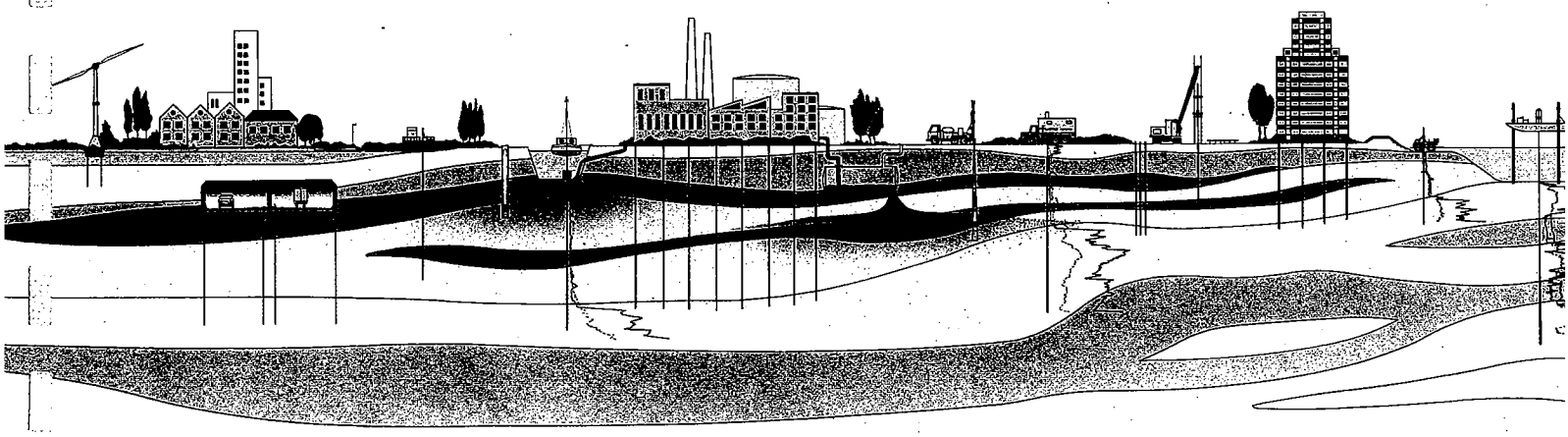
#385

FUGRO WEST, INC.

GEOTECHNICAL INVESTIGATION PROPOSED BUILDING 50X LAWRENCE BERKELEY NATIONAL LABORATORY BERKELEY, CALIFORNIA

Prepared for:
Lawrence Berkeley National Laboratory

August 5 2002



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August 5, 2002
Project No. 658.052

Lawrence Berkeley National Laboratory
One Cyclotron Road, Building 90G
Berkeley, California 94720

Attention: Mr. Steve Blair

Subject: Final Report, Geotechnical Investigation, Proposed Building 50X, Lawrence Berkeley National Laboratory, Berkeley, California, dated August 5, 2002

Dear Mr. Blair:

Fugro is pleased to present this geotechnical report for the proposed Building 50X at the Lawrence Berkeley National Laboratory.

We thank you for providing us the opportunity to provide services to Lawrence Berkeley National Laboratory. Please call if you have any questions regarding the information presented in this report.

REGISTERED PROFESSIONAL ENGINEER
STEVEN M. WU
No. C 058317
Expires 6/30/06
CIVIL
STATE OF CALIFORNIA

Sincerely,

FUGRO WEST, INC.

Steven M. Wu, PE
Senior Engineer

Wayne D. Magnusen, PE
Associate Engineer

SMW:WDM:ae

Copies Submitted: (6) Mr. Steve Blair, Lawrence Berkeley National Laboratory

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DISTRIBUTION

6 copies: Mr. Steve Blair
Lawrence Berkeley National Laboratory
Building 90G
One Cyclotron Road
Berkeley, California 94720

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1.0 INTRODUCTION

This report presents the results of a geotechnical investigation by Fugro West, Inc. for the proposed new Building 50X at the Lawrence Berkeley National Laboratory (LBNL). The project site is located on the east side of Cyclotron Road about 300 feet north of the Blackberry Canyon Entrance, as shown on the Vicinity Map and Site Plan, Plates 1 and 2, respectively.

We obtained information regarding the proposed project through discussions with Mr. Steve Blair of LBNL, and by reviewing preliminary site plans and concept drawings for the building provided by Mr. Blair.

1.1 PROJECT DESCRIPTION

The planned project will involve constructing a major new building in the northern portion of LBNL's Berkeley campus. The site is located west of Building 50F. The proposed building site is about 80 feet by 145 feet, in plan dimensions, and slopes upwards to the east, with about 40 feet of vertical relief across the short axis of the site.

As currently planned, a private developer will design and construct the new 5- to 6-story building and lease the facility back to LBNL. The building will be set back into the hillside with a ground floor entrance on Cyclotron Road and a rear 5th floor entrance on an upper access driveway. A partial ground floor is planned that extends about 50 feet back from the Cyclotron Road entrance. Based on the preliminary schematics provided, we anticipate that the building ground floor level will be at or below the level of Cyclotron Road. Floors 2 through 4 will occupy the entire 80- by 150-foot building footprint. The walls comprising the rear of the building will function as permanent retaining walls.

We understand that the current design of the new building is conceptual and preliminary and may change in the future. The actual design of the new building will be determined by the design team to be selected in the future.

1.2 PURPOSE AND SCOPE

The purpose of our geotechnical investigation was to evaluate the site conditions at the proposed location of the new building and to provide geotechnical design criteria and recommendations for the project. The scope of our geotechnical investigation, as outlined in our proposal dated March 22, 2002, consisted of drilling test borings, reviewing existing data, performing geologic mapping, performing a seismic refraction survey, performing engineering analyses, and summarizing our findings and conclusions in this geotechnical investigation report.

Fugro West, Inc., also conducted a concurrent Surface Fault Rupture Hazard Assessment (SFRHA) for the project. The results of our SFRHA study are summarized briefly within and are formally presented in a separate report.

2.0 FIELD INVESTIGATION AND TESTING

2.1 TEST BORINGS

On April 27, 2002, we investigated subsurface conditions at the site by drilling two exploratory test borings. The approximate locations of the borings are shown on the Site Plan, Plate 2. We also attempted to drill a third test boring along Cyclotron Road near the

Plate 2. We also attempted to drill a third test boring along Cyclotron Road near the southwestern corner of the site. However, after drilling approximately 2 feet, we encountered an unmarked underground utility. Upon encountering the underground utility, we notified LBNL site personnel and were directed to halt drilling activities in the area and not to attempt relocating our third boring.

The borings were drilled using a conventional, truck-mounted drill rig equipped with hollow stem augers. Our staff engineer supervised the drilling operations, continuously logged the soils and bedrock encountered, and collected samples of the subsurface materials for subsequent evaluation and laboratory testing. Soil samples and weathered bedrock were obtained using split barrel drive samplers equipped with brass liners. Logs of the borings and details regarding the field investigation are included in Appendix A.

2.2 LABORATORY EVALUATION AND TESTING

The soil and bedrock samples collected from the borings were reexamined in our laboratory to check field classifications. Laboratory tests including moisture content, dry density, and triaxial shear strength were performed on selected samples. The laboratory test results are presented on the boring logs in Appendix A. Details regarding the laboratory tests and the laboratory data sheets are presented in Appendix B.

2.3 GEOLOGIC MAPPING

On May 3, 2002, our certified engineering geologist visited the site to prepare a geologic map of the project area and plan the seismic refraction survey described in Section 2.4. The results of our geologic mapping are shown on Plate 2 and discussed in Section 3.3.

Between June 27, 2002 and July 12, 2002, three trenches were excavated during our concurrent SFRHA study. The results of this study were considered herein and are presented in a separate report.

2.4 SEISMIC REFRACTION SURVEY

On May 7, 2002, we conducted a seismic refraction survey at the site. Seismic refraction was used to determine the seismic velocities of the subsurface materials along two 110-foot traverses through the footprint of the new building. The approximate locations of the two traverses are shown on Plate 2.

Seismic velocities of the subsurface materials were determined by measuring the response of compression (pressure or "P") waves generated at the surface using an industrial seismic source. The P-waves propagate into the soils and refract at boundaries between subsurface materials having different properties. The refracted P-waves were measured by geophones spaced at approximately 10-foot intervals at the surface, and a Geometrics SmartSeis 12-channel seismograph collected the data. The recorded data was analyzed using software from Rimrock Geophysics.

Results from the survey were used to interpret geotechnical properties of the subsurface materials and generate an idealized soil profile. The seismic refraction survey supplements the

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subsurface information obtained from the exploratory borings. The velocity profiles and idealized soil sections from both traverses are presented in Appendix C.

3.0 SITE CONDITIONS

3.1 GEOLOGIC AND SEISMIC SETTING

3.1.1 Regional and Site Geology

The site is located on the west side of the Berkeley Hills within the Coast Ranges Geomorphic Province of Northern California. This province is characterized by a series of generally northwest-trending faults and folds. The Bay Area experienced uplift and faulting in several episodes during late Tertiary time (about 25 to 2 million years ago) producing series of northwest-trending valleys and mountain ranges, including the Berkeley Hills, the San Francisco Peninsula, and the intervening San Francisco Bay. The geology of the Berkeley Hills area is strongly influenced by the nearby Hayward Fault, which consists of a set of northwest-trending, right-lateral transcurrent faults along the base of the hills.

Review of published geologic maps[1] of the area shows the site to be underlain at relatively shallow depth by bedrock of the Panoche Formation. This formation generally consists of micaceous shale and sandstone. Bedrock consisting primarily of sandstone was encountered in both borings and was observed in road-cut exposures at the project site.

3.1.2 Seismic Setting

The San Francisco Bay region continues to be seismically active. The principal active faults in the Bay Area include the San Andreas, Hayward, Calaveras, Healdsburg-Rogers Creek, San Gregorio, and Green Valley-Concord faults. Earthquakes occurring along these faults are capable of generating strong groundshaking at the site. The site is about 320 feet (100 meters) northeast of the Hayward fault[2], and about 19 miles (30 kilometers) northeast of the San Andreas fault. Numerous other smaller faults have been mapped at LBNL, however the overall seismicity of the area is governed by the nearby Hayward fault. As shown on the Active Fault Near-Source Zones Map accompanying the 1997 Uniform Building Code (UBC), the site lies within 2 kilometers of a Type A Seismic Source (the Hayward fault).

3.2 SITE CONDITIONS

The site is located in an area of sloping terrain, on the east side of Cyclotron Road approximately 300 feet north of the Blackberry Canyon Entrance. In general, site grades in the vicinity of the proposed building slope upwards to the east at an average inclination of about 2:1 (horizontal to vertical). Locally, inclinations as steep as about 1:1 (horizontal to vertical) exist in cut slopes adjacent to Cyclotron Road.

Cyclotron Road bounds the downslope side of the site. The upslope side of the site is accessed by a driveway at the rear of Building 50F, a one-story addition constructed during the 1980's. Part of this earlier development included widening the upper access driveway in the downslope direction by means of a soldier pile and wood lagging retaining wall. Elevation

[1] Dibblee, T.W. Jr., 1980, *Preliminary Geologic Map of the Richmond Quadrangle, Alameda and Contra Costa Counties, California.* Department of the Interior, United States Geological Survey, Open File Report 80-1100.

contours shown on topographic maps provided by LBNL indicate that the site elevation ranges from about +630 feet to +710 feet (University of California Datum).

