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Mr. Richard Stanton
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RE: Geotechnical Investigation Report Supplement
Roadway Widening
General Purpose Laboratory at B25 Site
Lawrence Berkeley National Laboratory
Berkeley, California
(Subcontract No. 6859200)

Dear Mr. Stanton:

This supplement presents the results of a geotechnical investigation by Alan Kropp & Associates, Inc. (AKA) for the proposed widening of the existing roadway southeast of the future General Purpose Laboratory (GPL). We are currently providing services on this project under Lawrence Berkeley National Laboratory (LBNL) Architect-Engineer Subcontract No. 6859200, Modification No. 4.

This geotechnical letter report supplements our April 2, 2010 geotechnical investigation report for the GPL and relates to features shown on the 100% Design Development (DD) drawings prepared by RMW Architecture & Interiors. The approximate locations of the features discussed are shown on the attached Supplemental Investigation Site Plan, Figure 1.

INTRODUCTION

The GPL will be located at the site currently occupied by Building 25, which will be demolished prior to the construction of the project. As shown on the 100% DD drawings, the GPL will be a three-story building situated on a nearly level building pad with a ground floor level at or near the elevation of the existing ground surface. The approximate location of the GPL is indicated (in yellow) on Figure 1. Previous borings drilled at the site indicate that: (1) bedrock is present at or near the elevation of the ground surface in the northwest portion of the GPL; and (2) bedrock is present about 15 feet below the ground surface towards the southeast corner of the GPL.

As shown on the 100% DD drawings, the outboard side of the perimeter access road will be widened by several feet adjacent to the southeast corner of the GPL. The approximate area of the widened roadway is shown (in green) on Figure 1. The area of roadway widening is situated at the top of an east-facing hillslope; Sheet C4 of the 100% DD drawings shows a new retaining wall along the outboard edge of the widened roadway that is about 90 feet long and about ½ foot to 2½ feet high.

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This supplement focuses upon the localized area outlined in red on Figure 1. The primary purpose of our supplemental geotechnical investigation was to document the geotechnical conditions present within the study area and provide geotechnical conclusions and recommendations for the proposed roadway widening. The scope of our services included:

- Reviewing existing geotechnical and geologic information;
- Characterizing subsurface conditions;
- Performing geotechnical engineering analyses;
- Developing geotechnical engineering recommendations; and
- Preparing this geotechnical investigation report supplement.

Please note that it was not a purpose of this study to discuss chemical constituents in the onsite soil or groundwater, or to provide recommendations pertaining to soil handling/disposal, or other environmental aspects of the proposed work.

METHODS OF INVESTIGATION

We investigated subsurface conditions in the vicinity of the planned roadway widening by: (1) reviewing existing data from two previous borings drilled within the study area; and (2) drilling one new exploratory boring (AKA-4). The approximate locations of known borings in the site vicinity are shown on Figure 1.

Our April 2, 2010 geotechnical investigation report for the GPL includes appendices containing logs of previous borings, two of which are within our current study area. Boring HLA (1988) 1 was drilled by Harding Lawson Associates (HLA) as part of a geotechnical investigation for an addition to Building 26. Boring MW25-95-26 was drilled by LBNL's Environmental Health & Safety (EH&S) division as part of onsite geologic characterization and monitoring well installation activities. Logs of these two borings are in Appendix B and C, respectively, of our April 2, 2010 geotechnical investigation report for the GPL.

Boring AKA-4 extended to a depth of approximately 30½ feet and was drilled in April 2010 using a truck-mounted drill rig equipped with continuous flight augers. An AKA engineer supervised the drilling of AKA-4, logged the soils and bedrock encountered, and obtained samples of the subsurface materials for subsequent evaluation. Samples were obtained using a 2-inch outside diameter (O.D.) Standard Penetration Test (SPT) sampler without liners and a 3-inch O.D. Modified California sampler with liners. Both samplers were advanced with a standard 140-pound hammer falling 30 inches using a rope and cathead system. Following drilling, the boring was backfilled to near the ground surface with grout and the pavement was patched. The location and elevation of the boring was later surveyed by Bates & Bailey Land Surveyors, Inc. (B&B) of Berkeley, California, an LBNL subcontractor.

Soil and rock samples were examined in our laboratory to check field classifications and to select suitable specimens for laboratory testing. Soils were classified in general accordance with ASTM D2487, which is described on the attached the Key to Exploratory Boring Logs (Figure 2). Rock materials were evaluated and described in general accordance with the Physical Properties Criteria for Rock Descriptions (Figure 3). John Baldwin (C.E.G.) of Fugro William Lettis & Associates assisted in the geologic classification of materials obtained from the borings. Geotechnical laboratory tests performed for this study consisted of Moisture Content (ASTM D-2216), Dry Density (ASTM D-2937) and Atterberg Limits determinations (ASTM D-4318) involving soil plasticity. The log of Boring AKA-4 is attached and includes material descriptions, sampler blow counts, laboratory test results, and groundwater depth information (where encountered).

When referring to the attached boring log, please note that the log depicts subsurface conditions only at the specific boring location on the date that the boring was drilled. The passage of time may result in changes in the subsurface conditions. In addition, the sampler blow counts presented on the log are field values obtained using the specific sampler and hammer combination indicated. Care should be exercised in making comparisons of sampler blow counts between borings or with published data (blow counts obtained using the Modified California sampler are higher than what would be obtained using an SPT sampler under the same conditions).

SURFACE CONDITIONS

Figure 4 presents two north-facing photographs that show the general area of the proposed roadway widening. In each photograph, Building 25 can be seen in the upper left-hand corner and Building 26 can be seen in the distance in the upper right. The right-hand side of the photographs generally show that the area of the roadway widening is located at the top of an east-facing hillslope, which is inclined at about 2:1 (horizontal to vertical). At the time of our investigation (April 2010) the hillslope was generally covered in grassy vegetation and included numerous small trees; several stumps from larger trees were also observed on the slope that are not apparent in the photographs.

