Appendix E.1 Final Geologic and Soils Engineering Report Part 3

THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
1461 E. CHEVY CHASE DR. #200, GLENDALE, CA 91206
818•549•9959 TEL 818•543•3747 FAX
"Trust the Name You Know"

October 9, 2006
JB 17866-B

Harvard-Westlake School
700 North Faring Road
Los Angeles, California 90077

Attention: Jim De Matte, Director of Campus Operations

Subject

Transmittal of Geotechnical Engineering Exploration
Proposed Sports-Field Lighting
Lot 1111, Tract 1000
3700 Coldwater Canyon Avenue
Studio City, California

Gentlepersons:

The J. Byer Group has completed our report dated October 9, 2006, which describes the geotechnical engineering conditions with respect to construction of the proposed sports-field lighting. The reviewing agency for this document is the City of Los Angeles, Department of Building and Safety, Grading Section. The reviewing agency requires three unbound copies, one with a wet signature with an application form and a filing fee. Seven copies of the report are enclosed.

It is our understanding that you will file the report with the City of Los Angeles. It is suggested that you read the report carefully prior to submittal to any governmental agency. Any questions concerning the report should be directed to the project consultant. The J. Byer Group appreciates the opportunity to offer our consultation and advice on this project.

Very Truly Yours,
THE J. BYER GROUP, INC.

John W. Byer
President

THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
1461 E. CHEVY CHASE DR. #200, GLENDALE, CA 91206
818•549•9959 TEL 818•543•3747 FAX
"Trust the Name You Know"

GEOTECHNICAL ENGINEERING EXPLORATION

PROPOSED SPORTS-FIELD LIGHTING

LOT 111, TRACT 1000

3700 COLDWATER CANYON AVENUE

STUDIO CITY, CALIFORNIA

FOR HARVARD-WESTLAKE SCHOOL

THE J. BYER GROUP, INC. PROJECT NUMBER JB 17866-B

OCTOBER 9, 2006

GEOTECHNICAL ENGINEERING EXPLORATION
PROPOSED SPORTS-FIELD LIGHTING
LOT 111, TRACT 1000
3700 COLDWATER CANYON AVENUE
STUDIO CITY, CALIFORNIA
FOR HARVARD-WESTLAKE SCHOOL
THE J. BYER GROUP, INC. PROJECT NUMBER JB 17866-B
OCTOBER 9, 2006

INTRODUCTION

This report has been prepared per our signed Agreement dated September 20, 2006, and summarizes findings of The J. Byer Group, Inc., geotechnical engineering exploration performed on the site. The purpose of this study is to evaluate the nature, distribution, and engineering properties of the earth materials underlying the site with respect to the proposed sports-field lighting.

INTENT

It is the intent of this report to assist in the design and completion of the proposed project. The recommendations are intended to reduce geotechnical risks affecting the project. The professional opinions and advice presented in this report are based upon commonly accepted standards and are subject to the general conditions described in the NOTICE section of this report.

EXPLORATION

The scope of the field exploration was determined from our initial site visit and consultation with Jim De Matte, Director of Campus Operations. The preliminary plans prepared by Musco Sports-Lighting, Inc., dated September 26, 2006, and the survey by Iacobellis & Associates, Inc., dated January 23, 2001, were considered during the preparation of this report. Exploration was conducted using techniques normally applied to this type of project in this setting. This report is limited to the area of the exploration and the proposed project as shown on the enclosed Site Plan. Conditions affecting portions of the property outside the area explored are beyond the scope of this report.

Exploration was conducted on September 22, 2006, with the aid of a hollow-stem auger drill rig and hand labor. It included excavating three test pits and drilling one hollow-stem auger boring to a maximum depth of 50 feet. Samples of the earth materials were obtained from the boring at frequent intervals and delivered to the soils engineering laboratory for testing and analysis.

Office tasks included engineering analysis, laboratory testing of selected soil samples, review of our geotechnical records for the site, and preparation of the Site Plan. The earth materials exposed in the test pits and boring are described on the enclosed Log of Test Pits and Log of Boring. Appendix I contains a discussion of the laboratory testing procedures and results. The proposed project and the locations of the test pits and boring are shown on the Site Plan.

PRIOR WORK

Report by Kovacs-Byer and Associates Inc.:

Addendum Geologic and Soils Engineering Exploration, Proposed Swimming Pool and Pool House, Part of Lot 1111, Tract 1000, 3700 Coldwater Canyon Avenue, North Hollywood, California, dated October 29, 1984.