At the time of our investigation, the site was covered with weeds, grassy vegetation, and trees. An existing sanitary sewer and utility easement is located south of the proposed site.

3.3 GEOLOGIC SURFACE MAPPING

The results of our geologic reconnaissance and fault trenching indicate that the site is generally underlain by a relatively thin layer of colluvium overlying undivided, upper Cretaceous rocks. Near-surface colluvium that has recently experienced downslope movement under gravity (soil creep) is mapped on Plate 2. The colluvial deposits mapped on Plate 2 coincide with a swale present across the central portion of the site (extending from the upslope, eastern margin down to Cyclotron Road), and two other subtle swales to the north and south of the central swale. Fill materials used to construct the upper access roadway were found to extend onto the eastern margin of the site.

Bedrock was found exposed in the road cut for Cyclotron Road. The approximate upslope limit of the cut-slope face is shown on Plate 2. The observed rock exposures consist of greywacke sandstone interbedded with conglomerate. The rock is variable in strength and hardness, ranging from weak to moderately strong and soft to moderately hard. The sandstone is generally micaceous and fine-grained, with lithic fragments up to coarse-grained sand size common. Some beds contain round to subround, fine and coarse gravel-size clasts. Thickness of bedding in sandstone ranged from 2 to 5 inches in some locations, to massive sandstone at other locations. The conglomerate interbeds contain round to subround gravel and cobble-size clasts with a sandy matrix. Attitudes measured from the exposure suggest bedding striking between N45-55°W, and dipping 80°S to near vertical.

3.4 SUBSURFACE CONDITIONS

The results of our field investigation, seismic refraction survey, and fault trenching indicate that the site is underlain by a relatively thin layer of sandy to gravelly lean clay over weathered sandstone bedrock and more competent, sound bedrock at depth. Our interpreted cross-section through the site is shown on Plate 3. The subsurface materials encountered by our borings and seismic refraction survey is discussed below.

Boring B-1, drilled on the access driveway upslope of the site, encountered approximately 5 feet of fill and 9 feet of colluvium overlying bedrock. The near-surface soil is generally stiff and consists of gravelly lean clay. Beneath the colluvium, weathered sandstone bedrock was encountered. Drilling refusal in hard bedrock was encountered at approximately 33 feet.

Boring B-2, drilled on Cyclotron Road downslope of the site, encountered approximately 4 feet of fill and 5 feet of colluvium overlying bedrock. The near-surface soil is generally stiff and consists of sandy lean clay with gravel. Beneath the colluvium, weathered sandstone bedrock was encountered. Drilling refusal in hard bedrock was encountered at approximately 24 feet.

Our seismic refraction survey consisted of two traverses, Spread A and Spread B, through the center of the site. In general, the two traverses encountered three distinct velocity zones. The shallow velocity zone, with average compression wave velocities of 629 and 579 feet per second, for Spread A and B, respectively, is representative of shallow colluvium present across the surface of the site. Beneath the surface soils, a mid-depth velocity zone with compression wave velocities of 2232 and 2025 feet per second, for Spread A and Spread B, respectively, is representative of a zone of weathered bedrock. The weathered bedrock zone includes bedrock that has locally weathered to firm soil. Less weathered, more competent bedrock is represented by the deep velocity zone where compression wave velocities of 7840 and 5881 feet per second were determined for Spread A and Spread B, respectively.

3.4 GROUNDWATER

Free groundwater was not observed during drilling. However, in previous geotechnical investigations that we reviewed by LBNL for adjacent structures (i.e. Building 50B), groundwater was encountered as high as Elevation +686 feet, which is above the proposed floor level of Building 50X. The groundwater levels encountered in previous investigations may be reflective of seepage zones or perched groundwater that may be present beneath the site. Fluctuations in the groundwater level could occur due to change in seasons, variations in rainfall, and other factors

4.0 DISCUSSION AND CONCLUSIONS

We believe that the project is feasible from a geotechnical standpoint, provided that the conclusions and recommendations presented in this report are incorporated into the project design and specifications. The principal geotechnical considerations regarding the project are discussed in the following sections.

4.1 SEISMIC CONSIDERATIONS

4.1.1 Seismicity

The site is located in a seismically active region of California. Significant earthquakes in the Bay Area have been associated with movements along well-defined fault zones. Earthquakes occurring along the Hayward or any of a number of other Bay Area faults have the potential to produce strong groundshaking at the site. For this reason, the structure should be designed to resist lateral and uplift forces generated by earthquake shaking, in accordance with local design practice. Seismic design criteria for the 1997 UBC is presented in Section 5.6.

The site is located within a State of California Earthquake Fault Zone[2] for the Hayward fault. The Alquist-Priolo Act requires that a Surface Fault Rupture Hazard Assessment (SFRHA) be conducted by a State of California Certified Engineering Geologist for certain types of projects that fall within an Earthquake Fault Zones to evaluate the potential for surface fault rupture. Projects requiring a SFRHA under the Act include all significant new buildings that are intended for human occupancy. Fugro conducted a SFRHA for the Building 50X project as a separate scope of work. The results of the SFRHA study are presented in a separate report. Fugro's concurrent SFRHA study indicates that no active fault traces cross the site.

4.1.2 Liquefaction

Cyclic shear stresses induced by earthquake shaking can result in the development of excess pore water pressure in saturated soil masses. The tendency for the soil structure to densify as a result of shaking causes the intergranular stresses to be transferred to the pore water, resulting in increased pore water pressure. If the pore pressure builds up to a level equal to or greater than the overburden pressure, the soil can lose a large portion or all of its strength until the pore water pressures dissipate. This phenomenon is called liquefaction. Soils most susceptible to liquefaction are loose deposits of saturated sands and silts.

Cyclic shear stresses induced by earthquake shaking can also affect unsaturated soils. Because the soil is unsaturated, pore water pressures do not increase sufficiently to significantly lower the strength of the soil mass. However, the soil can densify resulting in differential settlement of the ground surface. Soils most susceptible to densification are cohesionless loose sands and gravels.

The soils encountered in our borings consist of stiff lean clay and sandstone rock that are not susceptible to liquefaction. Therefore, based on the available data, the potential for liquefaction or densification at the site is considered very low.

4.2 FOUNDATION SUPPORT AND SETTLEMENT

Based on the results of our geotechnical investigation, we judge that the proposed new Building 50X can be supported on conventional spread footing foundations bearing on weathered or hard bedrock. If necessary, net uplift resistance in excess of the building's dead weight can be resisted by grouted tiedown anchors that gain resistance through skin friction in bedrock. Tiedowns, for the purposes of this report, are high-capacity drilled elements ranging from about 7 to 8 inches in diameter. The tiedowns consist of a central reinforcing element (usually a high-strength steel bar or strand tendons) secured in place with cement grout. The grout can be injected under pressure and/or the tiedown can be "post-grouted" to increase capacity.

As an alternative to spread footing foundations, the new building can be supported on cast-in-drilled-hole (CIDH) pier foundations that gain support from skin friction in rock. Drilled CIDH foundations may be appropriate where foundations on or near the slope are required or where foundations need to resist significant uplift loads.

Due to the preliminary and conceptual nature of the design, the final building design may differ from the assumptions used in this report. Regardless of the foundation type, the foundations should extend to bedrock, as recommended in Section 5 below. Depending on the final design, this may require a combination of conventional spread footings, deepened spread footings, or drilled pier foundations to support the building. During final design, we should be consulted check the foundation design for conformance with the intent of our recommendations and provide additional recommendations, as warranted.

[2] *State of California Special Studies Zones, Richmond, Revised Official Map, Effective: January 1, 1982*, California Division of Mines and Geology, State of California, Department of Conservation, Richmond Quadrangle.

Settlement of properly-designed new foundations under moderate to heavy compressive loads should be small, less than about ½-inch. Differential settlements will likely be less than 50 percent of the total settlement.

4.3 CONSTRUCTION CONSIDERATIONS

We anticipate that soil and rock can be excavated by conventional earthmoving equipment, however, some localized harder areas may require jackhammering or hoe-ramming to excavate. Excavations that will be deeper than 5 feet that will be entered by workers should be shored or sloped for safety in accordance with the Occupational Safety and Health Administration (OSHA) standards. Temporary shoring will be required where safe cutback slopes are not possible or where the protection of existing structures or other site improvements is necessary.

Selection of an appropriate shoring system is influenced by a number of factors, including: (1) soil conditions adjacent to and below the excavation, (2) the maximum depth of excavation, (3) groundwater conditions and site dewatering considerations, (4) location and depth of adjacent foundations, (5) allowable deflection of the shoring system, (6) space and access requirements, and (7) cost considerations. Commonly used temporary shoring types for excavations in rock include soldier piles and lagging or shotcrete and tiebacks. Tiebacks may also be used to reduce the size of steel sections for soldier piles.

The performance of shoring systems is highly dependent on the construction methods and procedures employed. The design, installation and maintenance of all necessary shoring and temporary excavation slopes, and the protection of adjacent offsite site improvements and utilities are the responsibilities of the contractor.

4.4 GROUNDWATER CONSIDERATIONS AND UNDERDRAINS

Groundwater levels are anticipated to fluctuate due to seasonal variations and other factors. Basement retaining walls should be backdrained to prevent the buildup of hydrostatic pressures. Localized groundwater may also be present or rise to near or above the elevation of the basement floor. Therefore, as a precautionary measure, we recommend that an underdrain system be installed to prevent the buildup of water beneath the basement floor slab. Below-grade portions should be appropriately waterproofed.

Groundwater was not encountered in our test borings. Because the main excavation will likely be above the water table, overall site dewatering will likely not be necessary. However, localized groundwater control may be required in portions of the site depending upon groundwater conditions encountered during construction or to address seepage zones and/or locally perched groundwater. We anticipate that, should dewatering be required, it will likely involve pumping from sumps or other low points within the excavation. The design, installation, and maintenance of all necessary systems for groundwater control and dewatering during construction are the responsibilities of the contractor.

5.0 RECOMMENDATIONS

5.1 SEISMIC DESIGN BY UNIFORM BUILDING CODE (UBC)

The structure should be designed to resist the lateral forces generated by earthquake shaking in accordance with local design practice. This section presents seismic design criteria for the 1997 UBC. The Hayward fault is the governing earthquake for the site.

As defined in the 1997 UBC, we judge the following criteria to be appropriate for the site:

Seismic zone factor (Z) = 0.40

Soil profile type = S_C

Seismic coefficient: $C_a = 0.40\ N_a$
$C_v = 0.56\ N_v$

Near source factor: $N_a = 1.5$
$N_v = 2.0$

5.2 SITE GRADING

5.2.1 Site Preparation and Excavation

Underground utilities within the area to be graded should be identified and either protected or relocated. Areas within the limits of grading should be cleared of brush, trees, and grassy vegetation and be grubbed to the depth necessary to remove any tree roots or stumps. Near-surface soils containing organic material should be stripped. Site strippings are not suitable for later use as engineered fill and should be removed from the site or stockpiled for later use as landscape material.