The photographs generally show that the roadway and parking area directly along the east side of Building 25 has been resurfaced with new asphalt concrete that appears in generally good condition; whereas south of Building 25 (i.e., in the foreground of the photographs presented on Figure 4) the pavement surface exhibits signs of significant distress. Both photographs show a substantially thickened section of pavement along the outboard edge of the roadway in the general area of the proposed roadway widening and the asphalt concrete curb on top of the thickened pavement section has cracked and deflected. The lateral extent of the thickened pavement section was not able to be directly observed as it is masked by the pavement resurfacing. However, about 18 inches of asphalt concrete was encountered in Boring AKA-4, which was drilled in the parking strip east of Building 25.

Pavement markings present at the time of our investigation (Figure 4) indicate that there are utility lines buried beneath the existing roadway, including a high-pressure natural gas (HPNG) line that crosses the area of the proposed roadway widening. The HPNG line markings (in yellow) can be seen on Figure 4 at the southern end of the newly-repaved area; similar markings are also present at the base of the hillslope east of the study area. Various utility marking flags and the vertical riser of a sprinkler can also be seen on the slope in the vicinity of the proposed roadway widening (Figure 4).

SUBSURFACE CONDITIONS

The study area is situated along the eastern side of a ridgeline underlain by resistant volcanic rock of the Moraga Formation. Previous grading to construct the relatively level pad upon which Building 25 is situated included lowering grades (by cutting) in the north and raising grades (by filling) in the south. The subsurface material present in the current study area, located near the southeast corner of Building 25, includes fill, natural soil and rock. A recent geologic study (WLA 2009) indicates that the rock materials upon which Building 25 is situated are considered geologically stable; in this report, we refer to these rock materials as bedrock.

Bedrock Conditions

The bedrock materials present in the project vicinity consist of Moraga Volcanics (including andesite and tuff or agglomerate breccias) and Orinda Formation (including siltstone, sandstone, claystone and conglomerate).

The bedrock layers that exist within the Moraga Formation are highly variable and include materials that are harder and more resistant than the Orinda Formation. The three borings drilled within the study area encountered varying thicknesses of soil directly overlying volcanic rock of the Moraga Formation. In Boring HLA (1988) 1, the upper 5 feet of rock is logged as Moraga Formation (andesite); with red-brown claystone of the Orinda Formation logged below it. In Boring MW25-95-26, "rock weathered to clay" is noted at a depth of about 10 feet, with "volcanic rock" logged below about 20 feet. In Boring AKA-4, the surface of bedrock was logged at a depth of 15.5 feet.

Interpreted bedrock depths are summarized in the table that follows together with the corresponding "top of bedrock" elevation. A map depicting interpreted elevation contours of the top of the bedrock surface is presented on Figure 5.

Interpreted Top of Bedrock Depth/Elevation

Boring No.	Surface Elevation (feet)	Approximate Depth to Bedrock (feet)	Approximate Elevation of Bedrock (feet)	Top of Bedrock Description
AKA-4	935.01	15.5	919.5	Volcanics
HLA (1988) 1	~920	8	912	Grey Andesite
MW25-95-26	936.17	~15	921.2	Volcanic Rock

In general, the bedrock materials at the site are directly overlain by soils comprised of bedrock materials that have weathered in place (residual soil). The transition from residual soil to bedrock can be gradual, therefore the interpreted depths of bedrock presented in this report and the attachments should be considered approximate.

Soil Conditions

Within the study area, the soils that overlie rock vary in thickness and include fill materials placed in association with the development of Building 25 and (possibly) Building 26. Approximate fill thicknesses and the corresponding elevation at the top of natural soil are indicated in the following table.

Interpreted Top of Natural Soil Depth/Elevation

Boring No.	Surface Elevation (feet)	Approximate Depth of Fill (feet)	Approximate Elevation – Top of Natural Soil (feet)	Top of Natural Soil Description
AKA-4	935.01	10.5	924.5	Clay (dark brown)
HLA (1988) 1	~920	4	916	Clay (red brown)
MW25-95-26	936.17	~4	~932.2	Clay (red brown)

We performed Atterberg Limits determinations on two samples of soil from Boring AKA-4 to evaluate the plasticity and expansion potential of the onsite soil materials. The results of our laboratory Atterberg Limits determinations are presented on the attached boring logs and are summarized in the following table.

Atterberg Limits Test Results

Boring No.	Approximate Depth of Sample (feet)	Liquid Limit	Plasticity Index	Soil Classification	Layer Description
AKA-4	6 – 6.5	39	22	Lean Clay (CL)	Fill
AKA-4	11 – 11.5	46	25	Lean Clay (CL)	Colluvium

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.

Groundwater Conditions

Boring AKA-4 did not encounter groundwater and the boring logs presented in Appendices A through C of our April 2, 2010 geotechnical investigation report for the GPL provide only limited data on groundwater conditions in the general area of the GPL site. However, previous data and interpretations developed by LBNL's EH&S Division generally indicate that groundwater can be present beneath the site above the contact between the Moraga Formation (which can be highly permeable) and the underlying Orinda Formation (which is typically less permeable).

Pavement Conditions

Boring AKA-4 initially penetrated about 18 inches of asphalt concrete pavement over about 6 inches of aggregate base. The abnormally-thick asphalt concrete layer appeared to have been built up over multiple episodes of paving and re-paving, which we have seen elsewhere at LBNL in areas that have settled and/or crept laterally downslope. The general condition of the pavement within the study area can also be seen on the site photographs presented on Figure 4.

DISCUSSION AND CONCLUSIONS

Past Site Performance

The thickened asphalt concrete pavement observed at the site and encountered in Boring AKA-4 generally indicates that portions of the existing roadway have settled since they were constructed (in the 1940s). Whether this apparent vertical settlement has been accompanied by horizontal movement (i.e., in a downslope direction) could not be determined based on site observations due to the repaving that has been performed. Possible geotechnical "causes" of the pavement settlement are discussed below.