Reports by The J. Byer Group, Inc.:

Preliminary Findings, Harvard-Westlake School, Gymnasium Renovation and Gymnasium Addition, 3700 Coldwater Canyon Avenue, North Hollywood, California, dated October 5, 1998;

Geologic and Soils Engineering Exploration, Proposed Parking Lot Extension and Gymnasium Addition, Portion of Lot 1111, Tract 1000, 3700 Coldwater Canyon Avenue, North Hollywood, California, dated October 16, 1998;

Geologic and Soils Engineering Update, Proposed Field House Addition, Harvard-Westlake School, Portion of Lot 1111, Tract 1000, Coldwater Canyon Avenue, North Hollywood, California, dated October 16, 1998;

Addendum Report, Proposed Gymnasium Addition, Portion of Lot 1111, Tract 1000, 3700 Coldwater Canyon Avenue, North Hollywood, California, dated October 23, 1998;

Proposed Export of Soils, Harvard-Westlake School, West Campus, Portion of Lot 1112, Tract 1000; Portion of Lot 135, Tract 6293, 3801 Coldwater Canyon Avenue, North Hollywood, California, dated January 18, 1999;

Addendum Report, Proposed Placement of New Fill over Existing Fill, Portion of Lot 1111, Tract 1000, 3700 Coldwater Canyon Avenue, North Hollywood, California, dated January 26, 1999.

The data contained in these reports was reviewed and considered as part of our work on this project.

PROPOSED DEVELOPMENT

Information concerning the proposed project was provided by Jim De Matte. The preliminary plan prepared by Musco Sports-Lighting, Inc., dated September 26, 2006, was a guide for the preparation of this report. It is proposed to install four light poles. The two light poles (F3 and F4) on the east side of the track will be 80 feet high. The two light poles (F1 and F2) on the west side of the track will be 60 feet high. Formal plans have not been prepared and await the conclusions and recommendations of this report.

SITE DESCRIPTION

The subject property consists of a graded hillside property located in Coldwater Canyon in the Studio City section of the City of Los Angeles, California. It is located on the east side of Coldwater Canyon Avenue, approximately ½ mile north of Ventura Boulevard. The site is developed with a synthetic turf sports-field, with metal bleachers to the west and concrete bleachers to the east. The surrounding area has been developed with scattered hillside residences.

Physical relief across the sports field is about four feet with slope gradients less than five percent. Vegetation along of the perimeters of the field consists of a moderately thick assemblage of domestic shrubs and trees. Surface drainage is by sheetflow runoff down the contours of the land to the northwest.

GROUNDWATER

Groundwater was encountered in Boring 1 at 28 feet below grade. Higher groundwater levels may be expected to the south and east of Boring 1. Seasonal fluctuations in groundwater levels may occur due to variations in climate, irrigation, and other factors not evident at the time of the exploration. Fluctuations in groundwater levels may also occur across the site. Rising groundwater can saturate earth materials, causing subsidence of the site.

EARTH MATERIALS

Fill

Fill, associated with previous site grading, underlies the project site to a maximum observed depth of four feet in Boring 1. The fill consists of silty sand, gravelly clay, clayey silt that is brown, medium brown, dark brown, gray, moist, dense, and contains some asphalt debris.

Alluvium

Natural alluvium underlies the site. The alluvium is 37½ feet thick in Boring 1 and is anticipated to thicken toward the north. The alluvium consists of clayey silt, sandy clay, clayey sand that is light brown, medium brown, dark brown, dark gray, moist, moderately firm to firm.

Bedrock

Bedrock underlying the site and encountered in the test pits consists of shale mapped as part of the Modelo Formation by H. W. Hoots in the United States Geological Survey Professional Paper 165, *Geology of the Eastern Part of the Santa Monica Mountains, Los Angeles County, California*, 1931. The bedrock is light gray, gray-brown, moderately hard, thinly bedded, and is diatomaceous.

GENERAL SEISMIC CONSIDERATIONS

The subject property and all of southern California are located in an active seismic region (CBC Seismic Zone IV). Moderate to strong earthquakes can occur on numerous local faults. The United States Geological Survey, California Geological Survey, private consultants, and universities have been studying earthquakes in southern California for several decades. Early studies were directed toward earthquake prediction and estimation of the effects of strong ground shaking. Studies indicate that earthquake prediction is not practical and not sufficiently accurate to benefit the general public. Governmental agencies are shifting their focus to earthquake resistant structures as opposed to prediction. The purpose of the code seismic design parameters is to prevent collapse during strong ground shaking. Cosmetic damage should be expected.

Within the past 35 years, southern California and vicinity have experienced an increase in seismic activity beginning with the San Fernando earthquake in 1971. In 1987, a moderate earthquake struck the Whittier area and was located on a previously unknown fault. Ground shaking from this event caused substantial damage to the City of Whittier, and surrounding cities.