Prior to excavation for the new building, the site should be appropriately shored and adjacent improvements protected. Excavated soil and rock should be checked and approved by Fugro prior to reuse. Materials not approved for reuse should be removed from the site.

Where significant depressions or fill from abandoned utilities, grubbing, or fault trenching extend below planned improvements, they should be excavated to expose firm soil or rock and backfilled with properly compacted fill (as described in subsequent sections of this report).

5.2.2 Fill and Backfill Materials

Fill materials may be required as backfill beneath the slab-on-grade, around site utilities and new foundations. On-site soils having an organic content of less than 3 percent by volume may be used as fill. Imported fill should have a liquid limit not exceeding 40 percent and a plasticity index not exceeding 15. The fill should contain no environmental contaminants or construction debris. Fill should not contain rocks or lumps larger than 4 inches in greatest dimension and contain no more than 15 percent larger than 2.5 inches.

Based on the borings, we judge that the onsite soil is generally acceptable for use as fill; however, the project Geotechnical Engineer should confirm this during construction when soil is

exposed. Imported fill should meet the requirements stated above. Recommendations for utility pipe bedding and utility trench backfill are presented in Section 5.2.6 below.

5.2.3 Fill Placement

Fill materials satisfying the criteria described in Section 5.2.2 should be placed at the optimum moisture content to two percent above the optimum moisture content, spread in lifts not exceeding 8 inches in uncompacted thickness, and compacted to at least 90 percent relative compaction (as determined by the American Society for Testing and Materials [ASTM] Method D1557). Fill materials should be placed in a manner that minimizes lenses, pockets and/or layers of materials differing substantially in texture or gradation from surrounding materials. Fill placed under structures should be compacted to at least 95 percent relative compaction.

5.2.4 Subgrade Preparation for Slab-on-grade

The areas to receive new concrete slabs-on-grade should be properly prepared prior to construction. Any soft or loose areas should be identified and compacted or replaced with properly compacted fill. The completed subgrade should be firm and non-yielding, and should be protected from damage caused by traffic.

5.2.5 Pipeline Bedding/Trench Backfill

Utility pipes should be bedded in clean sands (conforming to the State of California Department of Transportation (Caltrans) Standard Specification Section 19-3.025B) that extend to at least 12 inches above the tops of the pipes. Pipeline trenches should be backfilled with fill materials satisfying the criteria described in Section 5.2.2, placed in lifts of approximately 8 inches in uncompacted thickness. However, thicker lifts can be used provided the method of compaction is approved by Fugro and the required minimum degree of compaction is achieved. Trench backfill should be compacted to at least 90 percent relative compaction by mechanical means only (jetting should not be permitted).

5.2.6 Surface Drainage

The finished surface adjacent to the building should be graded to direct surface water away from foundations and toward suitable discharge facilities. Ponding of surface water should not be allowed adjacent to the structure or on pavements. Roof downspouts should be connected to suitable discharge facilities through closed pipes or discharged onto pavements that drain to an appropriate collection point.

5.3 FOUNDATION DESIGN

5.3.1 Spread Footing Foundations

The new building compressive loads can be supported on spread footing foundations that bear on bedrock. Spread footings should be setback at least 10 feet (horizontally) from the slope face. We recommend that the footings be designed using the maximum allowable bearing pressures presented in the following table:

Allowable Bearing Pressures

Load Condition	Allowable Bearing Pressure (pounds per square foot)
Dead load	4,200
Dead plus sustained live loads	5,000
Total loads, including wind or seismic	6,300

New footings should extend to a depth of at least 24 inches below lowest adjacent grades, or to the depth of bedrock, whichever is deeper. Footing excavations should be checked by the project Geotechnical Engineer for proper depth, bearing, and cleanout prior to the placement of reinforcing steel. All foundation excavations should be kept moist and free of loose soil and standing water prior to concrete placement.

5.3.2 Design of Tiedown Anchors

Seismic uplift can be resisted by tiedowns that gain support in skin friction in rock. The design loads that a tiedown must resist are determined during the design of the structure. The maximum allowable axial displacement at the top of the tiedown should also be determined during the structural design considering such factors as the structure's sensitivity to movement and the allowable displacements and/or rotations of the tied-down element(s) determined in the structural analysis.

Tiedowns should be spaced no closer than 3 drill hole diameters, on center. Tiedowns are a permanent structural component and should be protected against corrosion. The level of corrosion protection should be specified by the structural engineer. We recommend that tiedowns that are subject to permanent lock-off loads be equipped with double corrosion protection (DCP) and that appropriate materials and procedures be used during construction to assure the continuity of the DCP system throughout the length of the tiedown. Less stringent corrosion protection may be appropriate for tiedowns that are not subject to permanent lock-off loads.

We recommend that the contractor be responsible for providing tiedowns of the required design capacity that meet the specified deflection limits. Actual tiedown lengths and details regarding tiedown construction and installation procedures should be the responsibility of the contractor. Fugro should review the contractor submittals regarding the proposed lengths, materials, and installation and testing methods for tiedowns.

For costing and engineering purposes, we have performed engineering analyses and developed a preliminary design for the tiedowns. At the time of this report, the tiedown design loads had not yet been determined. Our preliminary design is based on the assumption that each tiedown will be required to resist a seismic design load of 200 kips. The actual tiedown design should be determined by the design-build team using the site-specific geotechnical data contained in this report subject to the review and approval of the structural engineer and Fugro.

For preliminary design purposes, we recommend that minimum tiedown lengths be estimated assuming a 15-foot unbonded length at the top of the tiedown. We estimate that a minimum 35-foot-long bond zone will be required to provide an allowable uplift capacity of 200 kips. This estimated bond length is based on the assumption that the tiedowns will be pressure grouted and/or post-grouted. The estimated bond length and capacity indicated above are

based on a geotechnical evaluation and include a factor of safety of 1.5. The project structural engineer should evaluate the structural capacities of tiedowns.

If tiedowns are to be installed in groups, the Geotechnical Engineer should check group capacities to assure that the sum of the individual capacities do not exceed that of the group.

5.3.3 Tiedown Load Testing

All tiedowns should be load tested in tension to verify that the specified capacity and deflection criteria are met. The magnitude of the required test loads should be considered when evaluating the structural capacity of the tiedowns. In general, the central reinforcing elements of the tiedown should be sized so that the axial stress during load testing does not exceed 80 percent of the bar's ultimate yield strength.

To verify the unfactored (ultimate) capacity of the tiedowns, at least one tiedown should be load tested to at least 150 percent of the design seismic capacity. Tiedowns to be tested to this load will likely require additional steel area relative to subsequent tiedowns that can be tested to lesser loads.

Subsequent tiedowns having the same bond length and outside diameter can be tested to a lesser load. We recommend that production tiedowns be load tested to at least 120 percent of the design seismic capacity or 160 percent of the design dead plus sustained live load capacity, whichever is greater.

During tensile load testing, the tiedowns should be stressed in increments not to exceed 25 percent of the test load while monitoring the axial movement of the pile. Acceptance criteria should include the following:

- At least one tiedown should resist a test load of at least 150 percent of the design seismic capacity or 200 percent of the design dead plus sustained live load capacity, whichever is greater without failure. Failure should be defined as continued pile top displacement without supporting an increase in applied load.
- All tiedowns should support service load values (100 percent of design seismic capacity or design dead plus sustained live load capacity, whichever is greater) with a total pile top displacement to be specified by the structural engineer. We recommend that the allowable deflections under service loads not exceed ½ inch.
- Tiedown load test procedures and acceptance criteria should be specified by the structural engineer, in consultation with Fugro. We should observe the installation and load testing of the tiedowns and confirm the specified deflection limits are not exceeded.

5.3.4 Drilled Pier Foundations

As an alternative to spread footing foundations, the new building can be supported on drilled CIDH piers that gain support from skin friction in the rock. The CIDH piers should be at least 24-inches in diameter and penetrate at least 10 feet into bedrock. The dead plus live load axial capacities of the drilled piers can be calculated using an allowable skin friction of 1,500 pounds per square foot (psf) for the bedrock. Skin friction along the upper 2 feet of the pier

shaft and end bearing should be neglected when calculating pier capacities. The allowable dead plus live load skin friction values may be increased by one-third for total loads including wind or seismic. The uplift capacity of CIDH piers in rock can be assumed to be equivalent to the calculated downward dead plus live load capacity of the CIDH pier. The structural engineer should check the structural capacity of the piers. The piers should have a minimum center-to-center spacing of three times the pier diameter. Piers located at closer spacing will have a reduced compressive capacity caused by group effects and should be evaluated on an individual basis by the Fugro.

We recommend that the contractor review the boring logs prior to establishing a method of drilling the piers. Holes for the drilled piers should be drilled straight and plumb (within 1 percent of the vertical) and cleaned of loose soil and rock fragments. We judge that the holes can be drilled using conventional heavy auger drilling equipment. However, localized zones of relatively hard rock could be encountered. The foundation contractor should be prepared to utilize suitable hard rock drilling techniques, if necessary.

Concrete placement should start as soon as possible after the drilling and clean out is complete. In all cases, pier holes should be concreted on the day they are drilled. The reinforcing cage should be constructed with a minimum of splices and should be placed in a relatively continuous operation. Holes should be concreted from the bottom up in a single operation using appropriate tremie methods. Concrete should be vibrated over the full depth of the reinforcing cage.

The drilled piers should be installed by a qualified drilling contractor with demonstrated experience in hillside drilling. We recommend that Fugro observe drilled pier installation to assure that the subsurface conditions are as anticipated and that the piers are constructed in accordance with the recommendations presented in this report.

5.3 LATERAL RESISTANCE

Lateral loads can be resisted by (1) passive pressure acting against the face of the footings or pile caps and grade beams; (2) frictional resistance acting over the bottom of the footings that are parallel to the direction of loading; and (3) the interaction of the piles with the surrounding soils.

A passive pressure equal to an equivalent fluid pressure of 350 pounds per cubic foot (pcf) can be used for lateral load resistance against the sides of footings or pile caps and grade beams perpendicular to the direction of loading. The upper one foot of soil should be ignored, unless it is confined by pavement or a slab. The passive resistance is based on a factor of safety of 2.0. However, relatively large deflections would be required to mobilize the ultimate passive resistance. Therefore, in order to limit deformations to less than about ¼-inch, we recommend that the passive resistance should be considered as an ultimate value.

A friction coefficient of 0.35 should be used to evaluate frictional resistance along bottoms of footings. Dead load should be used to compute the frictional resistance along the bottom of footings, and frictional resistance along the bottom of pier-supported elements should be ignored.

The lateral resistance from the interaction of drilled piers with the surrounding soils can be calculated using an equivalent fluid weighing 350 pcf acting on the upper 3 feet of the piers over twice the pier diameter. Additional lateral resistance can be provided by the structural rigidity of the piers. If required, Fugro can provide additional lateral capacity (p-y) curves for the drilled piers.

5.4 RETAINING WALLS

Retaining walls should be designed to resist the equivalent fluid pressures presented below. Below grade building walls should be designed as restrained retaining walls.

