR GW
		GRAVEL WITH FINES (Appreciable amount of fines)	GM	SILTY GRAVELS, POORLY GRADED GRAVEL-SAND-SILT MIXTURES	ATTERBERG LIMITS BELOW "A" LINE OR PI LESS THAN 4 / ABOVE "A" LINE WITH PI BETWEEN 4 AND 7 ARE BORDERLINE CASES REQUIRING USE OF DUAL SYMBOLS
			GC	CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES	ATTERBERG LIMITS ABOVE "A" LINE WITH PI GREATER THAN 7
	SANDS (MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE SIZE*)	CLEAN SANDS (Little or no fines)	SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	$C_u = \frac{D_{60}}{D_{10}}$ GREATER THAN 6 $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ BETWEEN 1 & 3
			SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES	NOT MEETING ALL GRADATION REQUIREMENTS FOR SW
		SANDS WITH FINES (Appreciable amount of fines)	SM	SILTY SANDS, POORLY GRADED SAND-SILT MIXTURES	ATTERBERG LIMITS BELOW "A" LINE OR PI LESS THAN 4 / ABOVE "A" LINE WITH PI BETWEEN 4 AND 7 ARE BORDERLINE CASES REQUIRING USE OF DUAL SYMBOLS
			SC	CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES	ATTERBURG LIMITS ABOVE "A" LINE WITH PI GREATER THAN 7
FINE GRAINED SOILS (MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE**)	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 SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY	
	SILTS AND CLAYS (LIQUID LIMIT GREATER THAN 50)		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACIOUS 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	

Determine percentages of gravel and sand from grain size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size) coarse grained soils are classified as follows: Less than 5% GW, GP, SW, SP; More than 12% GM, GC, SM, SC; 5% to 12% Borderline cases requiring use of dual symbols.



PLASTICITY CHART

For laboratory classification of fine grained soils

*for visual classification, the 1/4 inch size may be used as equivalent to the no. 4 size.
**the no. 200 sieve size is about the smallest particle visible to the naked eye.

UNIFIED SOIL CLASSIFICATION SYSTEM

SAMPLE DESIGNATION

- ■ Undisturbed Sample
- ▣ Disturbed or Bulk Sample
- □ Sample attempt with no recovery
- V Vane Shear Test
- ▽ Observed Water Level

STRENGTH TEST RESULTS

		Symbol	Test
		*	Vane Shear Test
			Unconfined Compression Test
1000	(30.0)		Unconsolidated - Undrained Triaxial Compression Test
1000	(30.0)		Consolidated - Undrained Triaxial Compression Test
1000	(30.0)		Consolidated - Drained Triaxial Compression Test
1000	(30.0)		"Quick" Direct Shear Test
1000	(30.0)		Consolidated - Drained Direct Shear Test

Bar Length Represents Shear Strength
Moisture Content of Shear Zone after Test in Percent
Stress Normal to Shear Plane in PSF

* F denotes in-situ field test
L denotes laboratory vane shear test
(R) denotes remolded vane shear strength

HARDING ASSOCIATES
SOIL MECHANICS ENGINEERS

SOIL CLASSIFICATION CHART AND KEY TO TEST DATA

PLATE A-2

MUB-13815

Boring HLA (1983) 4.169

Harding Lawson Associates (HLA), 1983, "Geotechnical Investigation, Building 17 to 25A Road Realignment, Lawrence Berkeley Laboratory, Berkleley, California," dated March 4, 1983, Job 2000,169.01 (LBNL File #404).

Equipment Flight Auger

Elevation 945 Feet Date 12/8/82

Laboratory Tests	Blows/foot	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	Cored Interval	Depth (ft)	Description
						0	BROWN SANDY CLAY (CL) stiff, dry (COLLUVIUM)
						5	change to very stiff, moist
Shear Strength: 4100 psf (TXUU at 7 psi)	22	25.7	100			10	
						15	
	28					20	
						25	with occasional andesite fragments
	50/5"					30	GREEN SANDY SILTSTONE closely fractured, low hardness, moderately strong, little weathered (ORINDA FM.)
						35	
	45	10.5	122			40	color change to blue-gray with occasional calcite veinlets

(Continuation of Log)

Laboratory Tests	Blows/foot	Moisture Content (%)	Dry Density (pcf)	Core Recovery (%)	Cored Interval	Depth (ft)	Description
						40	(no free water encountered)
						45	
						50	
						55	
						60	
						65	
						70	
						75	
						80	

HLA Harding Lawson Associates
Engineers, Geologists & Geophysicists

Log of Boring 4.169
Lawrence Berkeley Laboratory
Berkeley, California

PLATE 6

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
F. Hamilton	2000,169.01	[signature]	1/83		

404_00017

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 DIATOMACIOUS 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_s — 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

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

Soil Classification Chart and Key to Test Data
Lawrence Berkeley Laboratory
Berkeley, California

PLATE
7

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
	2000,169.01	SRK	2/83		

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 disintigration 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
Lawrence Berkeley Laboratory
Berkeley, California

PLATE
8

DRAWN	JOB NUMBER	APPROVED	DATE	REVISED	DATE
	2000,169.01	[signature]	2/83		

APPENDIX C
Logs of Previous Environmental Borings by LBNL

FUGRO WILLIAM LETTIS & ASSOCIATES, INC.