In our opinion, the general pattern of settlement that we observed appears generally consistent with surficial soil creep; a phenomenon whereby near-surface clayey soils gradually move downslope by gravity as a result of repeated cycles of wetting and drying. On Bay Area hillslopes, creep-type movements commonly affect soils that are within about 5 feet of the ground surface. It would therefore not be unusual to see indications of creep-type movement extending 10 or more feet back, horizontally, from the crest of a 2:1 (horizontal to vertical) slope.

It is also possible that deeper movements may have affected the existing roadway following its construction. Boring AKA-4 was drilled in the outboard portion of the roadway and encountered fill extending to a depth of 10.5 feet. The sampler blow counts obtained in the fill were not high (SPT blow counts of 7 and 13 were obtained at depths of about 4 and 7 feet, respectively) and the fill was underlain by natural dark gray to very dark brown clay that Atterberg Limits test results indicate are moderately expansive. These conditions suggest two other post-construction movement scenarios: (1) vertical compression (settlement) within the almost 10-foot-thick fill layer, which appears under-compacted; and/or (2) downslope movement of the heavy fill body, which appears not to have keyed and benched into competent underlying material (i.e., bedrock). Either or both scenarios could be the primary cause or a contributor to the observed settlement.

Roadway Widening Considerations

In our opinion, a significant non-geotechnical consideration for the project will involve evaluations of project cost versus long-term performance. Because the extent of the widening and the height of the planned retaining wall are somewhat minimal, a "reasonable" low-cost approach might be acceptable to LBNL even if such an approach would require future maintenance and/or repairs. Alternatively, a fully-engineered design could be developed that would optimize long-term performance of the widened pavement section and the adjacent section of roadway, but at a higher cost. The selection of an appropriate design may also be influenced by: (1) planned depths of excavation associated with the planned adjacent GPL building; (2) the presence of existing utilities or other adjacent improvements that need to be maintained/protected during construction; and (3) LBNL preferences involving aesthetics or other non-technical aspects of the project.

In this report, we present several roadway widening concepts for consideration. In general, the concepts are intended to represent a range of alternatives and <u>not</u> an exhaustive summary of possible project approaches. Supplemental geotechnical recommendations are provided later in this report for those aspects of the conceptual approaches not previously addressed in our April 2, 2010 geotechnical investigation report for the GPL. In general, the recommendations presented can be considered suitable for preliminary design; however, we strongly suggest that our firm be consulted as designs are developed so that we can: (1) check and verify the applicability of our recommendations to the design(s) being considered; and (2) provide additional geotechnical recommendations in the event that they are needed.

Roadway Widening Concepts

Four basic roadway widening concepts are introduced and discussed below:

> ***Mechanically Stabilized Earth (MSE) Embankment*** – In this concept, the widened section of roadway would be supported on georeinforced engineered fill that is appropriately keyed and benched into bedrock. Since the area to be widened is situated at the top of an existing slope inclined at 2:1; the face of the new embankment will have to be significantly steeper in order to intersect ("catch") the slope within a reasonable distance. An MSE fill embankment can be constructed as steep as about 1:1 without facing; with modular facing, the embankment face can be vertical. Georeinforcing will need to extend a significant distance back from the embankment face; the designer should therefore anticipate that this will in all likelihood require that an MSE embankment extend into the area currently occupied by the existing roadway.

> ***Retaining Wall*** – In this concept, the widened section of roadway would be supported upon engineered fill that is retained by a structural wall supported on/in bedrock. The wall would need to be designed to resist lateral loads from the retained soil and vehicle surcharges; with resistance to lateral

loads being provided by passive pressures acting upon the below-grade faces of foundation elements. The designer should anticipate that soil adjacent to the face of the wall foundations may continue to move downslope under gravity; therefore passive resistance should only be applied within bedrock. In this report, we provide supplemental recommendations for the design of a cantilever soldier pile and lagging wall; although it is possible that tiebacks would be needed to restrain the upper portion of the wall depending upon the wall height and the magnitude of the applied surcharge loads.

Structural Support – In this concept, the widened section of roadway would be structurally supported on one or more rows of vertical drilled piers that extend into bedrock. The piers would need to be designed to support the weight of the supported roadway and vehicle surcharges and would also need to be designed to resist unbalanced lateral loads associated with the movement of soil downslope of the piers (similar to the retaining wall case). The designer should anticipate that a roadway supported on drilled piers embedded in bedrock will likely experience negligible post-construction settlement; any continued settlement occurring in the adjacent at-grade section of roadway (not supported on piers) will therefore be manifested as differential settlement.

"Minimal" Project – In this concept, the widened section of roadway would be supported on engineered fill that is retained by a wall <u>not</u> supported on/in bedrock. In taking such an approach, LBNL would need to recognize and acknowledge that: (1) settlement and downslope movement may continue to occur that would need to be addressed by ongoing maintenance activities (e.g. repaving); (2) the wall at the outboard edge of the widened pavement area may tilt and deflect; (3) continued and progressive movements could ultimately result in LBNL needing to modify, augment or replace the improvements constructed under a minimal project approach with something more robust; and (4) it is possible that the improvements constructed under a minimal project approach could "fail" as a result of non-typical circumstances or events (e.g. unusually heavy surcharges, sustained heavy rainfall or earthquake loads). The designer should anticipate specialized materials or approaches might be used to increase the likelihood that a "minimal" project will perform acceptably over the long term; in this report, we suggest that lightweight geotechnical materials could be used to minimize future settlement and the lateral loads exerted on the retaining wall bounding the outboard edge of the widened roadway.

<u>SUPPLEMENTAL RECOMMENDATIONS</u>

This section presents supplemental geotechnical recommendations for the roadway widening concepts discussed in the previous section. As previously noted, a variety of factors will need to be considered and evaluated prior to developing a final design. We should be consulted as designs are being developed in order to augment or revise our geotechnical recommendations, if necessary, to suit the project design(s). The recommendations presented in the sections that follow supplement the recommendations presented in our April 2, 2010 geotechnical investigation report for the GPL.