Cantilever (Free to Rotate) Retaining Walls	
Backfill Condition	**Equivalent Fluid Weight**
Level	35 pcf
3:1	45 pcf
2:1	55 pcf

Restrained Retaining Walls	
Backfill Condition	**Equivalent Fluid Weight**
Level	50 pcf
3:1	60 pcf
2:1	70 pcf

Where traffic and/or heavy equipment loads are anticipated, walls should be designed for an additional uniform lateral pressure equal to 100 psf over the entire height of the wall. For general surcharge loads behind the wall, a uniform lateral pressure of 0.5 times the anticipated surcharge load applied over the full height of the wall may be used. For seismic design, the retaining wall should be designed to resist an additional uniform load equal to 18H pound per square foot, where H equals the height of soil retained by the wall in feet. The retaining wall can be supported on foundations designed in accordance with the recommendations presented in Section 5.3.

The above pressures are based on retaining walls that are fully backdrained to prevent the buildup of hydrostatic pressure. Wall drainage should consist of a drain rock layer at least 12 inches thick that extends to within 1 feet of the ground surface. A four-inch-diameter perforated PVC pipe shall be installed (with perforations down) at least 1 foot below the finished slab grade. The pipe should sit on a two-inch-thick bed of drain rock. The pipe shall be sloped to drain by gravity or be pumped to a suitable drainage facility.

Drain rock shall conform to Caltrans specifications for Class 2 permeable material. If clean 1/2- to 3/4-inch maximum size crushed rock or gravel are available locally, it could be used as an alternative, provided it is encapsulated in a non-woven geotextile filter fabric, such as Mirafi 140N or an approved alternative. A 1-foot-thick cap of clayey soil should be placed over the drain rock to inhibit surface water infiltration. As an alternative to drain rock, prefabricated drainage material (Miradrain or an approved alternative) may be used. Prefabricated drainage material should be installed in accordance with the manufacturer's recommendations.

5.5 CONCRETE FLOOR SLABS AND UNDERDRAINS

Soil subgrades beneath concrete slabs-on-grade for the building should be properly prepared according to the recommendations of Section 5.2.4 and should be relatively smooth and non-yielding under equipment loads.

We recommend that underdrains be installed to prevent the buildup of groundwater beneath the concrete floor slab. Underdrains should consist of four-inch-diameter perforated plastic pipe installed perforations down below the floor slab on a two-inch-thick bed of drain rock. We recommend that underdrain pipes be installed within 10 feet of the building perimeter and on 20-foot (maximum) centers. Underdrain pipes should be sloped to drain by gravity to a sump or other suitable discharge facility.

A layer of clean, angular crushed rock, at least 4 inches thick, should be placed beneath interior slabs to provide a capillary moisture break. The crushed rock should conform to the following gradation criteria:

Sieve Size	Percent Passing
1 inch	100
3/4 inch	90 – 100
No. 200	0 – 3

If the migration of water vapor through the floor slab is unacceptable, a vapor barrier should be considered. The vapor barrier should consist of an impermeable membrane at least 10 mil thick placed above the crushed rock. The membrane should be covered with 2 inches of sand for protection during construction.

5.6 SERVICES DURING CONSTRUCTION

We should review the geotechnical aspects of project plans and specifications, to check for conformance with the intent of our recommendations. Prior to construction, we should review all submittals from contractors or vendors that are geotechnical in nature including: shoring, dewatering, drilled piers, spread footings, tiedown anchors, retaining walls, reinforced slabs-on-grade and earthwork. During construction, our engineer should observe and/or test the following:

- Site and subgrade preparation
- Spread footing installation
- Tiedown installation and testing (if appropriate)
- Drilled pier installation (if appropriate)
- Installation of backdrains and underdrains
- Placement and compaction of fill and backfill

6.0 LIMITATIONS

Our services consist of professional opinions, conclusions, and recommendations that are made in accordance with generally accepted geotechnical engineering principles and practices. This warranty is in lieu of all other warranties, either expressed or implied.

Variations may exist and conditions not observed or described in this report could be encountered during construction. Our conclusions and recommendations are based on an analysis of the observed conditions. If conditions other than those described in this report are encountered, our offices should be notified so that additional recommendations, if warranted, can be provided.

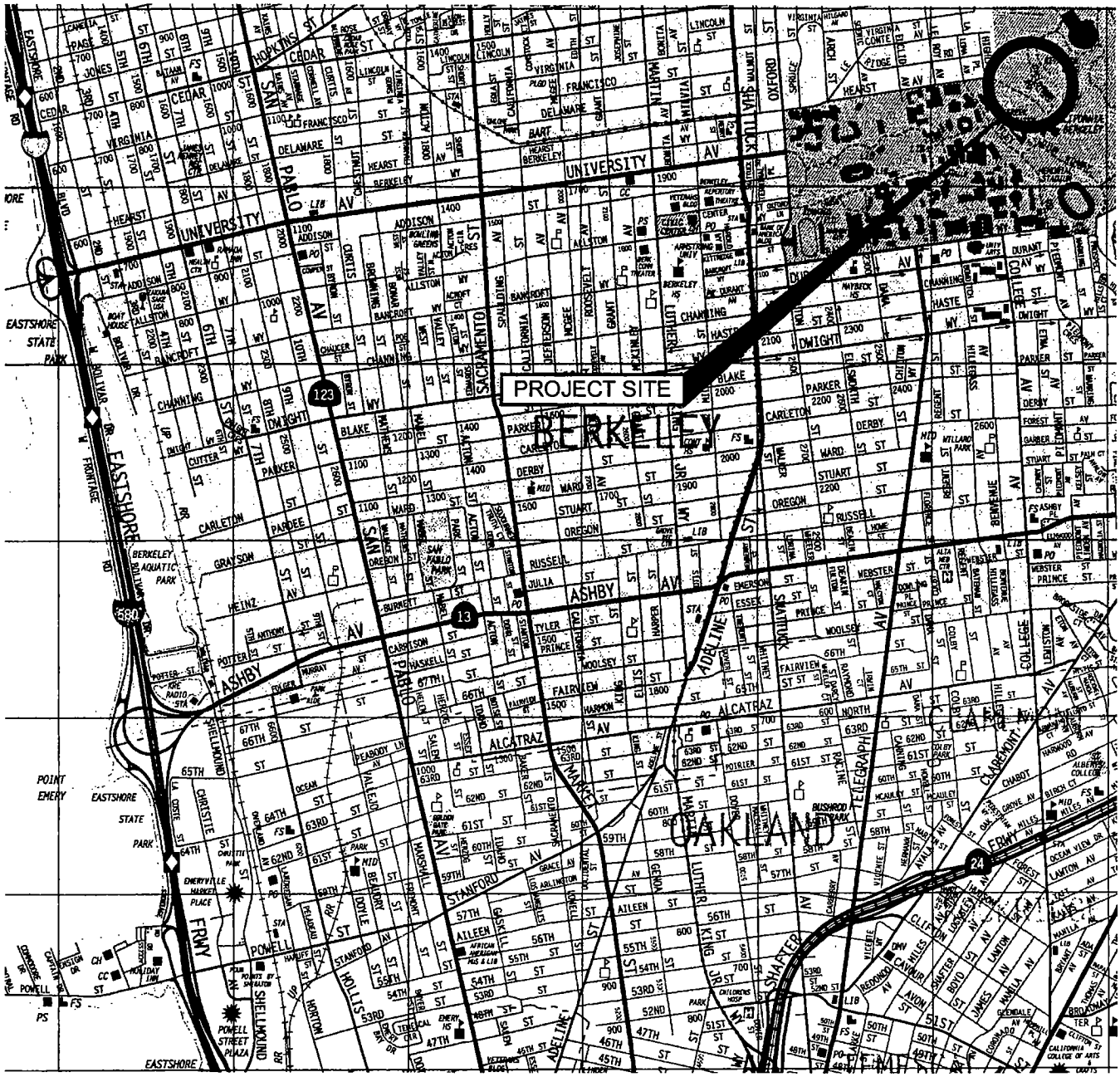
This report has been prepared for the exclusive use of the LBNL and their consultants for specific application to the proposed Building 50X project as described herein. In the event that there are any changes in the ownership, nature, or design, of the project, the conclusions and recommendations contained in this report shall not be considered valid unless (1) the project changes are reviewed by Fugro West, Inc., and (2) conclusions and recommendations presented in this report are modified or verified in writing. Reliance on this report by others must be at their risk unless we are consulted on the use or limitations. We cannot be responsible for the impacts of any changes in geotechnical standards, practices, or regulations subsequent to performance of services without our further consultation. We can neither vouch for the accuracy of information supplied by others, nor accept consequences for unconsulted use of segregated portions of this report.

ILLUSTRATION(S)

385_00020

PLATES

May 10, 2002
Project No. 658.052



NOTE:

This Vicinity Map Is Based On A Thomas Guide Map For San Francisco, Alameda And Contra Costa Counties, California, Map 57, Year 2000

NORTH

0 2400

FEET

SITE MAP

Building 50X

Berkeley, California

PLATE 1

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385_00022

May 10, 2002
Project No. 658.052

BASE MAP SOURCE: Provided by Mr. Steve Blair of Lawrence Berkeley National Laboratory.

LEGEND

B-1	Approximate Location of Test Boring
	Upper Cretaceous Rocks, Undivided
Qc	Quaternary Colluvium
Qaf	Artificial Fill
SR-2	Approximate Location of Seismic Refraction Traverse
	Soil Creep
	Approximate Location of Geologic Contact