1777 Botelho Drive, Suite 262
Walnut Creek, CA 94526
Tel: (925) 256-6070
Fax: (925) 256-6076

December 10, 2009

Wayne Magnusen
Alan Kropp & Associates
2140 Shattuck Avenue, Suite 910
Berkeley, CA 94704

RE: DRAFT LBNL Building 25 – Core Review for the General Purpose Laboratory Geotechnical Study

Dear Mr. Magnusen,

This letter presents Fugro William Lettis & Associates, Inc.'s (FWLA) review and evaluation of boring logs and archived soil samples to support Alan Kropp & Associates' (AKA) design-level geotechnical study for the proposed General Purpose Laboratory (GPL) project at the Lawrence Berkeley National Laboratory (LBNL). The scope of this work was developed through multiple conversations with you and is summarized in the AKA proposal submitted to Mr. Richard Stanton dated October 27, 2009.

Proposed GPL site Geology

The proposed GPL site will be located at the site currently occupied by LBNL Building 25 within the "Old Town" area of LBNL, which includes some of the earliest buildings constructed at the LBNL. Although the natural topography in the vicinity of the proposed GPL footprint has been extensively modified by grading, it is apparent that Building 25 is situated atop a roughly north-south trending ridgeline that extends down into Strawberry Canyon (Figure 1). The general area of Building 25 has been graded to an approximately level pad near Elevation +940 feet (LBNL datum); the nose of the ridgeline intersects the floor of Strawberry Canyon at about 500 feet below the level of the building pad.

Regional geologic mapping (Graymer, 2000), recent subsurface investigations, and mapping performed in the general area of Building 25 (WLA, 2009) shows the ridgelines and ridgeline margins include thin colluvium overlying volcanic rocks of the Moraga Formation and sedimentary rocks of the Orinda Formation (See Figure 2 and Plate 1 from WLA, 2009). In addition, a "mixed unit" comprised of both Moraga and Orinda affinity has been interpreted through geologic borings, trenches, and geologic mapping (Parsons, 2000; WLA, 2009). Trenches excavated as part of a "paleolandslide" study for Building 25 characterizes the inferred Tertiary mixed unit as a series of dark reddish brown silty clays and reddish brown gravels. These deposits may have formed closely in time with the initiation of Moraga volcanism near the alluvial depocenter. These previous observations (detailed in trench logs and photographs) significantly aided the re-interpretation of core from the RCRA report (Table1).

Scope of Work

Our scope of work included a review and evaluation of the existing relevant and archived geologic borings near the proposed GPL foot print. The primary objective of the review was to evaluate: 1) the quality of previous boring logs; 2) identify the depth to bedrock; and 3) provide a general characterization of any overlying soils that may be present near the GPL footprint. To accomplish these tasks we compiled existing borehole logs from previous consulting reports (HLA 1963; 1988), LBNL's RCRA facility Investigation (RFI) report (Parsons, 2000), and compared these logs with FWLA's and AKA's previous boring and trench logs (AKA, 2009; WLA, 2009).

On November 30, 2009, Preston Jordan (LBNL) provided FWLA staff (Mr. Baldwin and Mr. Givler) access to the core library at LBNL. On December 1, 2009 David Baskin (LBNL) provided PDF copies of logs described in the RCRA report that cover the proposed development area of the GPL. After compiling these borehole data, FWLA staff reviewed pertinent samples from the LBNL core library on December 2 and 3, 2009. FWLA staff made hand-written notes to the existing boring logs as a means to evaluate the quality of previous soil and rock descriptions, and to place these original observations in context with the current geologic model of the GPL site vicinity. Edited boring logs are provided as Appendix A.

A list of the RCRA borings located in the general GPL site vicinity is provided in Table 1. In total, FWLA Staff reviewed 29 individual samples from 12 borings drilling between 1992 and 1999 (Table 1). The samples available for review included the tips of brass rings, or where possible the extruded sample. In some instances, samples appeared to be missing (possibly submitted for environmental testing) or these samples were not readily found by FWLA in the core library. According to LBNL staff, logs for soil borings SB25-95-1 through SB25A-95-1 are not available. However, Preston Jordan (LBNL) provided a table of field notes compiled during the drilling of these soil borings (Table 2).

Results of Core Review

Overall, the quality of the previous boring logs was generally high. Of the samples reviewed, FWLA's observations generally matched the original lithologic descriptions noted on the logs. In most instances, evaluating physical properties (e.g. stiffness and plasticity) is difficult to impossible because the samples are on the order of 10 to 15 years old and thus, the original moisture content is unknown. In a few instances, FWLA staff disagreed with the textural descriptions provided on the original logs (e.g. silty clay vs. clayey silt), but overall these differences were minor. We should note that some key samples were not located in the core library (borings W25-95-26) and thus, we are unable to evaluate the quality of these boring logs. See Table 1 for more details.

The colluvium overlying the Tertiary bedrock units consists primarily of a clayey SILT with coarse-grained sand, and angular clasts of Orinda and Moraga Formations (See boring log MW2593-15; Appendix A).The thickness of colluvium (or the depth to bedrock) across the GPL footprint is variable and ranges from zero to 16 ft thick. The compiled boring information suggests that colluvium is thin (0-2 ft) along the northern margin of the proposed GPL footprint and increases up to 16 ft thick near the southern third of the footprint. The colluvium abruptly thins (0 to 2 ft thick) directly south of the proposed footprint. Borehole and trench suggest that colluvium coincides with an east-trending buried topographic swale that extends east from WLA trench T-1 to boring MW25-95-26 (See Plate 1 of WLA, 2009).

Closing

We appreciate the opportunity to assist AKA in characterizing the subsurface conditions at the proposal GPL site at LBNL. Please feel free to call (925-256-6070), or email (givler@lettis.com or baldwin@lettis.com) if you have any questions or comments about this letter.