The January 17, 1994, Northridge Earthquake was initiated along a previously unrecognized fault below the San Fernando Valley. The energy released by the earthquake propagated to the southeast, northwest, and northeast in the form of shear and compression waves, which caused the strong ground shaking in portions of the San Fernando Valley, Simi Valley, City of Santa Clarita, and City of Santa Monica.

Southern California faults are classified as "active" or "potentially active." Faults from past geologic periods of mountain building that do not display evidence of recent offset, are considered "potentially active." Faults that have historically produced earthquakes or show evidence of movement within the past 11,000 years are known as "active faults." No known active faults cross the subject property.

According to the ICBO Publication *Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada,* February 1998, the nearest known potentially active fault is the Hollywood Fault, located approximately two kilometers to the south. From a Building Code (Chapter 16) standpoint, the Hollywood Fault is classified as a Type "B" fault. The following table lists the applicable seismic coefficients for the project:

BUILDING CODE SEISMIC COEFFICIENTS	
Earth Materials	Alluvium
Soil Profile Type	S_D
Seismic Coefficient (C_a)	$0.44N_a$
Seismic Coefficient (C_v)	$0.64N_v$
Near-Source Factor (N_a)	1.3
Near-Source Factor (N_v)	1.6

The principal seismic hazard to the subject property and proposed project is strong ground shaking from earthquakes produced by local faults. Modern, well-constructed buildings are designed to resist ground shaking through the use of shear panels and reinforcement. Additional precautions may be taken to protect personal property and reduce the chance of injury, including strapping water heaters and securing furniture. It is likely that the subject property will be shaken by future earthquakes produced in southern California. However, secondary effects such as surface rupture, lurching, liquefaction, consolidation, and landsliding should not occur at the subject property.

Ground Motion

The Seismic Hazard Zone Report for the Van Nuys 7.5 Minute Quadrangle, Seismic Hazard Zone Report 01 dated 1997, updated 2001, contains ground motion values assigned by the CGS for this area of Los Angeles. The Design Basis Earthquake (10 percent exceedance in 50 years) for the study area has a peak ground acceleration (PGA) of 0.52g (Figure 3.3, Probabilistic PGA). The de-aggregated predominant earthquake magnitude (M_w) is 6.4 (Figure 3.4, Predominate Earthquake). These ground motions could be expected at the site during the design lifespan of the structure and have been adopted for the liquefaction study of the site.

Liquefaction

The California Geological Survey has mapped the site within an area where historic occurrence of liquefaction or geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacement such that mitigation as defined in Public Resources Code Section 2963(c) would be required. The location of the subject property in relation to these mapped zones of required investigation is shown on the enclosed*Seismic Hazard Zones, Van Nuys Quadrangle, Official Map,* released February 1, 1998. According to *Special Publication 117, Guidelines for Evaluating and Mitigating Seismic Hazard in California,*adopted March 3, 1997, by the State Mining and Geology Board, the proposed sports-field lighting poles are not considered to be a "project" and is therefore exempt from a liquefaction study.

CONCLUSIONS AND RECOMMENDATIONS

General Findings

The conclusions and recommendations of this exploration are based upon one boring, four test pits, research of available records, consultation, years of experience observing similar properties in similar settings and review of the development plans. It is the finding of The J. Byer Group, Inc. that construction of the proposed project is feasible from a geologic and soils engineering standpoint provided the advice and recommendations contained in this report are included in the plans and are implemented during construction.

The recommended bearing material for the light poles is the alluvium. Friction pile foundations may be used.

Excavation Characteristics

The borings did encounter hard bedrock at a depth of 41.5 feet below grade in Boring 1. Excavation difficulty is a function of the degree of weathering and amount of fracturing within the bedrock. The bedrock generally becomes harder and more difficult to excavate with increasing depth. Hard cemented layers are also known to occur at random locations and depths and may be encountered during foundation excavation. Should a hard cemented layer be encountered, coring may be necessary.

Should groundwater be encountered in the pile excavations, it should be pumped out or the water may be placed by pumping concrete from the bottom with a hose. The tip of the hose shall be kept at least five feet below the concrete surface during pumping. When concrete is placed below water, the mix should be adjusted to achieve at least 1,000 psi more than the required strength. Should the pile shafts experience some caving, casing should be available.

FOUNDATION DESIGN

General Conditions

The following foundation recommendations are minimum requirements. The structural engineer may require footings that are deeper, wider, or larger in diameter, depending on the final loads.

Deepened Foundations - Friction Piles

Drilled, cast-in-place concrete friction piles may be used to support the proposed light poles and resist lateral forces. Piles should be a minimum of 18 inches in diameter. Piles in compression may be designed for a skin friction of 500 pounds per square foot for the portion of the piles into alluvium. Piles in tension may be designed for a skin friction of 250 pounds per square foot for that portion of piles into alluvium. The skin friction values are for the total of dead and frequently applied live loads and may be increased by one third for short duration loading, which includes the effects of wind or seismic forces.