NORTH

0 30 60
FEET

SITE PLAN
Building 50X
Berkeley, California

PLATE 2

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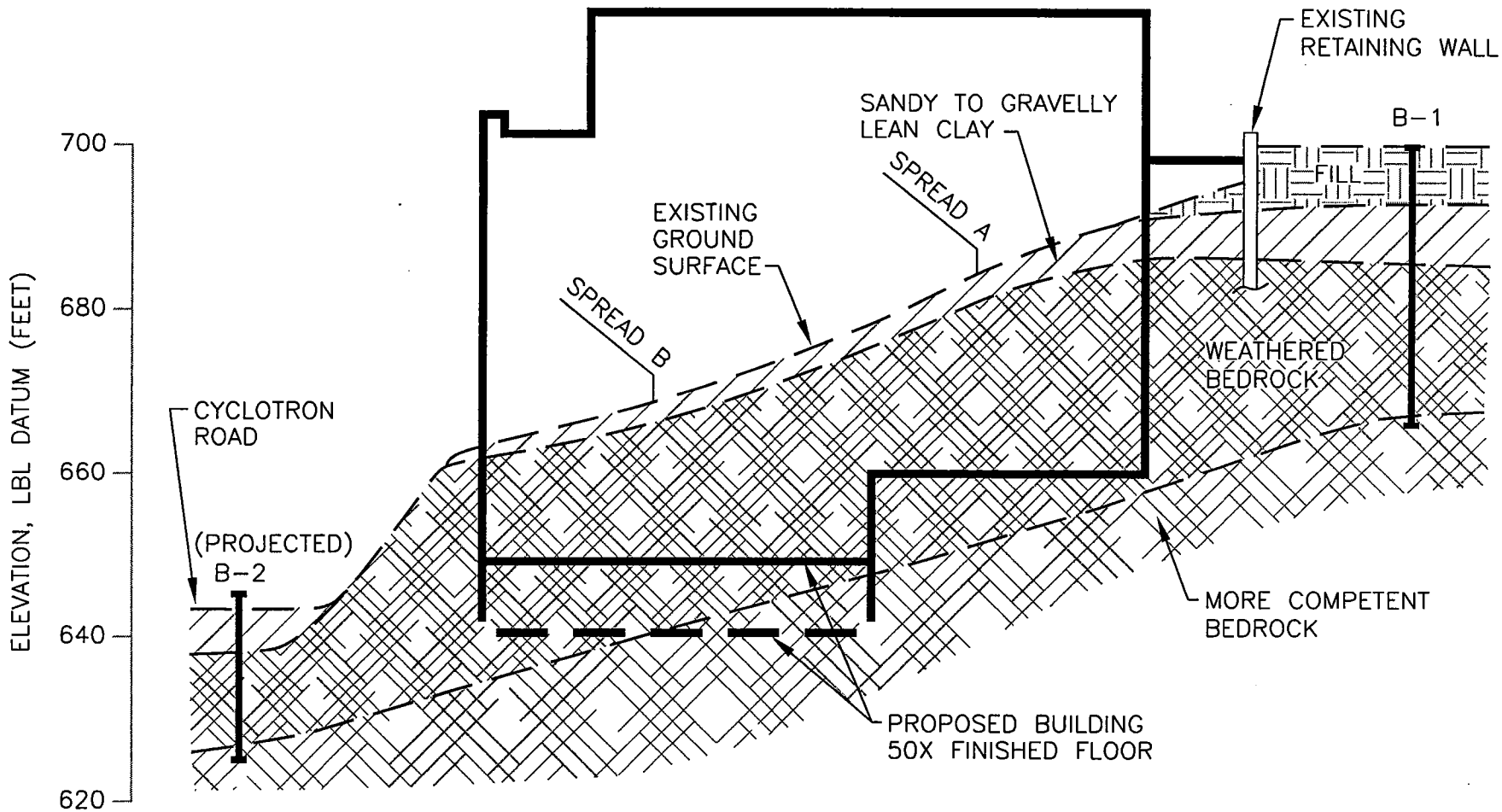

NOTES:

1. Elevations referenced to Lawrence Berkeley Laboratory (LBL) project datum.
2. The stratification lines shown were interpreted from boring explorations and represent the approximate boundaries between soil types. The actual transitions between soil types may vary from those shown and may be far more or less gradual.

CROSS-SECTION A-A'
Building 50X
Berkeley, California

APPROXIMATE SCALE
0 20
FEET

PLATE 3

APPENDIX A

385_00025

APPENDIX A
FIELD INVESTIGATION

APPENDIX A
FIELD INVESTIGATION

Our field exploration was performed on April 27, 2002. Our work included two exploratory borings drilled using conventional truck-mounted drilling equipment equipped with 6-1/2 inch diameter hollow stem flight augers. The borings extended to depths of approximately 24 to 33 feet. The approximate locations of the borings are shown on the Site Plan, Plate 2. The soils encountered in the borings were logged in the field by our representative. The soils are described in accordance with the Unified Soil Classification System (ASTM D2487). The logs of the borings as well as a key for the classification of the soil (Plate A-1) and rock classification criteria (Plate A-2) are included as part of this appendix.

Representative soil samples were obtained from the borings at regular intervals using a Modified California split-barrel drive sampler (outside diameter of 3.0 inches, inside diameter of 2.5 inches) and a Standard Penetration Test (SPT) split-barrel drive sampler (outside diameter of 2.0 inches, inside diameter of 1.375 inches).

Resistance blow counts were obtained with the samplers by dropping a 140-pound down-hole hammer through a 30-inch fall. The sampler was driven 18 inches, or a shorter distance where hard resistance was encountered, and the number of blows were recorded for each 6 inches of penetration. The blows per foot recorded on the boring logs represent the accumulated number of blows that were required to drive the last 12 inches, or the number of inches indicated where hard resistance was encountered.

The attached boring logs and related information show our interpretation of the subsurface conditions at the dates and locations indicated, and it is not warranted that they are representative of subsurface conditions at other locations and times.

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D2487-93)

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

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

KEY TO TEST DATA AND SYMBOLS

Perm	-	Permeability
Consol	-	Consolidation
LL	-	Liquid Limit
PI	-	Plasticity Index
Gs	-	Specific Gravity
MA	-	Particle Size Analysis
-200	-	Percent Passing No. 200 Sieve
ND	-	Not Detected
■	-	Tube Sample
⊠	-	Bag or Bulk Sample
◫	-	Lost Sample
▽	-	First Groundwater
▼	-	Stabilized Groundwater

	Shear Strength (psf)	Confining Pressure (psf)	
TxUU	3200	(2600)	Unconsolidated-Undrained Triaxial Shear
TxCU	3200	(2600)	Consolidated-Undrained Triaxial Shear
TxCD	3200	(2600)	Consolidated-Drained Triaxial Shear
SSCU	3200	(2600)	Consolidated-Undrained Simple Shear
SSCD	3200	(2600)	Consolidated-Drained Simple Shear
DSCD	2700	(2000)	Consolidated-Drained Direct Shear
UC	470		Unconfined Compression
LVS	700		Laboratory Vane Shear
FV	300		Field Vane Shear
RFV	300		
TV	800		Torvane Shear
PP	400		Pocket Penetrometer *(actual reading divided by 2)*

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Building 50X
Berkeley, California

JOB NUMBER	DATE	APPROVED
658.052	5/02	

PLATE **A1**

BEDDING OF SEDIMENTARY ROCKS

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

FRACTURING

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

HARDNESS

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

STRENGTH

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

WEATHERING

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

ROCK CLASSIFICATION CRITERIA

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Building 50X
Berkeley, California

JOB NUMBER	DATE
658.052	5/02

PLATE **A2**

LOG OF BORING

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

Depth (feet)	Sampler Type	Blows/6 inches or pressure	Blows/12 inches	Sample Interval	Graphic Log	SOIL DESCRIPTIONS: GROUP NAME (GROUP SYMBOL) color, consistency/density, moisture condition, other descriptions (Local Name or Material Type)	Moisture Content (%)	Dry Density (pcf)	Other
0						**ASPHALTIC CONCRETE 3 - INCHES THICK**			
						GRAVELLY LEAN CLAY (CL) Dark brown, soft, moist (fill)			
	A	2 2 2	4				13.9	92	
5	A	12 17 25	42			**GRAVELLY LEAN CLAY (CL)** Brown, stiff to very stiff, moist With rock fragments at 5.0'	15.1	102	
	A	8 7 8	15			With reddish brown sand pockets	18.3	102	
10									
	A	12 17 25	42						
15						**SANDSTONE** Brown, intensely to moderately fractured, low to moderately hard, friable, moderate to deep weathering			
	A	18 35 40	75				21.6	102	TxUU = 2830 psf PP = 3250 psf
20									
	A	16 23 28	51				18.8	108	PP = 3750 psf
25						Trace reddish brown sandstone fragments Gray clay filled seams			
	B	7 11 15	26			Color change to brownish gray at 29.0'			
30						Hard drilling at 32.0'			
	B	50/4"	50/4"			Refusal at 33.0'			
35						Bottom of boring at 33.4 feet below ground surface.			
40									

Notes
Groundwater not encountered during drilling

FUGRO LOG OF BORING 658-052.GPJ SCI_CORP.GDT 6/13/02

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Building 50X
Berkeley, California

JOB NUMBER 658.052 | DATE 6/02 | BORING **B-1**

LOG OF BORING

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

Depth (feet)	Sampler Type	Blows/6 inches or pressure	Blows/12 inches	Sample Interval	Graphic Log	SOIL DESCRIPTIONS GROUP NAME (GROUP SYMBOL) color, consistency/density, moisture condition, other descriptions (Local Name or Material Type)	LABORATORY DATA Moisture Content (%)	Dry Density (pcf)	Other
0						**ASPHALTIC CONCRETE 4 - INCHES THICK**			
						SANDY LEAN CLAY WITH GRAVEL (CL) Dark brown, stiff, moist, with siltstone at 3 feet (fill)			
	A	8 11 16	27				13.3	111	
5	A	6 8 13	21			**SANDY LEAN CLAY WITH GRAVEL (CL)** Brown, stiff, moist	16.7	110	
	A	9 11 30	41				18.4	104	TxUU = 1740 psf
10						**SANDSTONE** Brown, intensely to moderately fractured, low to moderately hard, friable to weak, moderate weathering			
	A	50/6"	50/6"				10.5	111	PP = 2000 psf
15	B	17 18 22	40				18.4	99	
20	B	20 30 50	80				7	121	
	B	50/4"	50/4"						
25						Bottom of boring at 24 feet below ground surface. Notes Groundwater not encountered during drilling			
30									
35									
40									

FUGRO LOG OF BORING 658-052.GPJ SCI_CORP.GDT 6/13/02

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Building 50X
Berkeley, California

JOB NUMBER	DATE	BORING
658.052	6/02	B-2

APPENDIX B

385_00032

APPENDIX B
LABORATORY TESTING

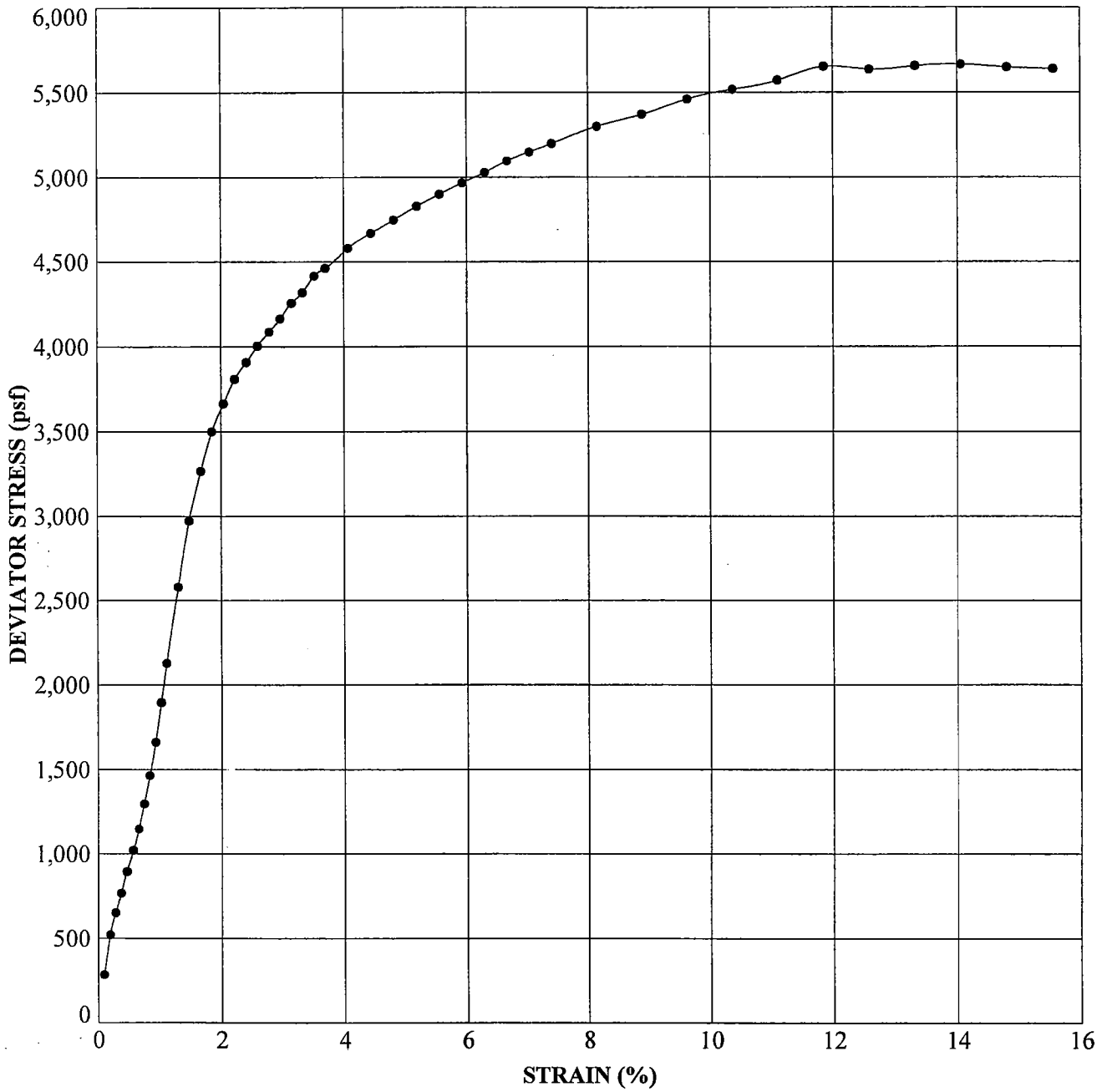
APPENDIX B
LABORATORY TESTING PROGRAM

The laboratory testing program was directed towards an evaluation of the physical and mechanical properties of the soils underlying the site.