Sincerely,

FUGRO William Lettis & Associates, Inc.

Robert W. Givler, C.E.G. #2533
Senior Geologist

John Baldwin, C.E.G. # 2167
Principal Geologist

REFERENCES

Alan Kropp & Associates, 2009, Slide Investigation Building 25 Area Project (LBNL Subcontract No 6881630), unpublished consulting memorandum with 3 pages and 3 figures; no LBNL file number.

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

Harding-Lawson Associates (HLA), 1964, "Report: Consultation Regarding Foundations Building 62, Lawrence Radiation Laboratory, Berkeley, California," March 1964, Project No. 2000.170.

Harding-Lawson Associates (HLA), 1988, "Foundation Investigation Building 26 Addition, Lawrence Berkeley Laboratory, Berkeley, California," October 31, 1981; LBNL # 269.

Parsons Engineering Science, Inc., 2000, "draft final RCRA Facility Investigation Report for the Lawrence Berkeley National Laboratory Environmental Restoration Program" Prepared for Lawrence Berkeley National Laboratory, dated September 29, 2000.

William Lettis & Associates, Inc. (WLA), 2009, Paleolandslide Investigation Building 25, Lawrence Berkeley National Laboratory Berkeley California,: unpublished consulting report to Lawrence Berkeley National Laboratory, dated August 31, 2009, 41 p. plus figures, plates, and appendices.

Table 1. List of borings surrouding the proposed GPL Footprint

ERP boring	UC East	UC North	Coordinate Source	Logs Available	WLA GIS ID	Ground Elevation (ft)	Elevation Source	Samples Logged by WLA	Intervals Logged
MW26-92-11	3173.89	160.07	?	Yes	451			Yes	8.5-9.0 ft.
MW25-93-15	3057.62	-46.77	survey	Yes	450	936	survey	Yes	3.1-3.6, 8.0-8.3, 23.5-23.8 ft
MW25-94-12	3021.73	24.6	survey	Yes	424	937.93	survey	Yes	4.0-4.2, 9.0-9.5, 14.0-14.2 ft
MW25A-95-4	3033.97	219.82	survey	Yes		938.67	survey	No	
MW25-95-5	3091.6	-154.47	survey	Yes	428	933.24	survey	Yes	3.0-3.1, 93.5-94.0 ft
MW25A-95-15	2960.59	148.22	survey	Yes	427	932.14	survey	No	
MW25-95-26	3139.2	-54.01	survey	Yes	426	936.17	survey	NA*	Could not find samples
MW25-95-27	3045.68	-327.09	?	Yes				No	Could not find samples
SB25-95-1	3065	-31	approximate tape and compass	NA*		937	contour map interpolation	NA*	
SB25-95-2	3131	106	approximate tape and compass	NA*		941	contour map interpolation	NA*	
SB25-95-3	3145	130	approximate tape and compass	NA*		940	contour map interpolation	NA*	
SB25-95-4	3072	-96	tape and compass	NA*		935	contour map interpolation	NA*	
SB25-95-5	3045	-62	tape and compass	NA*		934	contour map interpolation	NA*	
SB25A-95-1	2963	141	tape and compass	NA*				NA*	
SB25A-96-1	2949	127.86	survey	Yes	423	932.55		NA*	
SB25A-96-2	2986.96	99.96	survey	Yes	424?	937.29		Yes	1.5-2.0 ft only sample
SB25A-96-3	3036	107	tape and compass	Yes	421	939	contour map interpolation	Yes	5.5-6.0, 15.2-15.7 ft
MW25A-98-1	2986.86	99.79	survey	Yes	449			Yes	3.0-4.0, 8.7-9.2, 19.0-19.5 ft.
MW25A-98-3	3027.87	175.76	survey	Yes	433			No	Could not find samples
MW25A-98-6	3091.47	134.29	survey	Yes	448			Yes	4.0-4.5, 10.5-11.0, 40.8-41.3 ft
MW25A-98-7	3001.67	140.51	survey	Yes				Yes	15.6-16.1, 20.3-20.8, 30.6-31.1 ft
MW25-98-10	3087.97	-105.23	survey	Yes	425			Yes	20.2-20.7 ft - only sample
MW25A-99-2	3037.07	137.7	survey	Yes				Yes	5.5-6.0, 10.5-11.0 ft
MW25A-99-5	3062.06	166.42	survey	Yes	434			Yes	5.5-6.0, 7.6-8.1, 13.9-14.4, 23.5-24.0, 33.9-44.4 ft
MW25A-00-5	2965.28	139.64	survey	Yes	422			No	
SB25A-02-1	3080.73	217.66	?	Yes				No	

*NA - Boring logs were not available and samples were not logged or not available.

Table 2. List of Building 25 soil boring data where logs are unavailable.