MSE Embankment Recommendations

Prior to clearing, all active subsurface utilities in and immediately surrounding areas to be graded should be located, marked and protected or relocated. Any trees within the limits of grading should be cleared and tree stumps and rootballs should be grubbed to the extent sufficient to remove organic-laden materials. The site should be cleared of pavements, aggregate base, irrigation lines, and other near-surface improvements. Soils containing vegetation and organic matter should be stripped. Stripped and grubbed 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. Pavements, aggregate base, broken concrete, and other cleared improvements should be removed from the site or stockpiled for later use, subject to the approval of our firm and LBNL.

Excavation cuts should be laid back or shored in accordance with Cal-OSHA requirements. Temporary shoring will be required where safe cutback slopes are not possible, or where the protection of adjacent utilities, structures or other improvements is necessary. The design, installation and maintenance of all shoring and excavation slopes and the protection of workers, utilities and adjacent improvements during construction are responsibilities of the contractor.

A keyway should be excavated into in-place bedrock at the toe of the georeinforced fill embankment. The keyway should be at least 12 feet wide and should extend at least 3 feet into in-place bedrock at its downslope edge. Keyways in MSE areas should be wide enough to accommodate the length of any required georeinforcing elements, plus at least 5 feet to allow for the installation of subdrainage at the excavation back cut. We recommend that we observe the keyway excavation to check that suitable in-place bedrock materials are exposed.

Based on the available data, we judge that excavation can likely be accomplished using conventional heavy earthmoving equipment. However, zones of relatively hard rock could be encountered. The contractor should be prepared to utilize suitable hard rock excavation techniques such as jackhammering or hoe-ramming, if necessary. Excavated soil and rock that do not meet the requirements for fill materials should be processed to conform to project specifications or removed from the site.

Subdrainage should be installed along the back cut of the keyway and up the back cut of the excavation. Subdrainage should consist of 4-inch, rigid, perforated pipes, surrounded by ¾-inch, open-graded drainrock encapsulated in non-woven polyester geotextile filter fabric such as Mirafi 140N or 140NL, or a 4-ounce equivalent. Caltrans Class 2 Permeable Material can be used in lieu of drainrock, in which case the filter fabric can be omitted. Subdrain pipes should be installed along the back cut of the new fill at no greater than 5-foot vertical intervals as filling proceeds. The perforated drainpipe should be sloped to drain with a minimum positive gradient of 2 percent. We recommend the use of SDR 35 or Schedule 40 PVC for the subdrain system. The subdrain pipe at each level should be connected to a closed pipe (non-perforated) that drains, by gravity, to a suitable discharge facility. The "high" end and all 90-degree bends of the subdrain pipe should be connected to a riser that extends to the surface and acts as a cleanout.

MSE construction involves the placement of tensile elements (georeinforcing) between layers of compacted fill and it is common for MSE fill embankments to be designed by the georeinforcing manufacturer. In general, we do not take exception to this approach provided that: (1) the Civil Engineer on the project design team review and take responsibility for the manufacturer's proposed design; and (2) the design substantially conform with the intent of our geotechnical recommendations. Most guidelines for MSE construction specify that granular non-expansive materials be used as fill as the long-term creep behavior of georeinforcing placed within clayey backfill materials is not well understood. Because soil materials at the site consist predominantly of clays that are moderately expansive, we anticipate that the specified granular non-expansive materials will likely need to be obtained from an offsite source. If onsite materials are to be considered for this purpose, additional consultation between the geogrid manufacturer, the project Civil Engineer and our firm would be needed prior to developing suitable recommendations.

All proposed fill materials should be approved by AKA prior to their use; import material should be evaluated by our firm prior to its importation to the site. Materials previously used for construction (aggregate base, crushed concrete or crushed asphalt) may also be suitable for re-use as fill from a geotechnical standpoint;

however, the use of such materials as fill would need to be approved by the design team and LBNL. Georeinforcing elements should consist of Tensar Structural Geogrid UX 1500HS, or an approved alternative. Wall facing material, if used, should be selected by the project Civil Engineer, subject to the review and approval of LBNL.

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. Fill should be placed in layers such that the surface of each layer is nearly level and the fill should be benched into bedrock as filling proceeds. Backfill within the georeinforceing zone shall be spread in uniform layers not exceeding 8 inches in loose thickness and compacted to at least 95 percent relative compaction as determined by ASTM D-1557. Construction equipment shall operate on top of the fill and shall not be permitted to operate directly on top of exposed georeinforcing materials. Compaction at the faces of slopes should be obtained by "overbuilding" followed by trimming the completed slope back to design grades. We recommend that AKA observe excavation, subdrainage installation, keying and fill placement to confirm that subsurface conditions are as anticipated and that the MSE embankment is constructed in accordance with the intent of our geotechnical recommendations.

Soldier Pile Retaining Wall Recommendations

We envision that the soldier pile wall would consist of a row of drilled holes in which steel beams will be inserted and concreted in place. For the exposed above-grade portion of the wall, reinforced concrete or treated wood lagging would be installed between the steel beams to construct the wall face. This section provides recommendations for a cantilever retaining wall design that is founded in bedrock and neglects any contribution to passive resistance from fill and/or colluvium downslope of the wall (i.e., all soil above bedrock). If the designer determines that tiebacks are needed to restrain the top of the wall, we should be consulted to provide additional recommendations.

Holes for cantilever soldier piles should extend at least 8 feet into bedrock. An AKA representative should observe during pier drilling operations to determine the depth at which bedrock is encountered; where bedrock is deeper than estimated, pier holes will need to be deepened to provide the required 8 feet of embedment into bedrock. Lagging should be provided on the above-grade portion of the retaining wall structure, and should extend at least 2 feet below the lowest adjacent surface grade. If lagging is not carried down to bedrock, individual piers should be spaced such that the clear distance between the piers does not exceed 5 feet. Where lagging is not used, soil pressure should be assumed to be transferred to the piers through soil bridging. Pier spacings wider than 5 feet, edge to edge, can be used where lagging extends down to bedrock.