The natural water content determinations were determined on 11 samples of the materials recovered from the borings in accordance with ASTM Test Designation D2216. Dry density determinations were determined on 11 samples of the materials recovered. The results are recorded on the boring logs at the appropriate sample depths.

Unconsolidated undrained triaxial tests were performed on two relatively "undisturbed" samples to evaluate the undrained shear strength of the materials. The strength tests was performed in accordance with ASTM Test Designation D2850 on a sample having a diameter of 2.4 inches and a height-to-diameter ratio of at least two. Failure was taken as the peak normal stress.

385_00034

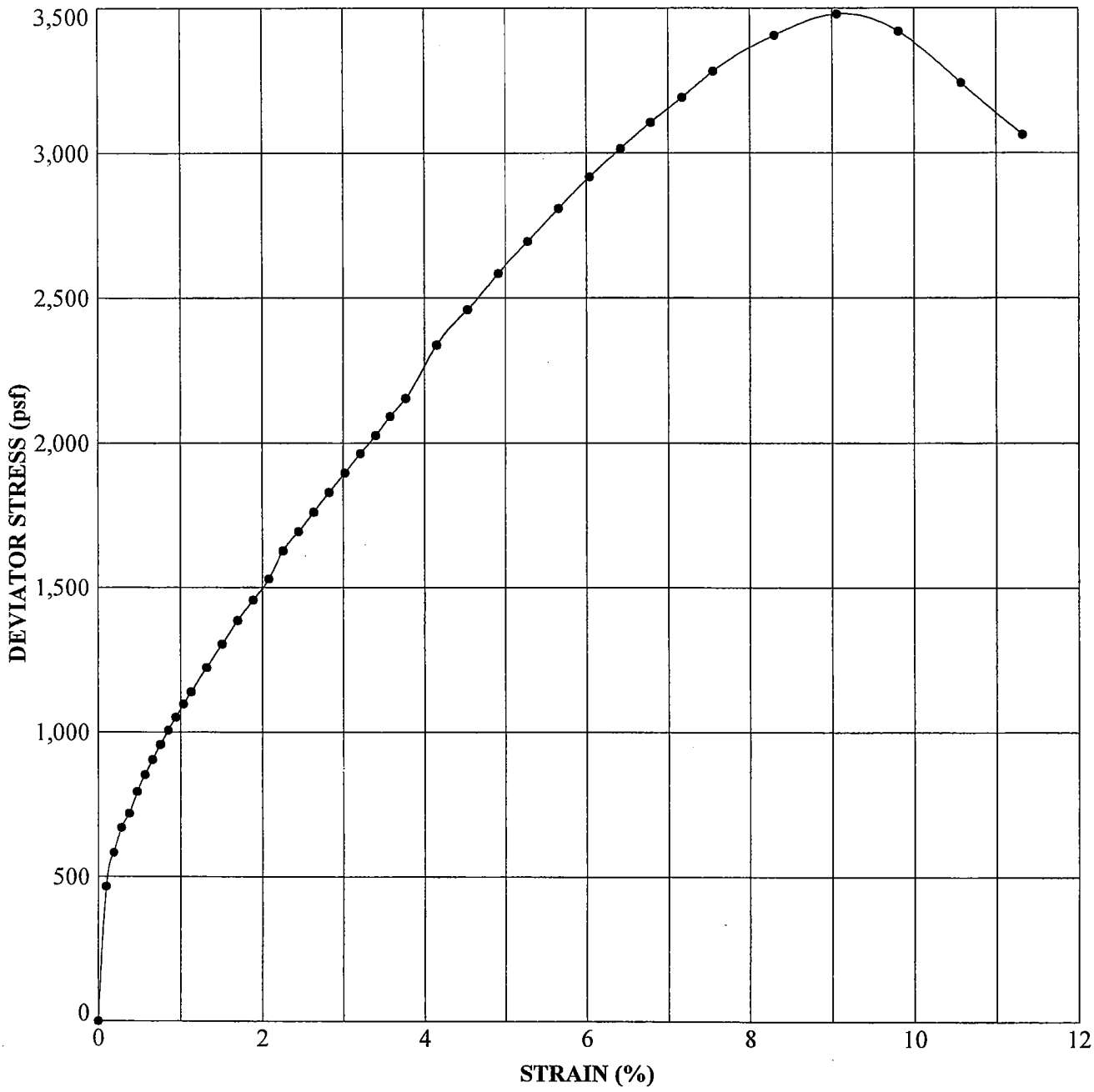


Key Symbol	Boring	Depth (Feet)	Sample Description (USCS)	Dry Density (pcf)	Water Content (%)	Peak Deviator Stress (psf)	Strain (%)
●	1	19.0	Lt. Brown Silty CLAY, Some Sand (CL)	102.3	21.6	5,663	14.1

File Name: H:\ENGINEER\GINTW\PROJECTS\658052.GPJ Report Template: TRIAXIAL Output Date: 5/14/02

FUGRO

PREP'D BY:	TRIAXIAL COMPRESSION TEST DATA	FIGURE
APP'D BY		B-1
DATE: 5/14/02	BUILDING 50X Berkeley, California	PROJECT No.
DWG FILE: 658052.GPJ		658.052



Key Symbol	Boring	Depth (Feet)	Sample Description (USCS)	Dry Density (pcf)	Water Content (%)	Peak Deviator Stress (psf)	Strain (%)
●	2	9.0	Dark brown silty CLAY, some sand (CL)	103.5	18.4	3,479	9.1

FUGRO

PREP'D BY

APP'D BY:

DATE: 5/14/02

DWG FILE: 658052.GPJ

TRIAXIAL COMPRESSION TEST DATA

BUILDING 50X
Berkeley, California

FIGURE

B-2

PROJECT No.

658.052

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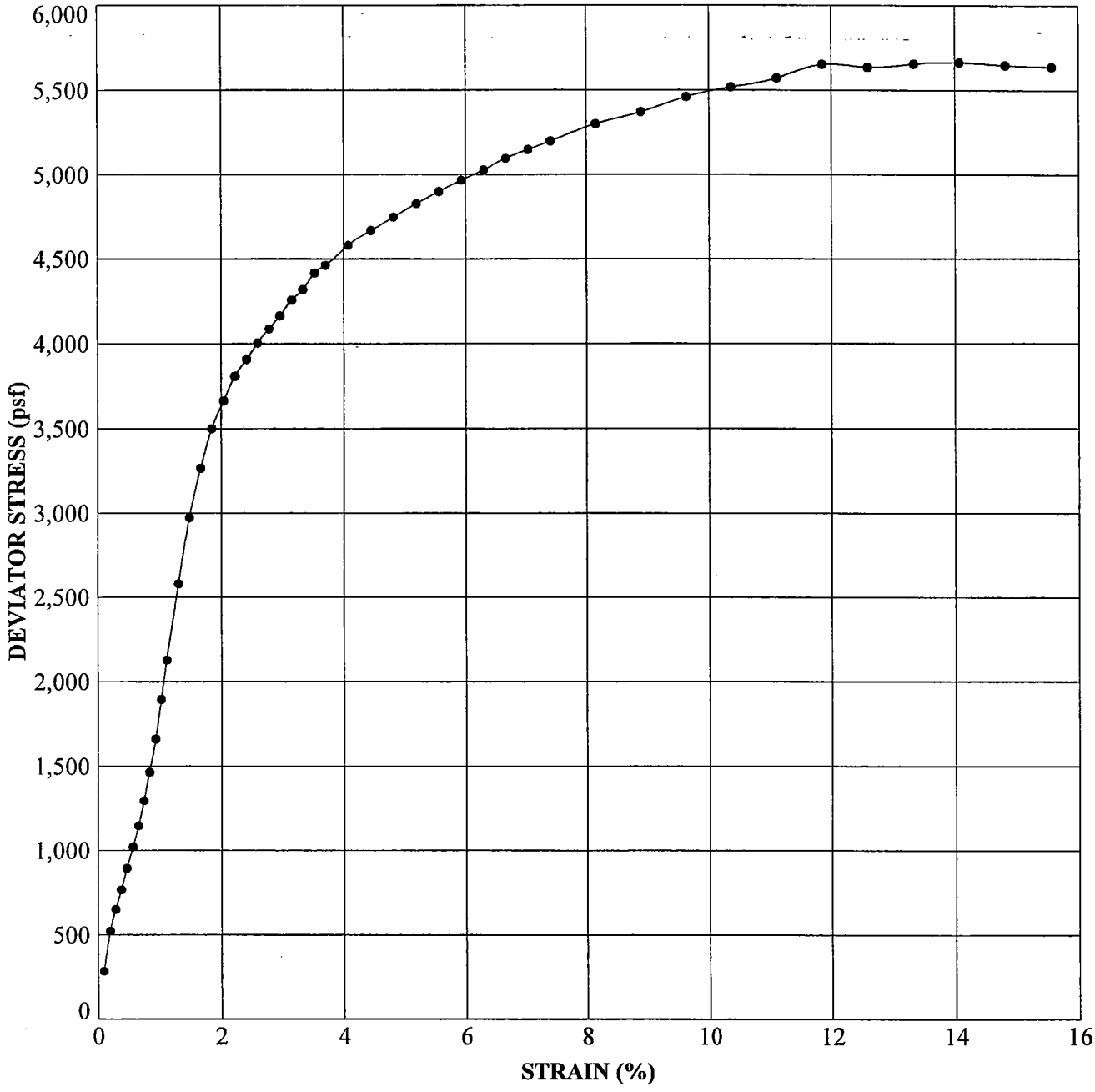
APPENDIX B
LABORATORY TESTING PROGRAM

The laboratory testing program was directed towards an evaluation of the physical and mechanical properties of the soils underlying the site.