Boring	Logged by	Field notes?	Date drilled	UC East	UC North	Coordinate Source	Datum	Ground surface elevation	Ground surface elevation comment	Qaf thick.	Depth to Qc top	Elev. of Qc top	Qc thick.	Depth to Moraga material	Elev. of Moraga material top	Moraga material thick.	Depth to Mixed Unit top	Elev. of Mixed Unit top	Mixed Unit thickness	Depth to Orinda material	Elev. of Orinda material top	BOH depth	BOH elev.
SB25-95-1	pdj	yes	21-Apr-95	3065	-31	approximate tape and compass	UC	937	contour map	1	1	936	7	8	929							16	921
SB25-95-2	pdj	yes	21-Apr-95	3131	106	approximate tape and compass	UC	941	contour map	1	1	940	0	1	940							10	931
SB25-95-3	pdj	no	19-Apr-95	3145	130	approximate tape and compass	UC	940	contour map	0	0	940	0	0	940	35	35	905	1	36	904	37	903
SB25-95-4	pdj	no	19-Apr-95	3072	-96	tape and compass	UC	935	contour map	1	1	934	2	3	932							13	922
SB25-95-5	pdj	yes	21-Apr-95	3045	-62	tape and compass	UC	934	contour map	1	1	933	8	9	925							13	921
SB25-95-6	sf	yes	19-Dec-95	3027	-290	tape and compass	UC	862	contour map	1	1	861	7							8	854	13	849
SB25A-95-1	pdj	no	19-Apr-95	2963	141	tape and compass	UC	933	contour map	0	0	933	2							2	931	46	887

*Table provided by P. Jordan via email December 2, 2009.

Monitoring Well Log from 1992

MW26-92-11

DRILL RIG:	SURFACE ELEVATION:	LOGGED BY:
Mobile B-61	≈ 941 feet	Preston Holland
DEPTH TO GROUNDWATER: ≈18 feet	BORING EQUIPMENT: 8.5" hollow-stem auger	DATE DRILLED: 3/9/92

DESCRIPTION AND CLASSIFICATION

DESCRIPTION AND REMARKS	DEPTH (FEET)	Penetration Resistance (Blows/ft)	REMARKS
CLAYEY GRAVEL (GC), red-brown	0		Traffic-grade well cover with locking cap
CLAYEY GRAVEL (GC), volcaniclastic; mottled green-grey, orange-brown, and purple-black;low to moderate hardness; plastic to moderately strong; subvertical clay seams; moist to WET (deeply weathered ?andesite)	5	56	Cement grout seal
SANDY CLAYSTONE, green-grey and red-brown; subhorizontally sheared; plastic to weak; soft to low hardness; crushed; deeply to moderately weathered; calcite veinlets; WET	10	29	2" PVC solid casing Sch. 40, 10' sections, O-ring seals
SANDY SILTSTONE, grey-green; intensely subvertically fractured; moderately weathered; low to moderate hardness; friable to weak; WET	15	62	Bentonite pellets seal
SILTY SANDSTONE, green-grey; fine to medium grained; intensely to closely fractured; moderately weathered; calcite-cemented; moderately hard; weak to moderately strong; WET to SATURATED • drilling slower 21-25 ft • at 25 ft: red-brown; low to moderate hardness; friable to weak	20 25	71 40/3.5"	No. 1/20 sand filter pack Machine-slotted PVC well screen, slot size 0.010"
SANDY SILTSTONE, grey-green; moderately fractured; moderately weathered; plastic to weak; low hardness; SATURATED	30	40/2"	Silt trap
Total Drilled Depth = 31 ft	35, 40, 45, 50		

To

LAWRENCE BERKELEY LABORATORY	EXPLORATORY BORING LOG		
	PROJECT #: Site Restoration Project	DATE: 3/20/92	BORING #: 26-92-11

Soil Boring Logs from 1996

SB25A-96-2
SB25A-96-3

BORING #	SURF ELEV:	UC EAST:	UC NORTH:	LOGGED BY:	DATE DRILLED:	PAGE:
SB25A-96-2	feet	feet	feet	Preston Holland	7/31/96	1/1

DEPTH TO GROUNDWATER:	DRILLING CONTRACTOR:	BORING EQUIPMENT:	DRILL RIG:
not equilibrated	Gregg	6" hollow-stem auger	B-61

DESCRIPTION AND CLASSIFICATION

DESCRIPTION AND REMARKS	SOIL TYPE	DEPTH (FEET)	SAMPLE	Standard Penetration Resistance (Blows/ft)	ambient FID readings (ppm)	REMARKS	Well Construction
CONCRETE							
SILTY CLAY (CH), brown, stiff, moist, minor medium to coarse, subround to subangular, deeply weathered, andesitic gravel.				24		Bentonite chip	
SILTY SANDSTONE, greenish gray with brown mottling and subhorizontal layering, closely fractured, friable, low hardness, moderately weathered, moist.		5		45/10"			
SANDY SILTSTONE, greenish gray with red brown mottling and zones, closely fractured or less, friable, low hardness, little to moderately weathered, moist, some zones without sand.		10 15		37 39		2" PVC solid casing, Sch. 40, 10' sections, O-ring seals	
SANDSTONE, gray, closely fractured, weak, moderately hard, moderately weathered, moist, fine to medium.		20		51/11"			
SILTY SANDSTONE, greenish gray with red brown zones, closely to intensely fractured, weak, moderately hard, little weathered, moist, fine.		25		48			
SANDSTONE, bluish gray, closely fractured, weak, moderately hard, little weathered, moist to wet, calcite veins. • becomes silty, closely to intensely fractured, without calcite veins, with occasional red brown sandy siltstone zones with depth • saturated at 35 ft		30 35 40		42/10" 51/10" 27		Machine-slotted PVC well screen, slot size 0.020" Silt trap	
Total depth = 44.0 feet		45 50					

all samples taken with 2" ID split spoon

LAWRENCE BERKELEY NATIONAL LABORATORY	EXPLORATORY BORING LOG		
	PROJECT # Site Restoration Project	DATE: 8/29/96	BORING # **SB25A-96-2**

BORING #	SURF ELEV:	UC EAST:	UC NORTH:	LOGGED BY:	DATE DRILLED:	PAGE:
SB25A-96-3	feet	feet	feet	Preston Jordan	8/23/96	1/1