Resistance to lateral loading may be provided by passive earth pressure with the alluvium. Passive earth pressure may be computed as an equivalent fluid having a capacity of 250 pounds per cubic foot. The maximum allowable earth pressure is 3,000 pounds per square foot. For design of isolated piles the allowable passive earth pressure may be increased by 100 percent. Piles spaced more than 2½ pile diameters on center may be considered isolated.

Foundation Settlement

Settlement of the foundation system is expected to occur on initial application of loading. A settlement of ¼ to ½ inch may be anticipated. Differential settlement should not exceed ¼ inch.

DRAINAGE

Control of site drainage is important for the performance of the proposed project. Surface drainage should be collected and transferred to the street or approved location in non-erosive drainage devices. Drainage should not be allowed to pond on the pad or against any foundation. Drainage control devices require periodic cleaning, testing and maintenance to remain effective.

PLAN REVIEW

Formal plans ready for submittal to the Building Department should be reviewed by The J. Byer Group. Any change in scope of the project may require additional work.

SITE OBSERVATIONS DURING CONSTRUCTION

The Building Department requires that the geotechnical company provide site observations during construction. The observations include foundation excavations. All fill that is placed should be tested for compaction and approved by the soils engineer prior to use for support of engineered structures.

Please advise The J. Byer Group, Inc. at least 24 hours prior to any required site visit. The agency approved plans and permits should be at the job site and available to our representative. The project consultant will perform the observation and post a notice at the job site of his visit and findings. This notice should be given to the agency inspector.

FINAL INSPECTION

Many projects are required by the agency to have final soils engineering reports upon completion of the grading.

CONSTRUCTION SITE MAINTENANCE

It is the responsibility of the contractor to maintain a safe construction site. When excavations exist on a site, the area should be fenced and warning signs posted. All pile excavations must be properly covered and secured. Soil generated by foundation and subgrade excavations should be either removed from the site or properly placed as a certified compacted fill. Workers should not be allowed to enter any unshored trench excavations over five feet deep.

GENERAL CONDITIONS

This report and the exploration are subject to the following NOTICE. Please read the NOTICE carefully, it limits our liability.

NOTICE

In the event of any changes in the design or location of any structure, as outlined in this report, the conclusions and recommendations contained herein may not be considered valid unless the changes are reviewed by us and the conclusions and recommendations are modified or reaffirmed after such review.

The subsurface conditions, excavation characteristics, and geologic structure described herein have been projected from excavations on the site as indicated and should in no way be construed to reflect any variations that may occur between these excavations or that may result from changes in subsurface conditions.

Fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, irrigation, and other factors not evident at the time of the measurements reported herein. Fluctuations also may occur across the site. High groundwater levels can be extremely hazardous. Saturation of earth materials can cause subsidence or slippage of the site.

If conditions encountered during construction appear to differ from those disclosed herein, notify us immediately so we may consider the need for modifications. Compliance with the design concepts, specifications, and recommendations requires geotechnical engineering review during the course of construction.

THE EXPLORATION WAS PERFORMED ONLY ON A PORTION OF THE SITE, AND CANNOT BE CONSIDERED AS INDICATIVE OF THE PORTIONS OF THE SITE NOT EXPLORED.

This report is issued and made for the sole use and benefit of the client, is not transferable and is as of the exploration date. Any liability in connection herewith shall not exceed the fee for the exploration. No warranty, expressed or implied, is made or intended in connection with the above exploration or by the furnishing of this report or by any other oral or written statement.

THIS REPORT WAS PREPARED ON THE BASIS OF THE PRELIMINARY DEVELOPMENT PLAN FURNISHED. FINAL PLANS SHOULD BE REVIEWED BY THIS OFFICE AS ADDITIONAL GEOTECHNICAL WORK MAY BE REQUIRED.

The J. Byer Group appreciates the opportunity to provide our service on this project. Any questions concerning the data or interpretation of this report should be directed to the undersigned.

Respectfully submitted,
THE J. BYER GROUP, INC.