Lateral earth pressures on cantilever retaining walls that are free to rotate at their tops can be evaluated using an equivalent fluid weight of 45 pounds per cubic foot (pcf). This triangular lateral earth pressure distribution is applicable for a level backslope condition and should be applied to the portion of the retaining structure above bedrock. For design purposes, the elevation of the top of bedrock can be estimated using the interpreted bedrock elevation contours shown on Figure 5. The increases in lateral wall pressures caused by vehicles operating behind the top of the wall can be evaluated by adding an additional uniform 100 psf horizontal pressure over the upper 10 feet of the wall height. Temporary surcharge loads near the top of the wall can be evaluated using a uniform lateral pressure equal to one-half of the vertical surcharge load, applied over the upper 10 feet of the wall. If unusually heavy and/or long-term surcharge loads are anticipated, we should be consulted to provide supplemental recommendations, as appropriate.

To evaluate increases in lateral loads caused by earthquake shaking, a uniform lateral seismic load increase of 18H (in pounds per square foot [psf] for H in feet) should be applied over the full effective height of the wall.

The effective height (H) of the wall should be taken as the vertical distance from the elevation of the widened roadway down to the top of bedrock and can be estimated using the interpreted bedrock elevation contours shown on Figure 5. The resulting seismic lateral earth pressure distribution is considered unfactored; normal factors of safety for short-term dynamic loading should be used when utilizing the load distribution for wall design. Where seismic lateral pressures are to be considered, they should be added to the earth pressure distribution used for static conditions, but factored appropriately for the load condition. In our judgment, it is reasonable to consider vehicle surcharge loads and increases in lateral loads caused by earthquake shaking as independent load cases that do not need to be applied simultaneously.

Lateral earth pressure recommendations presented in this section are based on the assumption that the wall will be fully drained to prevent the build-up of hydrostatic pressures behind the wall. This is commonly accomplished by providing small gaps between the lagging or through weepholes, although a wall backdrain system involving gravel and plastic pipe could also be used. Where wall backfill is needed, we recommend that Caltrans Class 2 Permeable Material generally be used for backfill within the one-foot zone directly behind the back face of the wall. Spacers should be provided between individual horizontal lagging elements to allow water to drain through the wall. We recommend that lagging be spaced no more than about ¼ to ½ inch apart, to reduce loss of permeable material from behind the lagged portion of the wall. If solid panels are used in lieu of lagging or if a continuous wall is to be cast, we should be consulted to provide additional recommendations for appropriate backdrainage.

Resistance to lateral loads can be provided by passive pressures acting against the downslope sides of below-grade structural elements. Passive resistance can be applied over two (horizontal) pier diameters starting at the top of bedrock. Passive resistance in bedrock can be evaluated using the truncated triangular distribution described below:

- Starting at the top of bedrock, a passive resistance value of 1,000 psf can be used; and
- Passive resistance in bedrock can be assumed to increase at a rate of 350 psf per foot of depth.

The above passive resistance values include a factor of safety of at least 2. Soldier piles should be designed to extend at least 8 feet into bedrock, regardless of lateral load. For design purposes, the elevation of the top of bedrock can be estimated using the interpreted bedrock elevation contours shown on Figure 5.

Holes for soldier piles should be drilled straight and plumb (within 1 percent of vertical) and should be cleaned of loose soil and rock fragments. 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 soldier piles should be concreted on the day they are drilled. Following placement of the reinforcing steel or H-section, 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, any casing used to stabilize the hole should be withdrawn and the casing should be maintained not more than 5 feet or less than 1 foot below the surface of the concrete as it is withdrawn.

Soldier piles should be installed by a qualified drilling contractor. AKA should observe during drilling to confirm that subsurface conditions are as anticipated and observe the various geotechnical aspects of construction to check conformance with the intent of our recommendations. Protection of retaining walls throughout the construction phase is the responsibility of the contractor. We recommend that particular care be exercised directly upslope of retaining walls with respect to: (1) construction vehicles (including cranes); (2) material stockpiles; and (3) any necessary soil compaction operations.

Structural Support Recommendations

We envision that structural support would involve the installation of one or more rows of drilled piers in the area of the widened roadway. This section provides recommendations for drilled piers that are founded in bedrock and considers unbalanced lateral load effects associated with the continued movement of fill and/or colluvium downslope of the outboard row of piers (similar to the retaining wall concept). If the designer determines that tiebacks are needed to restrain the top of the piers, we should be consulted to provide additional recommendations.

Foundation piers should be spaced no closer than three pier diameters, center-to-center. Piers at closer spacings may have a reduced compressive capacity due to group effects and would need to be evaluated on an individual basis. Drilled pier groups should be structurally tied together at their tops by grade beams or a thickened structural concrete slab. Grade beams and structurally supported slabs should be underlain by at least 18 inches of non-expansive fill to mitigate potential expansive soil uplift effects. Alternatively, a 4-inch minimum vertical space (void) could be provided between the ground surface and overlying structural elements.

The axial capacity of drilled piers can be evaluated using an allowable skin friction value of 500 psf in soil and 1,200 psf in bedrock. These skin friction values can be increased by one third for total compressive loads, including wind and seismic, but should not be increased for uplift loads. For the outboard row of piers, skin friction should be applied over only one-half of the pier diameter in soil to account for the possibility that the adjacent soil may move downslope. In other cases, skin friction can be applied over the full pier diameter.

Drilled piers should extend at least 8 feet into bedrock, regardless of load. For design purposes, the elevation of bedrock can be estimated using the bedrock contours shown on Figure 5. We recommend that any contribution to axial capacity from end bearing in bedrock be ignored due to difficulties associated with obtaining and/or assuring a clean bearing surface at the bottom of the pier holes.

Drilled piers should be designed to resist unbalanced lateral pressures caused by the continued movement of soil downslope of the drilled piers. Unbalanced lateral earth pressures acting on cantilever drilled piers that are free to rotate at their tops can be evaluated using the equivalent fluid weights presented in the following table.