The natural water content determinations were determined on 11 samples of the materials recovered from the borings in accordance with ASTM Test Designation D2216. Dry density determinations were determined on 11 samples of the materials recovered. The results are recorded on the boring logs at the appropriate sample depths.

Unconsolidated undrained triaxial tests were performed on two relatively "undisturbed" samples to evaluate the undrained shear strength of the materials. The strength tests was performed in accordance with ASTM Test Designation D2850 on a sample having a diameter of 2.4 inches and a height-to-diameter ratio of at least two. Failure was taken as the peak normal stress.

385_00037



Key Symbol	Boring	Depth (Feet)	Sample Description (USCS)	Dry Density (pcf)	Water Content (%)	Peak Deviator Stress (psf)	Strain (%)
●	1	19.0	Lt. Brown Silty CLAY, Some Sand (CL)	102.3	21.6	5,663	14.1

FUGRO

PREP'D BY:
APP'D BY:
DATE: 5/14/02
DWG FILE: 658052.GPJ

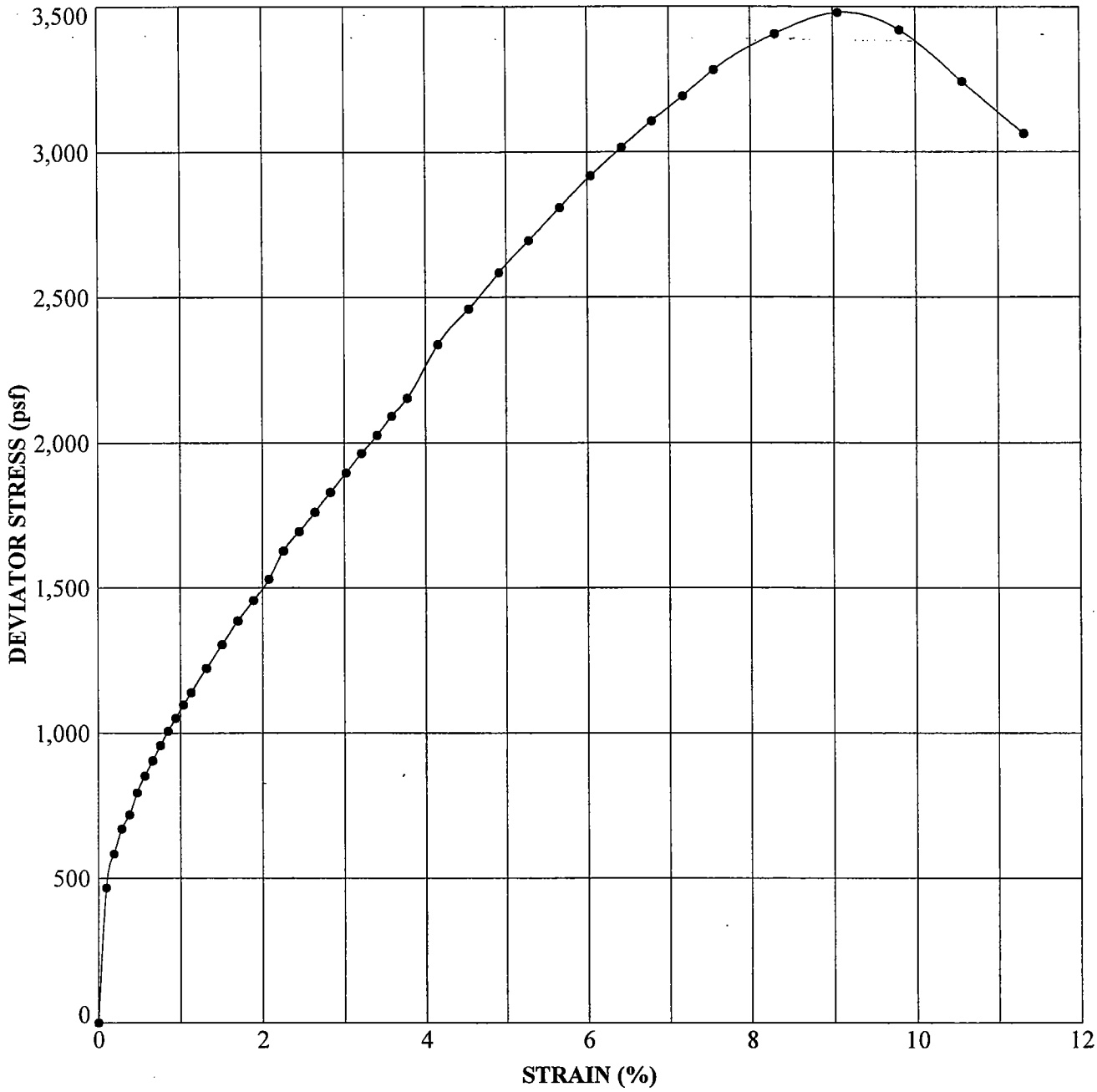
TRIAXIAL COMPRESSION TEST DATA

BUILDING 50X
Berkeley, California

FIGURE
B-1

PROJECT No.
658.052

File Name: H:\ENGINEER\GINTW\PROJECTS\658052.GPJ Report Template: TRIAXIAL Output Date: 5/14/02



Key Symbol	Boring	Depth (Feet)	Sample Description (USCS)	Dry Density (pcf)	Water Content (%)	Peak Deviator Stress (psf)	Strain (%)
●	2	9.0	Dark brown silty CLAY, some sand (CL)	103.5	18.4	3,479	9.1

FUGRO

PREP'D BY:
APP'D BY:
DATE: 5/14/02
DWG FILE: 658052.GPJ

TRIAXIAL COMPRESSION TEST DATA

BUILDING 50X
Berkeley, California

FIGURE
B-2

PROJECT No.
658.052

File Name: H:\ENGINEER\GINTW\PROJECTS\658052.GPJ Report Template: TRIAXIAL Output Date: 5/14/02