Peter Kilbury
E. G. 2401

Robert I. Zweigler
G. E. 2120

REGISTERED PROFESSIONAL ENGINEER
ROBERT I. ZWEIGLER
No. 2120
Exp. 06-30-08
GEOTECHNICAL
STATE OF CALIFORNIA

PK:RIZ:flh
S:\FINAL\REPORTS\17866_Harvard_Westlake_Geotechnical_Report.wpd

Enc: Appendix I - Laboratory Testing
Shear Test Diagram
Topographic Map
Regional Geologic Map
Seismic Hazard Zones Map
Seismic Hazard Evaluation Report 08
Figure 3.3 - Alluvium Conditions
Figure 3.4 - Predominant Earthquake
Log of Test Pits
Log of Boring 1 (2 Pages)
Site Plan

xc: (7) Addressee

The J. Byer Group, Inc.
1461 East Chevy Chase Drive • Suite 200 • Glendale, California 91206 • (818) 549-9959 • Fax (818) 543-3747
"Trust the Name You Know"

APPENDIX I

LABORATORY TESTING

Undisturbed and bulk samples of the fill, alluvium, and bedrock were obtained from the borings and transported to the laboratory for testing and analysis. The samples were obtained by driving a ring lined barrel sampler conforming to ASTM D 3550-01 with successive drops of the Kelly bar. Experience has shown that sampling causes some disturbance of the sample, however the test results remain within a reasonable range. The samples were retained in brass rings of 2.50 inches outside diameter and 1.00 inches in height. The samples were stored in close fitting, waterproof containers for transportation to the laboratory.

Moisture-Density

The dry density of the samples was determined using the procedures outlined in ASTM D 2937-04. The moisture content of the samples was determined using the procedures outlined in ASTM D 2216-05. The results are shown on the Log of Borings.

Shear-Tests

Shear tests were performed on samples of alluvium using the procedures outlined in ASTM D 3080-04 and a strain controlled, direct shear machine manufactured by Soil Test, Inc. The rate of deformation was 0.025 inches per minute. The samples were tested in an artificially saturated condition. Following the shear test, the moisture content of the samples was determined to verify saturation. The results are plotted on the "Shear Test Diagram."