Unbalanced Lateral Earth Pressures

Slope Behind Wall	Equivalent Fluid Weight (pcf)	Increase over Level Backslope
Level	45	1.00
3:1	50	1.11
2:1	60	1.33

The above lateral earth pressures can be assumed to act above the surface of bedrock on the upslope side of the drilled piers.

Resistance to lateral loads can be provided by passive pressures acting against the downslope sides of drilled piers. Passive resistance can be applied over two (horizontal) pier diameters starting at the top of bedrock. Passive resistance in bedrock can be evaluated using the truncated triangular distribution described below:

- Starting at the top of bedrock, a passive resistance value of 1,000 psf can be used; and
- Passive resistance in bedrock can be assumed to increase at a rate of 350 psf per foot of depth.

The above passive resistance values include a factor of safety of at least 2. All piers should be designed to extend at least 8 feet into bedrock, regardless of lateral load. For design purposes, the elevation of the top of bedrock can be estimated using the interpreted bedrock elevation contours shown on Figure 5.

Holes for drilled piers should be drilled straight and plumb (within 1 percent of vertical) and should be cleaned of loose soil and rock fragments. 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 drilled piers should be concreted on the day they are drilled. Following placement of the reinforcing steel or H-section, 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, any casing used to stabilize the hole should be withdrawn and the casing should be maintained not more than 5 feet or less than 1 foot below the surface of the concrete as it is withdrawn.

Drilled piers should be installed by a qualified drilling contractor. AKA should observe during drilling to confirm that subsurface conditions are as anticipated and observe the various geotechnical aspects of construction to check conformance with the intent of our recommendations.

"Minimal" Project Recommendations

Because the scope of the roadway widening is limited, LBNL may wish to consider a "minimal" alternative with a low initial cost. In our opinion, such a choice could reasonably be made provided that LBNL understands and acknowledges the various tradeoffs associated with cost, long-term performance and risk. We suggest that these decisions ultimately be made in consultations between LBNL and the project design team. AKA would be pleased to participate in these subsequent design discussions, at LBNL's discretion.

A "minimal" project design might involve the following geotechnical elements: (1) constructing a low retaining wall at the outboard edge of the roadway widening area; (2) placing new fill behind the retaining wall to raise grades; (3) paving on top of the new fill. If such an approach is to be taken, we recommend that the designer consider means by which the chances of reasonable or "acceptable" long-term performance might be improved. In our opinion, rigid lightweight fill materials could offer advantages; two such materials are as follows:

Expanded Polystyrene (EPS) – EPS is an ultra light (1 to 2 pcf) fill material that is delivered as rigid blocks (additional information is available on the Geofoam website: http://www.geofoam.com/).

Cellular Concrete – Cellular concrete is a light (about 20 to 40 pcf) flowable fill material that hardens into a rigid mass (additional information is available on the Geofill website: http://www.geofill.com/).

Either type of material could be used to replace near-surface soil in the area of the roadway widening (thereby reducing vertical loads) and to raise grades.

LIMITATIONS AND CLOSURE

This report has been prepared for the exclusive use of LBNL and their consultants for specific application to the GPL roadway widening project in accordance with generally accepted soil and foundation engineering practices. No other warranty, expressed or implied, is made.

At the time that this report was written, the location of the roadway widening project was shown on the plans; however, no details were available pertaining to the design of this aspect of the project. In this report, we present design concepts that we developed based on our current understanding of the site conditions and project requirements. Future roadway widening concepts developed by the design team may vary appreciably from those presented in this report. Consequently, the conclusions and recommendations contained in this report should only be considered valid to the extent that the final design conforms to the descriptions presented in this report. In our judgment, it is essential that we be consulted as final designs are being developed in order to: (1) check conformance with the intent of our geotechnical recommendations; and (2) identify any aspects of the design that would require that the conclusions of this report be modified (in writing).

This report poses the possibility of "minimal" project approach, which would require that LBNL fully understand and accept the risk that the project may not perform acceptably over the long term. Our firm would be pleased to provide supplemental consultation to LBNL and the design team under these conditions; however, AKA cannot be held responsible in the event that a "minimal" project does not provide satisfactory post-construction performance.

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.

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.