DEPTH TO GROUNDWATER:	DRILLING CONTRACTOR:	BORING EQUIPMENT:	DRILL RIG:
not equilibrated	Clear Heart	4" solid-stem auger	RP-89

DESCRIPTION AND CLASSIFICATION

DESCRIPTION AND REMARKS	DEPTH (FEET)	Penetration Resistance (Blows/ft)	REMARKS
open concrete swale to ~1.5 feet below boardwalk			
CONCRETE			Bentonite chip
TUFFACEOUS SILTSTONE, red orange to orangish red brown, crushed, friable, low hardness, little weathered, dry, minor deeply weathered predominantly andesitic and tuffaceous, fine gravel to medium sand occurs in occasional red brown zones, minor carbon-rich laminae, subhorizontal structure.	5	19	
	10	39	
TUFFACEOUS SILTY SANDSTONE, brown to purplish brown, intensely fractured, friable, low to moderate hardness, little weathered, dry, fine, faint structure (bedding?) dipping ~25°. • brownish purple, medium grained interbeds at 20 ft	15	88	2" PVC solid casing, Sch. 40, 10' sections, O-ring seals
SANDSTONE, greenish gray, intensely to closely fractured, friable to weak, low to moderate hardness, little weathered, wet, fine, minor silt.	20	49	
• blue gray below 25.2 ft	25	81/11"	
• red brown mottled below 30 ft	30	70/6"	
	35	75/5"	Machine-slotted PVC well screen, slot size 0.010"
• bluish gray, saturated below 37 ft			
	40	77/5"	Silt trap Slough
Total depth = 40.4 feet			
	45		
	50		

all samples taken with 2" ID split spoon

Columns: SOIL TYPE, SAMPLE, ambient FID readings (ppm), Well Construction

LAWRENCE BERKELEY NATIONAL LABORATORY

EXPLORATORY BORING LOG

PROJECT #	DATE:	BORING #
Site Restoration Project	10/10/96	SB25A-96-3

Monitoring Well Logs from 1998

MW25A-98-1
MW25A-98-6
MW25A-98-7

ERNEST ORLANDO LAWRENCE BERKELEY NATIONAL LABORATORY

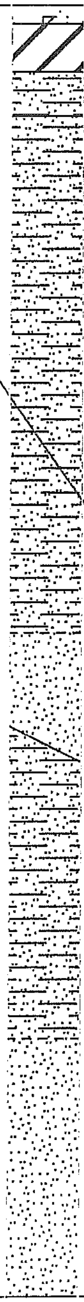
BERKELEY LAB

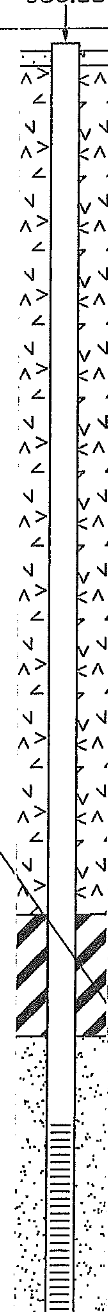
25A-98-1	2986.86	99.79	934.70'

West of the north end of 25

04/23/98 – 04/23/98	8.00in	50.00'	50.00'	This well replaced and deepened temporary well SB25A-96-2. Soil samples shown are from SB25A-96-2.
M-11 with hollow stem auger			PDJ	
Gregg	LBNL-ERP			

GEOLOGICAL MATERIAL DESCRIPTION	GRAPHIC LOG	DEPTH (ft)	RECOVERY	BLOW COUNT	ELEVATION (ft)	MP.EL. 936.88	WELL MATERIALS
CONCRETE							christy box set in concrete
SILTY CLAY (CH), brown, stiff, moist, minor medium to coarse, subround to subangular, deeply weathered, andesitic gravel.				8 18 24 2SS			
SILTY SANDSTONE, greenish gray with brown mottling and subhorizontal layering, closely fractured, friable, low hardness, moderately weathered, moist.		5		30 50 R/4" 2SS	930		
SANDY SILTSTONE, greenish gray with red brown mottling and zones, closely fractured or less, friable, low hardness, little to moderately weathered, moist, some zones without sand.		10		15 25 40 2SS			cement grout
		15		12 35 35 2SS	920		2" ID, Schedule 40 PVC casing
SANDSTONE, gray, closely fractured, weak, moderately hard, moderately weathered, moist, fine to medium.		20		40 50 R/5" 2SS			
SILTY SANDSTONE, greenish gray with red brown zones, closely to intensely fractured, weak, moderately hard, little weathered, moist, fine.		25		35 50 2SS	910		3/8" bentonite pellets
SANDSTONE, bluish gray, closely fractured, weak, moderately hard, little weathered, moist to wet, calcite veins.		30		25 50 R/4" 2SS			#2/12 sand
– becomes silty, closely to intensely fractured, without calcite veins, with occasional red brown sandy				20 40 50 R/4" 2SS	900		0.010" slotted screen