APPENDIX C

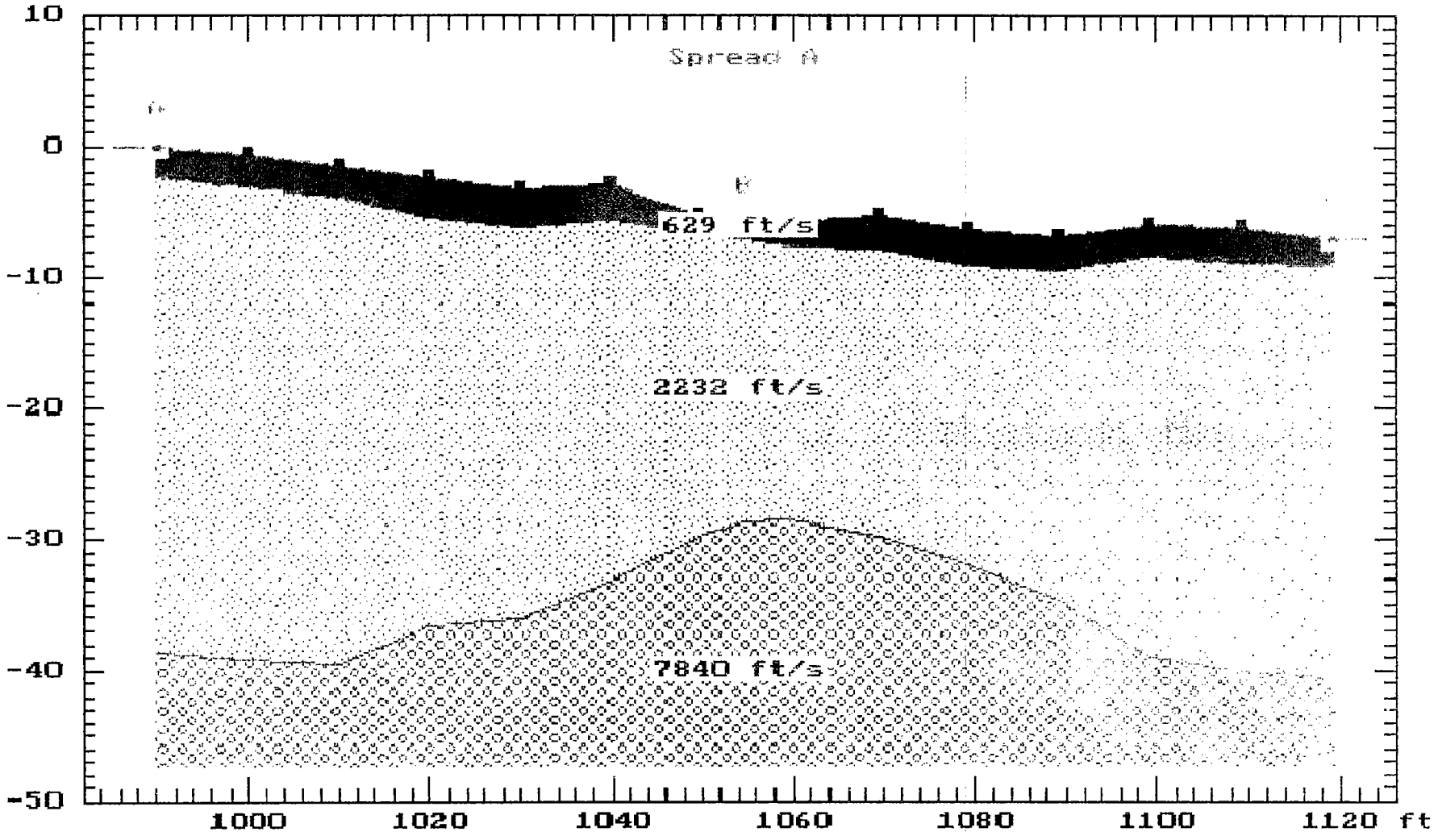
385_00040

APPENDIX C
SEISMIC REFRACTION SURVEY RESULTS

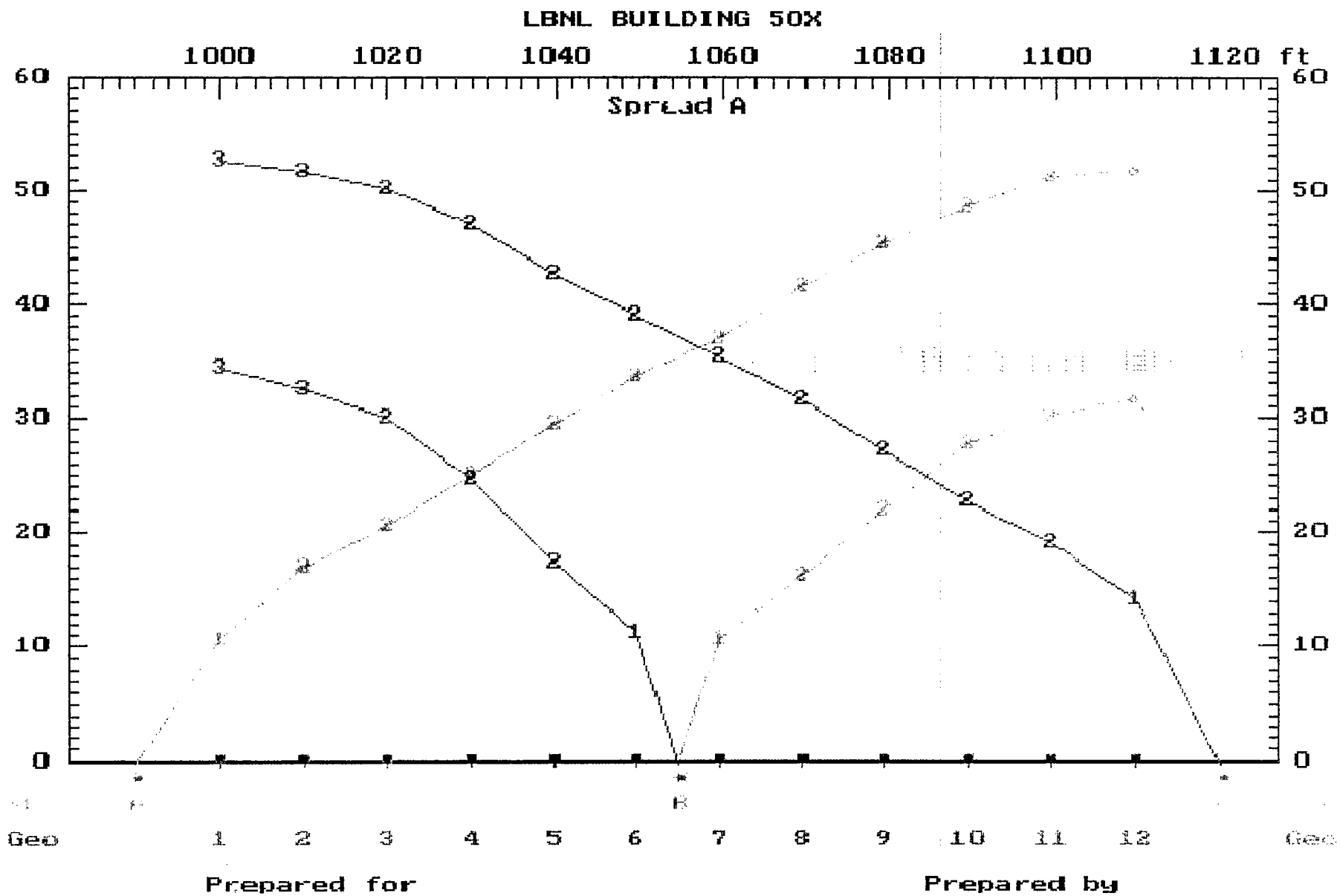
APPENDIX C
SEISMIC REFRACTION SURVEY

Two, 110-foot, surface seismic refraction traverses were performed at the project site on May 7, 2002. The purpose of utilizing the seismic refraction method was to develop a characterization of the subsurface materials that would be useful for planning of future excavation work required to develop the site. The location where the traverses were performed is shown on Plate 2. The equipment used to perform the surveys consisted of a SmartSeis, 12-channel, exploration seismograph by Geometrics. Data gathered from the field was further analyzed using software from Rimrock Geophysics. Velocity profiles and subsurface sections for both traverses can be seen in this appendix. Each profile and section should be viewed as if facing downslope. Average compression wave velocities in feet per second are presented on the subsurface sections corresponding with the three subsurface velocity zones recognized by the survey.

385_00042

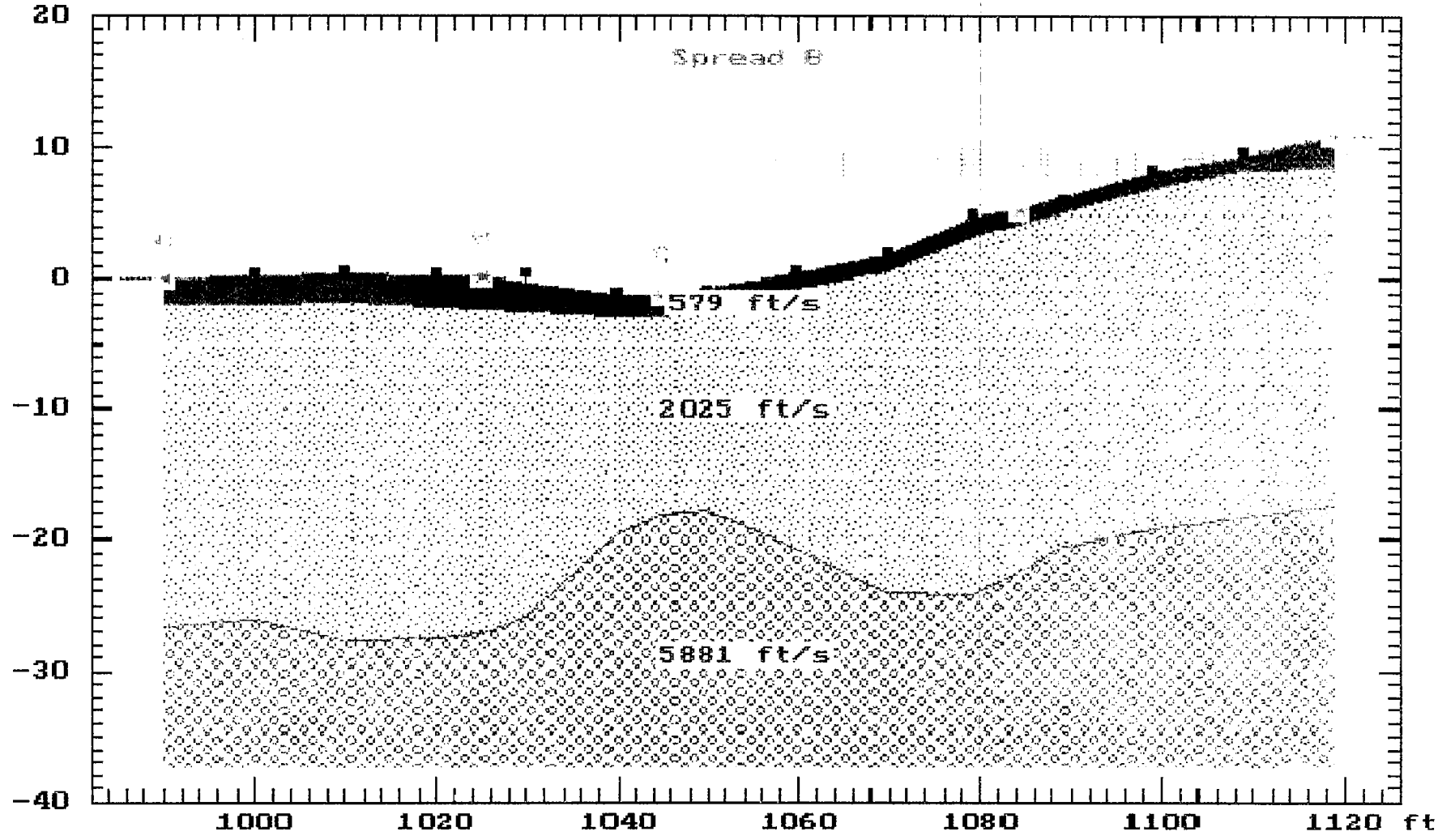


Prepared for	Prepared by
LBNL Building 50X	Fugro West, Inc.
Lawrence Berkeley National Laboratory	Oakland, CA 94621 Date: 05-15-2002



Prepared for	Prepared by
LBNL Building 50X	Fugro West, Inc.
Lawrence Berkeley National Laboratory	Oakland, CA 94621 Date: 05-15-2002

LBNL BUILDING 50X



Prepared for	Prepared by
LBNL Building 50X	Fugro West, Inc.
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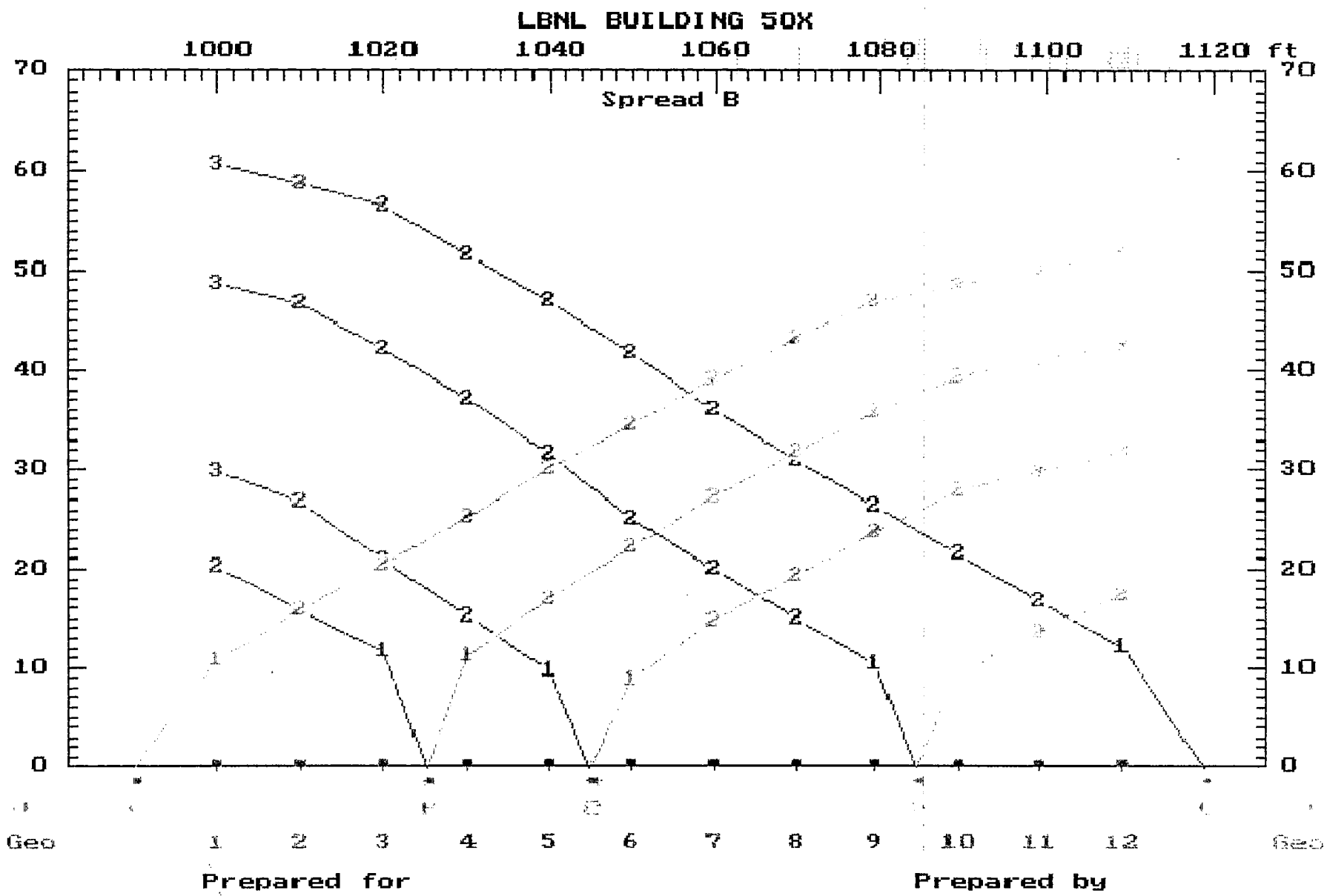

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Oakland, CA 94621 Date: 05-15-2002