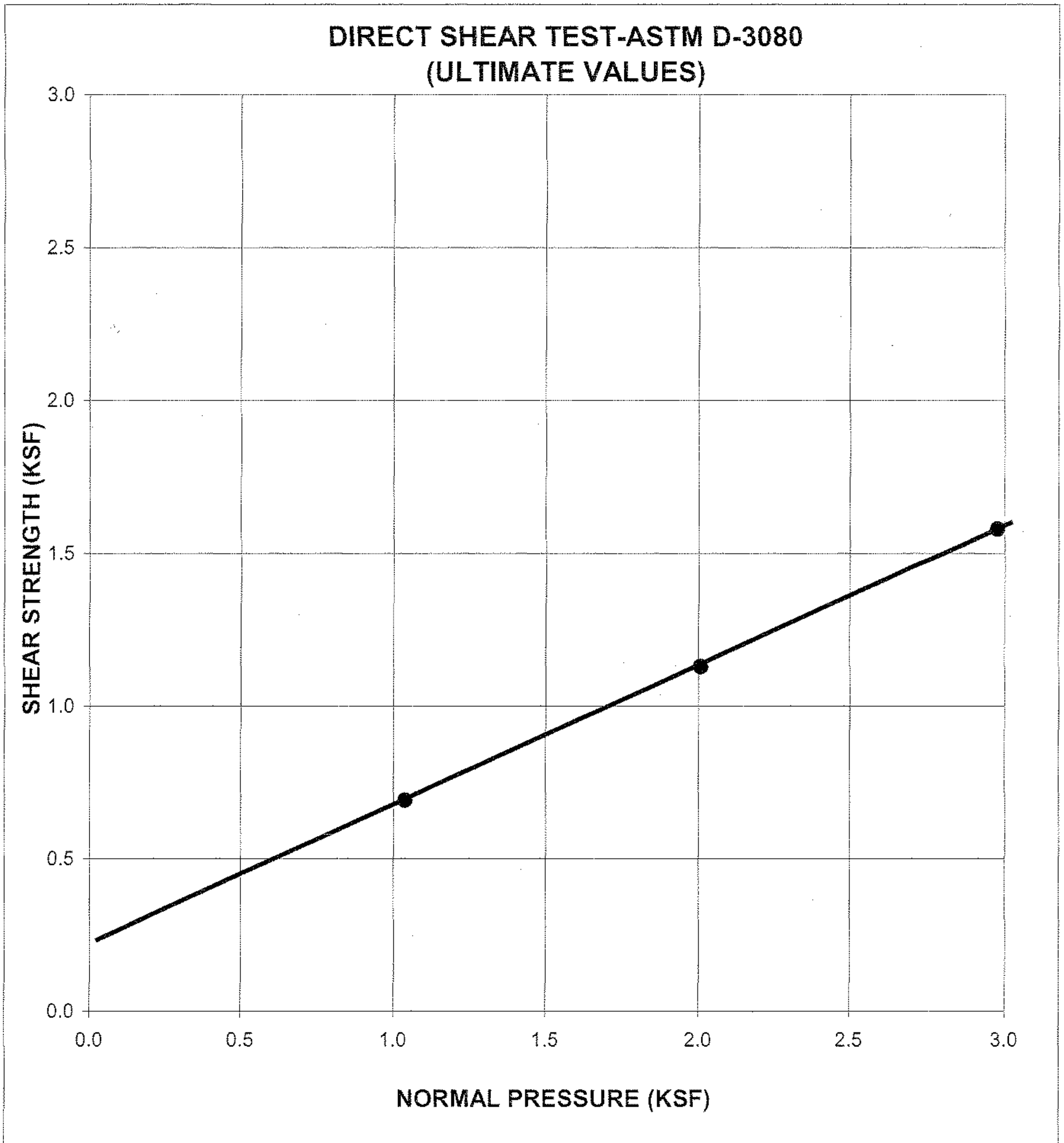
SHEAR DIAGRAM #1

THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
1461 E. CHEVY CHASE DRIVE, SUITE 200 GLENDALE, CA 91206
(818) 549-9959 Tel (818) 543-3747 FAX

JB: 17866-B CONSULT: PK
CLIENT: HARVARD-WESTLAKE

EARTH MATERIAL: ALLUVIUM (B1-15')

Phi Angle =	24 degrees	Moisture Content	25.3%
Cohesion =	222 psf	Dry Density (pcf)	98.9
		Percent Saturation	99.8%



OCTOBER 9, 2006

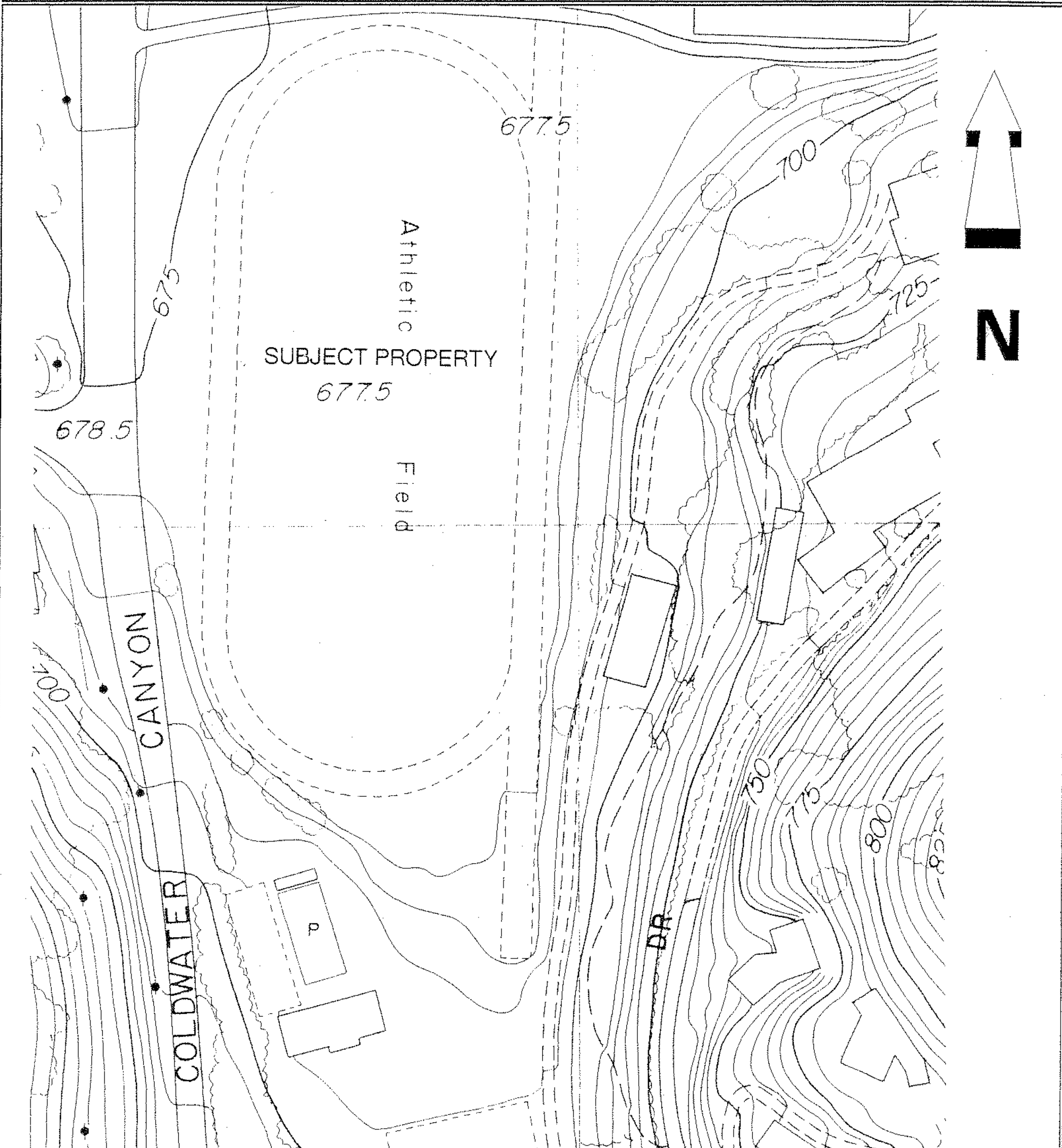
THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
512 E. WILSON AVENUE SUITE 201, GLENDALE, CA 91206
818•549•9959 Tel 818•543•3747 Fax