Dona K Mann

Dona Mann, C.E.
Senior Engineer

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Associate Engineer

REGISTERED PROFESSIONAL ENGINEER
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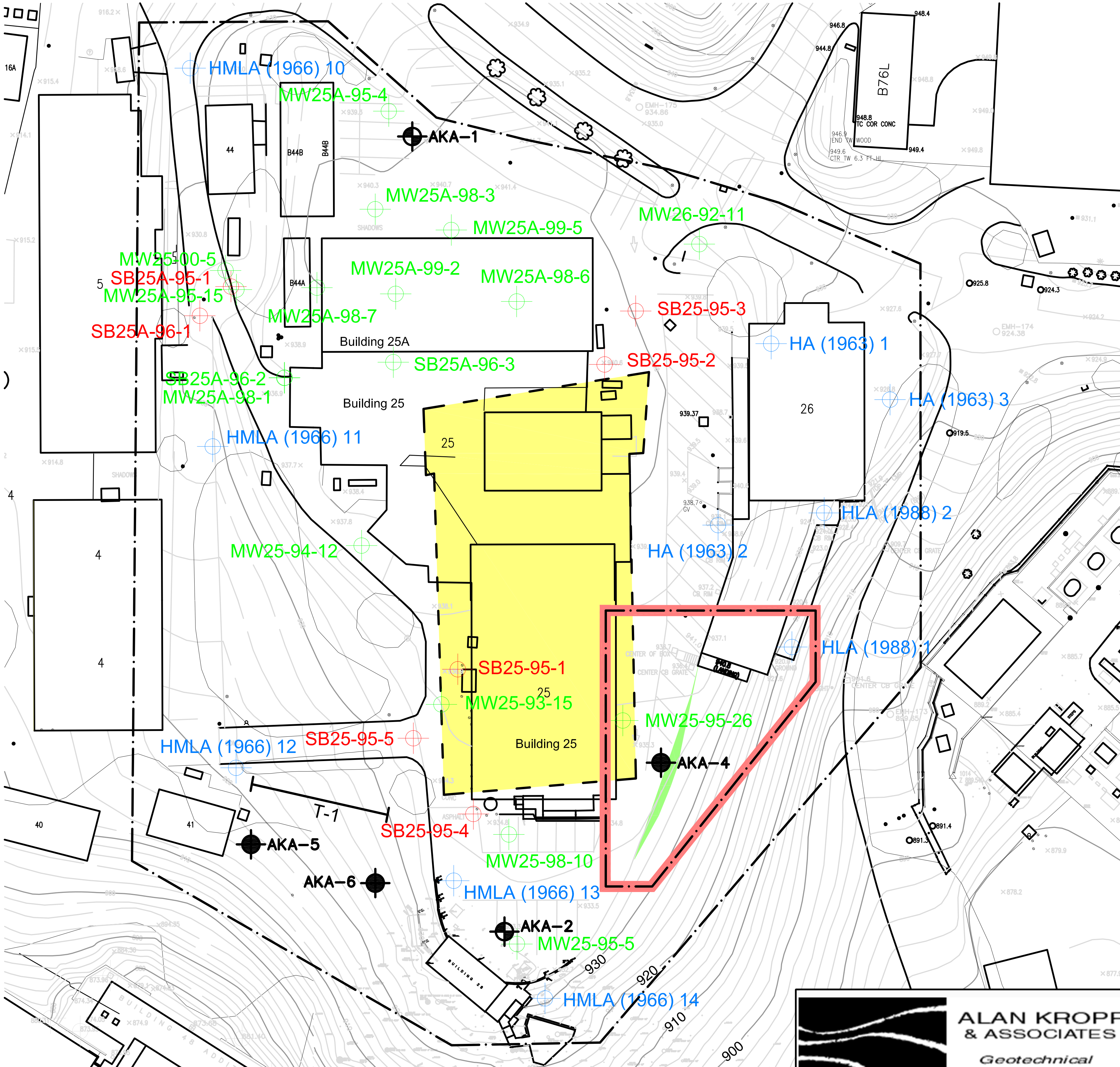
Attachments: Figure 1 Supplemental Investigation Site Plan
Figure 2 Key to Exploratory Boring Logs
Figure 3 Physical Properties Criteria for Rock Descriptions
Figure 4 Site Photographs
Figure 5 Bedrock Surface Contour Map
Log of Boring AKA-4

2335-12C B25 GPL Supplement Pavement Widening FINAL



Base: Topographic base received from LBNL May 2009.

LEGEND

Symbol	Description
AKA-5	Approximate location of supplemental AKA boring drilled in April 2010
	Approximate area of roadway widening
	Approximate limit of study area, this supplement
AKA-1	Approximate location of AKA/WLA boring (AKA-2009) Appendix A of Geotechnical Report (AKA-3 located offsite)
HLA (1988) 1	Approximate location of previous geotechnical/geologic boring by others (HA 1963; HMLA 1966; HLA 1988) Appendix B of Geotechnical Report
HLA (1988) 1	Approximate location of previous environmental boring/well by LBNL Appendix C of Geotechnical Report
HLA (1988) 1	Approximate location of previous environmental boring/well by LBNL for which logs are not available
T-1	Approximate location of exploratory test pit or trench
	Approximate footprint of General Purpose Lab (GPL)
	Approximate limits of GPL study area (borings and trenches outside study area not shown)

50 0 50 100
APPROXIMATE SCALE FEET

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SUPPLEMENTAL INVESTIGATION SITE PLAN

LBNL GENERAL PURPOSE LAB ROADWAY WIDENING
Berkeley, California

PROJECT NO.	DATE	FIGURE
2335-12C	**May 2010**	**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	$Cu \geq 4$ AND $1 \leq Cc \leq 3$ 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	$Cu \geq 6$ AND $1 \leq Cc \leq 3$	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	$\frac{\text{LIQUID LIMIT - OVEN DRIED}}{\text{LIQUID LIMIT - NOT DRIED}} < 0.75$	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	$\frac{\text{LIQUID LIMIT - OVEN DRIED}}{\text{LIQUID LIMIT - NOT DRIED}} < 0.75$	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

INDEX TESTS

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

STRENGTH TESTS

- 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)

MISCELLANEOUS

- 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 GENERAL PURPOSE LAB ROADWAY WIDENING
Berkeley, California

PROJECT NO.	DATE	FIGURE 2
2335-12C	**May 2010**	

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 GENERAL PURPOSE LAB ROADWAY WIDENING
Berkeley, California