ENVIRONMENTAL RESTORATION PROGRAM

ERNEST ORLANDO LAWRENCE BERKELEY NATIONAL LABORATORY

BERKELEY LAB

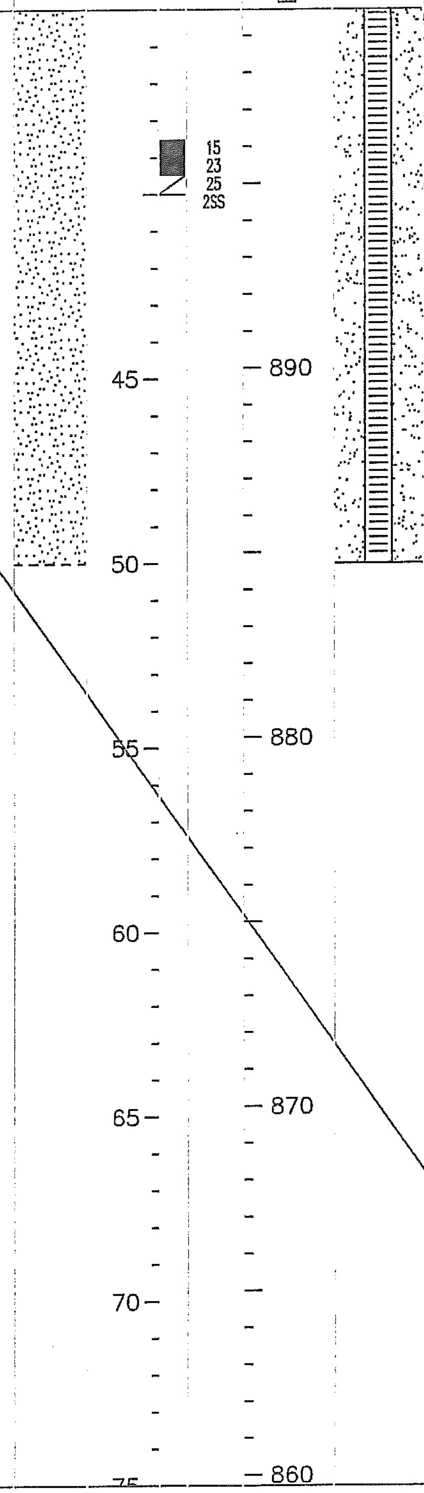
25A-98-1	2986.86	99.79	934.70'

West of the north end of 25

GEOLOGICAL MATERIAL DESCRIPTION	GRAPHIC LOG	DEPTH (ft)	RECOVERY	BLOW COUNT	ELEVATION (ft)		WELL MATERIALS

siltstone zones with depth

– saturated at 35 feet



ERNEST ORLANDO LAWRENCE BERKELEY NATIONAL LABORATORY

25A-98-6	3091.47	134.29	940.70'

Inside the east end of 25A

10/01/98 – 10/02/98	6.00in	42.00'	40.80'	Bridging problems due to highly turbid formation water in borehole reduced sand volume in filter pack one quarter from expected volume.
8x8 with solid stem auger			PDJ	
Clear Heart		LBNL-ERP		

GEOLOGICAL MATERIAL DESCRIPTION	DEPTH (ft)	BLOW COUNT	ELEVATION (ft)	WELL MATERIALS
CONCRETE			940	christy box set in concrete
GRAVEL (GP), base rock (fill).				
TUFF BRECCIA, reddish brown with white and green mottles, closely fractured or less, friable, low hardness, little weathered, moist, includes fine to medium gravel of angular gray siltstone and rounded lithics, contains calcite inclusions.	5	12 17 31 2SS; 5 13 29 2SS		cement grout
– red brown at 9 feet				2" ID, Schedule 40 PVC casing
– greenish white and purplish gray at 11 feet	10	9 39 51 R/5" 2SS	930	medium bentonite chip
ANDESITE, light orange brown, intensely fractured, weak to moderately strong, moderately hard, deeply weathered, moist, black vug fillings.				
– andesite supported in a dark gray to black sheared clay matrix from 12 to 17 feet	15	9 43 51 R/5" 2SS		
– grades to gray brown, closely fractured, friable to weak with depth				#2/12 sand
	20	12 28 51 R/3" 2SS	920	
– wet at 23 feet				centralizer
	25	12 26 33 2SS		0.010" slotted screen
	30	36 51 R/4" 2SS	910	centralizer

MP.EL. 939.90

Ernest Orlando Lawrence Berkeley National Laboratory

Berkeley Lab

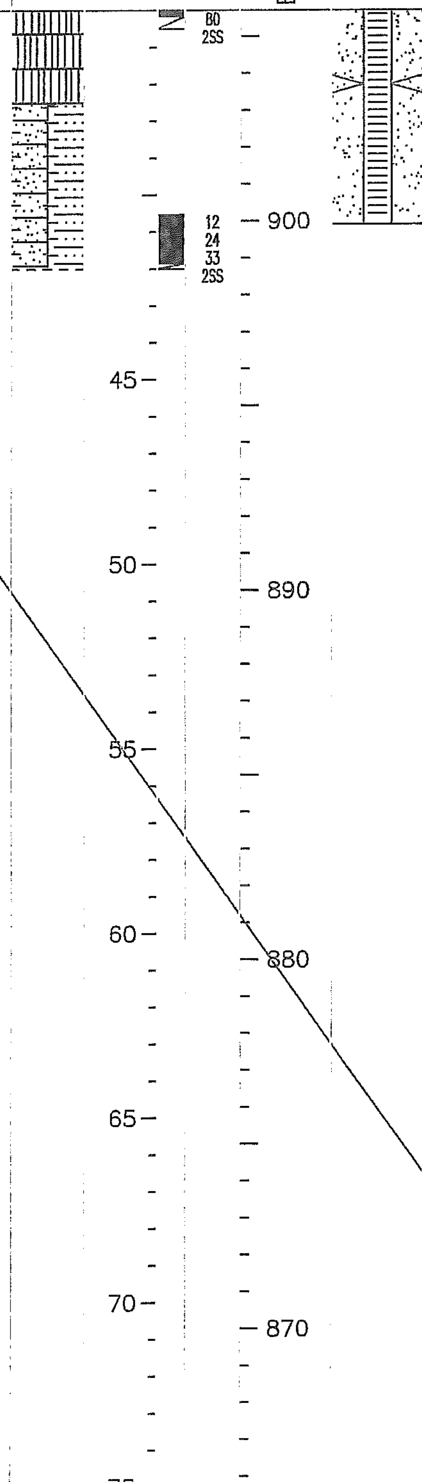
25A-98-6 3091.47 134.29 940.70'