TOPOGRAPHIC MAP

JB 17866-b HARVARD-WESTLAKE

CONSULTANT: JWB SCALE: 1"=100'

REFERENCE: SANTA MONICA MOUNTAINS TOPOGRAPHIC MAPS, SHEET #70 (JANUARY 1960)



OCTOBER 9, 2006

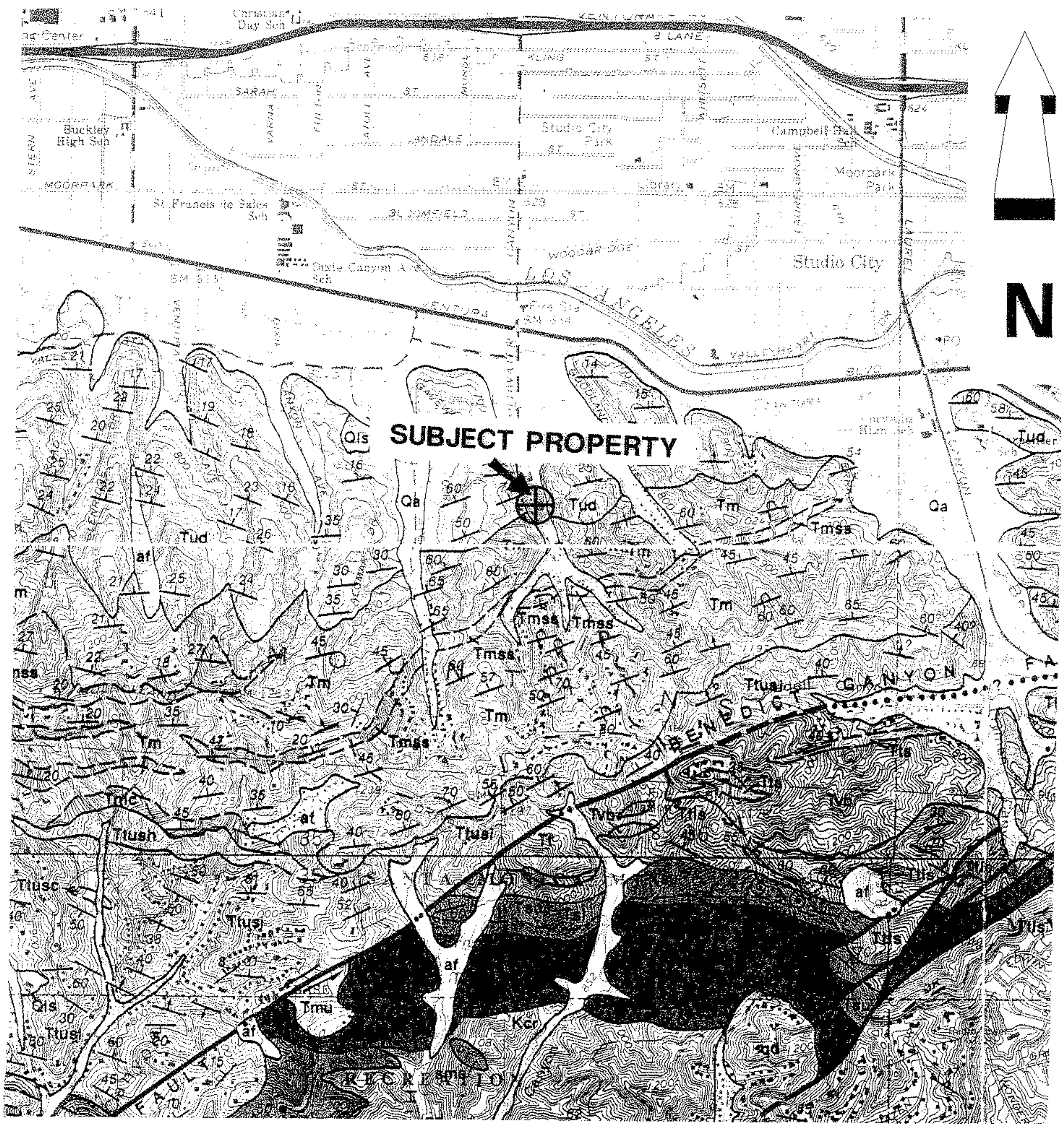
THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
512 E. WILSON AVENUE SUITE 201, GLENDALE, CA 91206
818•549•9959 Tel 818•543•3747 Fax