PROJECT NO.	DATE	FIGURE **3**
2335-12C	**May 2010**	

Looking North, Building 25 on left

Looking North, Building 25 on left

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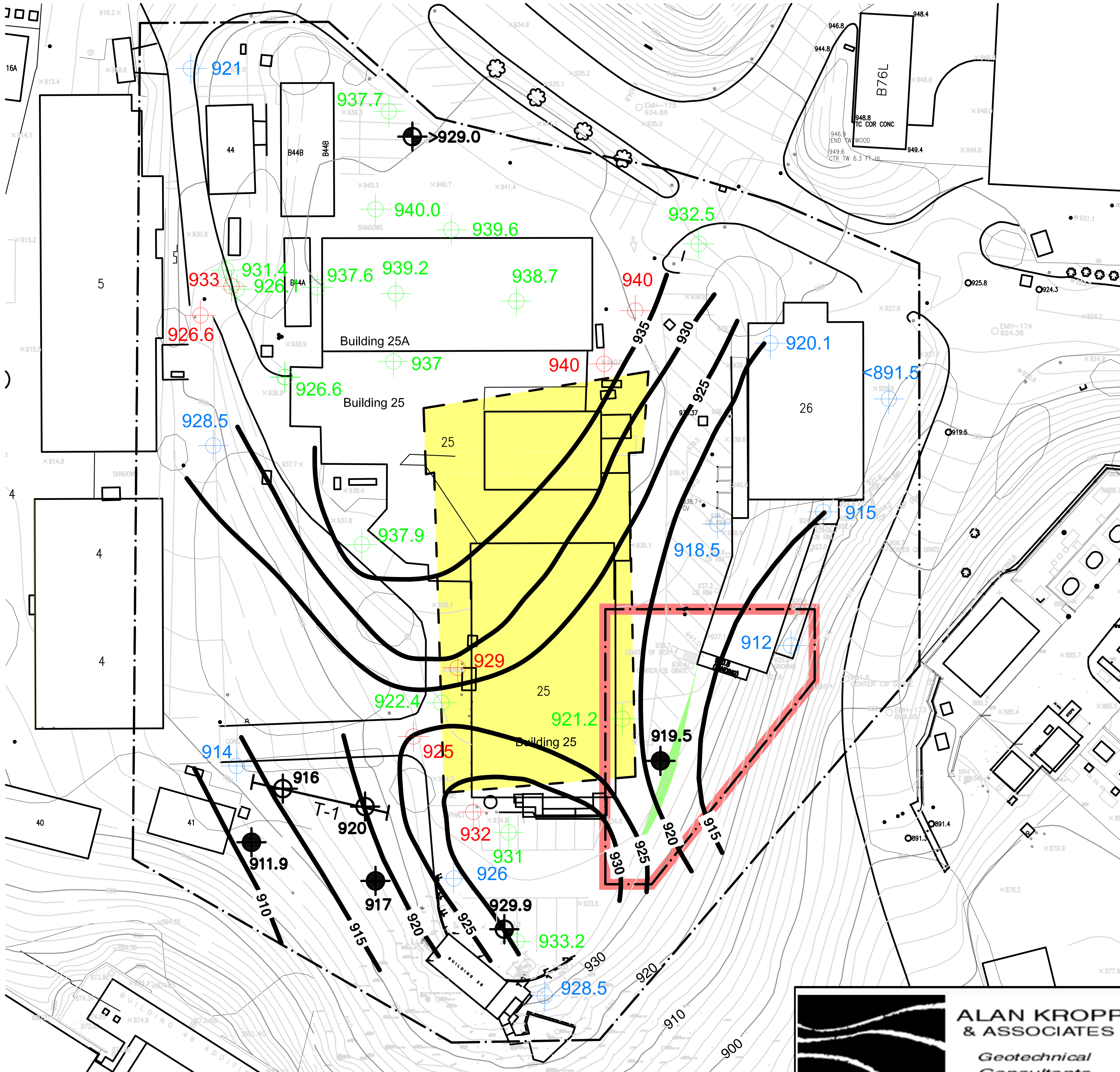
SITE PHOTOGRAPHS

LBNL GENERAL PURPOSE LAB ROADWAY WIDENING
Berkeley, California

PROJECT NO.	DATE	FIGURE 4
2335-12C	**May 2010**	



Base: Topographic base received from LBNL May 2009.

LEGEND

- Approximate area of roadway widening
- Approximate limit of study area, this supplement
- 936 Bedrock surface elevation reported on log of boring (in feet)
- 935 Interpreted bedrock surface contours (elevation in feet)
- Approximate footprint of General Purpose Lab (GPL)
- Approximate limits of GPL study area (borings and trenches outside study area not shown)

NOTE: Bedrock surface elevation contours are interpreted based on data from existing borings and should be considered approximate.

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BEDROCK SURFACE CONTOUR MAP

LBNL GENERAL PURPOSE LAB ROADWAY WIDENING
Berkeley, California

PROJECT NO.	DATE	FIGURE 5
2335-12C	**May 2010**	

DRILL RIG: B-24, Solid Flight Auger	**SURFACE ELEVATION:** 935 feet (see notes)	**LOGGED BY:** DI
DEPTH TO GROUNDWATER: (see notes)	**BORING DIAMETER:** 4 inches	**DATE DRILLED:** 4/12/10

DESCRIPTION AND REMARKS	COLOR	CONSISTENCY	SOIL TYPE	DEPTH (ft)	SAMPLER TYPE	SAMPLER BLOW COUNTS	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OTHER TESTS
~ 18" Asphaltic Concrete - (Several Generations)			AC	1					
~ 6" Aggregate Base Rock			AB	2					
CLAY, Lean - with silt and sand (fine grained), trace gravel (fine grained, rounded), dry to moist (FILL)	Dark Reddish Brown with some Grayish Brown	Stiff to Very Stiff	CL	3			14.5	103	PP = 2.75 tsf PP = 3.25 tsf
						[17]	12		
				4					
		Firm to Stiff		5		7			
				6			17.6		PP = 3.75 tsf LL = 39 PI = 22 -200 = 70%
	Mottled Grayish Brown and Brown with some Yellowish Brown	Stiff to Very Stiff				[19]	19.7	89	
				7					
		Stiff		8		13			
				9					
FILL				10					
CLAY, Lean - with silt and sand (fine to coarse grained)	Dark Gray to Very Dark Brown	Stiff to Very Stiff	CL	11					PP = 2.75 tsf LL = 46 PI = 25 -200 = 54%
						[21]			
				12					
				13					
				14					
				15					
VOLCANICS - deeply to moderately weathered, weak to moderately strong, low hardness, with angular clasts of weathered andesite	Reddish Brown to Grayish Brown		BED ROCK	16		[50/4"]	20		
				17					
				18					
				19					

(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)									
VOLCANICS - deeply to moderately weathered, weak to moderately strong, low hardness, with angular clasts of weathered andesite	Gray to Grayish Brown		BED ROCK						
				21		50/6"			
				22					
				23					
				24					
				25					
				26					
	Olive Gray with some Dark Brown					44	10.2		
				27					
				28					
				29					
				30					
						36			

Bottom of boring at 30.5 feet.

NOTES:

1. No groundwater was encountered at the time of drilling and the boring was backfilled immediately after drilling. (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. Boring location elevations were surveyed 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".