Inside the east end of 25A

GEOLOGICAL MATERIAL DESCRIPTION	GRAPHIC LOG	DEPTH (ft)	RECOVERY	BLOW COUNT	ELEVATION (ft)	WELL MATERIALS

BASALT, black, closely fractured, strong, hard, moderately weathered, saturated.

SILTY SANDSTONE AND SILTSTONE, blue gray and red brown, laminated to thin bedded, crushed to intensely fractured, friable, little weathered, saturated, sheared, slickensides dip ~35 degrees, includes minor brown claystone.



ERNEST ORLANDO LAWRENCE BERKELEY NATIONAL LABORATORY

BERKELEY LAB

25A-98-7	3001.67	140.51	940.10'

Between 25A and 44A

09/01/98 - 09/01/98 | 6.00in | 36.00' | 34.00'

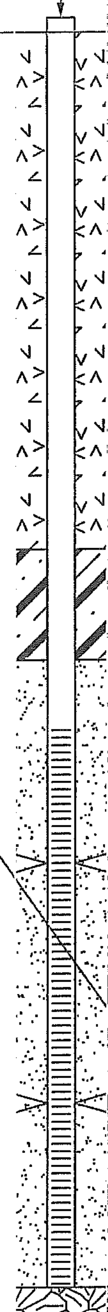
C&RP-98 with solid stem auger | PDJ

Gregg | LBNL-ERP

Note refusal encountered at original well location (SB25A-98-2) ~6 feet to the south. Refusal occurred at 9 feet in a calcite cemented sandstone.

GEOLOGICAL MATERIAL DESCRIPTION	DEPTH (ft)	BLOW COUNT	ELEVATION (ft)	WELL MATERIALS
SILTY GRAVEL (GM), reddish brown and gray, medium dense to dense, moist, andesitic gravel.			940	stand pipe
SILTSTONE, reddish brown, crushed, friable to weak, low hardness, little weathered, moist, calcite veined.	5			cement grout
	10	13 50 2SS	930	2" ID, Schedule 40 PVC casing
SILTY SANDSTONE, gray, intensely fractured, weak, low hardness, moderately weathered, moist.				
CLAYEY SILTSTONE AND SANDY SILTSTONE, brown and green gray, crushed, plastic, low hardness, little weathered, moist to wet.				medium bentonite chip
SANDY SILTSTONE WITH GRAVELLY SANDSTONE, light brownish gray with brown, friable, intensely to closely fractured, low hardness, little to moderately weathered, wet to saturated, fine to medium, subangular gravel, fine sand with minor silt.	15	20 23 30 2SS		
- GRAVELLY SANDSTONE not present	20	40 50 2SS	920	#2/12 sand
- SANDY SILTSTONE grades to SILTY SANDSTONE with minor SILTSTONE, reddish brown, weak with depth				centralizer
	25	12 14 25 2SS		0.010" slotted screen
CLAYSTONE WITH SANDY SILTSTONE, red brown with blue gray, crushed, plastic to moderately strong, low to moderate hardness, little weathered, saturated, calcite veined.				centralizer
	30	20 40 51 R/5" 2SS	910	
- hard drilling at 32 feet				
SANDSTONE, reddish brown, crushed, moderately strong, moderately hard, little weathered, saturated, calcite				slip cap slough

MP.EL. 942.71



ENVIRONMENTAL RESTORATION PROGRAM

25A-98-7	3001.67	140.51	940.10'

Between 25A and 44A

ERNEST ORLANDO LAWRENCE BERKELEY NATIONAL LABORATORY

GEOLOGICAL MATERIAL DESCRIPTION	GRAPHIC LOG	DEPTH (ft)	RECOVERY	BLOW COUNT	ELEVATION (ft)	WELL MATERIALS
cemented.				50 2SS		
SILTY SANDSTONE, blue gray, closely fractured or less, weak, low to moderate hardness, little weathered, saturated.					900	
		45				
		50			890	
		55				
		60			880	
		65				
		70			870	
		75				

Monitoring Well Logs from 1999

MW25A-99-2
MW25A-99-5

ERNEST ORLANDO LAWRENCE BERKELEY NATIONAL LABORATORY

25A-99-2	3037.07	137.70	940.69'

Inside the west end of 25A

05/01/99 - 05/01/99	8.00in	31.50'	30.00'	Boring initially drilled with 5 1/2" augers and then reamed with 8" augers. Boring drilled to 30.0 feet only. No blow counts due to use of hydraulic percussion hammer to drive sampler.
M5T with hollow stem auger			PDJ	
Gregg		LBNL-ERP		

GEOLOGICAL MATERIAL DESCRIPTION	GRAPHIC LOG	DEPTH (ft)	RECOVERY	BLOW COUNT	ELEVATION (ft)	MP.EL. 940.45	WELL MATERIALS

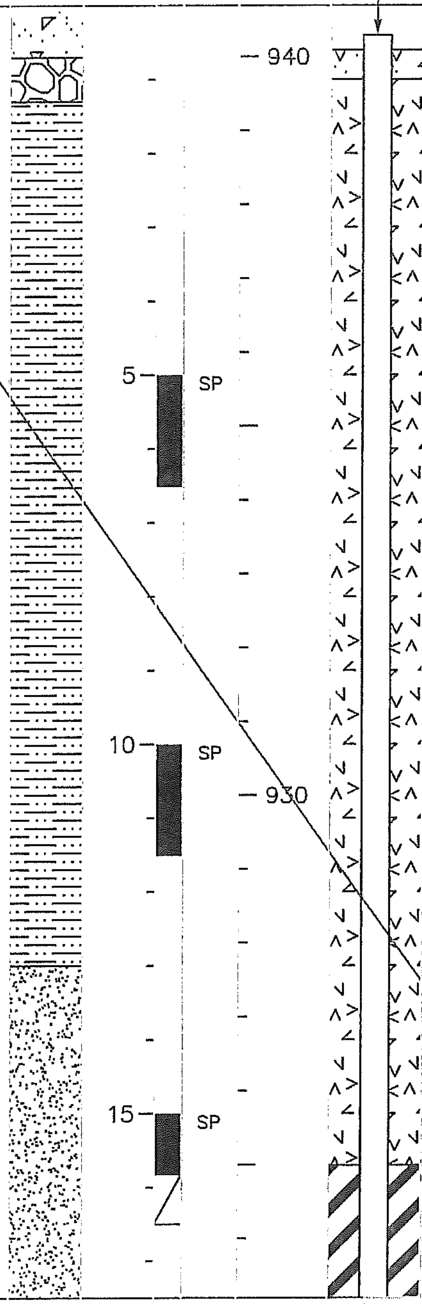
CONCRETE

GRAVEL (GP), base rock (fill).

SILTSTONE, orange brown, less than closely fractured, plastic to friable, soft to low hardness, little weathered, moist, bedding dips ~70 degrees, includes purplish brown with siltstone beds with greenish white mottling sandy which contains coarse-grained sand and minor fine-grained gravel of volcanics as well as clasts of tuff, also includes thin beds of brown sandstone.

- cuttings change at 13 feet

SANDSTONE, gray, less than closely fractured, weak to moderately strong, low to moderate hardness, little weathered, wet to saturated, sand medium-grained, angular, and predominantly basaltic, includes beds with fine to medium-grained, predominantly basaltic gravel, also includes red brown mottled, silty, tuffaceous thin beds.



christy box set in concrete

cement grout

2" ID, Schedule 40 PVC casing

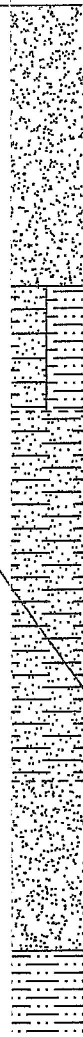
3/8" bentonite pellet

ERNEST ORLANDO LAWRENCE
BERKELEY NATIONAL LABORATORY

25A-99-2 3037.07 137.70 940.69'

Inside the west end of 25A

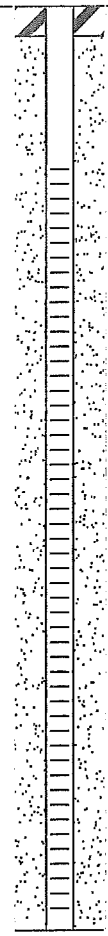
GEOLOGICAL MATERIAL DESCRIPTION	GRAPHIC LOG	DEPTH (ft)	RECOVERY	BLOW COUNT	ELEVATION (ft)	WELL MATERIALS



SILTY SANDSTONE AND SILTSTONE, greenish gray and reddish brown, closely fractured, friable, low hardness, little weathered, saturated, sandstone fine-grained.
– free water in boring at 22 feet
SILTY SANDSTONE, blue gray, less than closely fractured, friable, low hardness, little weathered, saturated, fine grained, occasional moderately strong to strong, hard, calcite-cemented zones.

SANDSTONE, dark blue gray, closely fractured, friable to weak, low to moderate hardness, little weathered, saturated, medium grained, minor silt matrix, occasional strong, hard, calcite-cemented zones.
SILTSTONE, reddish brown with blue gray mottling, closely fractured, weak, moderately hard, little weathered, saturated.

20 SP
920
25 SP
30 SP
910



#2/12 sand
0.010" slotted screen
silt trap

GEOLOGICAL MATERIAL DESCRIPTION	GRAPHIC LOG	DEPTH (ft)	RECOVERY	BLOW COUNT	ELEVATION (ft)	WELL MATERIALS
				50 2SS		
		20			920	3/8" bentonite pellet
- wet to saturated at 24 feet - shear zone with red brown and green white silt laminae and purple gray volcanic sand laminae dipping ~20 degrees at contact				15 10 13 2SS		#2/12 sand
SANDSTONE, red brown, crushed to intensely fractured, friable to weak, moderately hard, little weathered, saturated, fine grained. - becomes blue gray, fine to medium grained below 27 feet		25				centralizer
				60 2SS		
		30			910	2" ID, 0.010" slotted screen prepacked with #2/16 sand inside a 4" ID, 0.020" slotted screen
				30 30 40 2SS		centralizer

GEOLOGICAL MATERIAL DESCRIPTION	GRAPHIC LOG	DEPTH (ft)	RECOVERY	BLOW COUNT	ELEVATION (ft)	WELL MATERIALS

– silty with thin, subhorizontal beds of red brown siltstone at 39 feet