REGIONAL GEOLOGIC MAP

JB 17866-B HARVARD-WESTLAKE

CONSULTANT: JWB SCALE: 1"=2000'

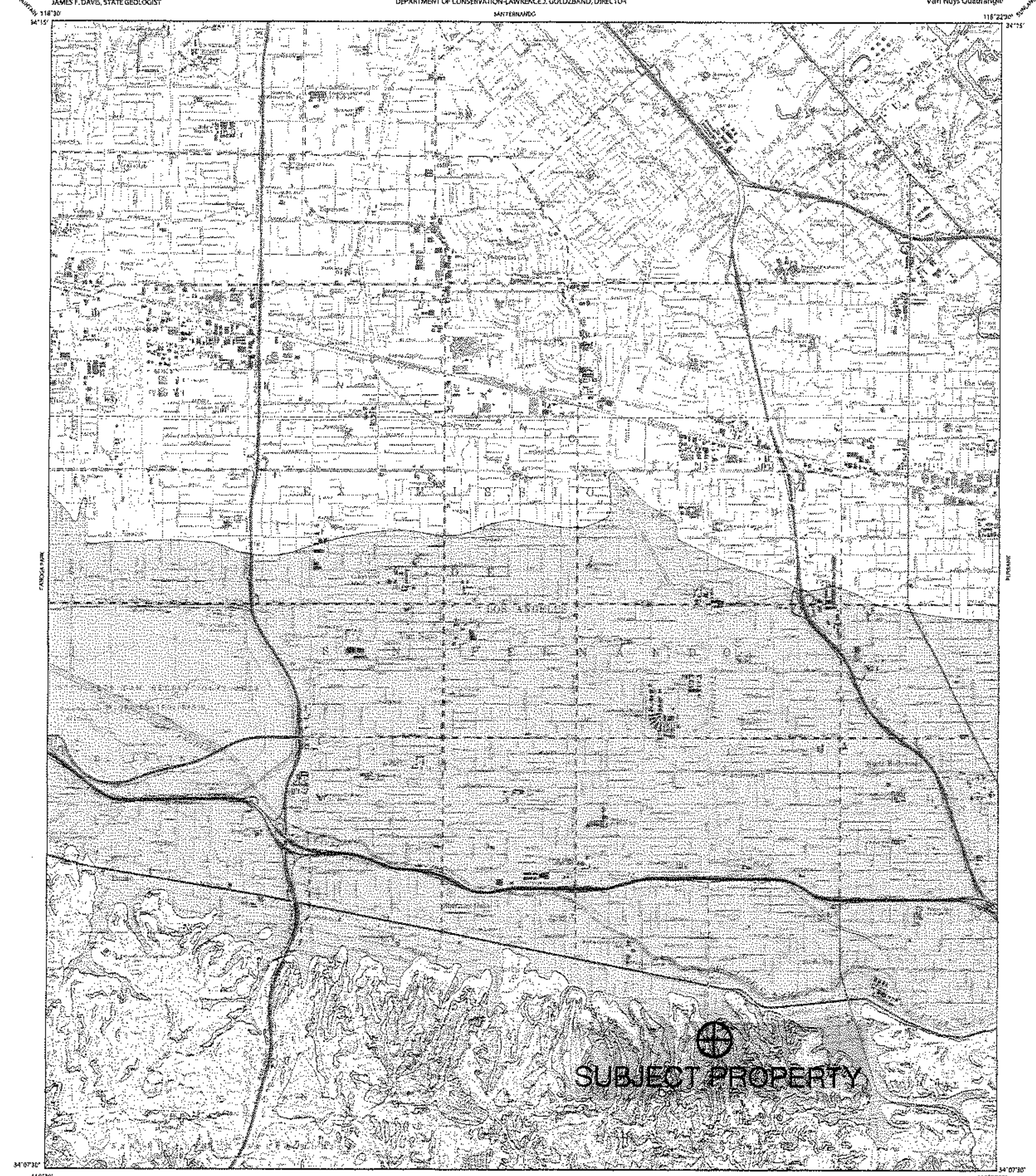
REFERENCE: GEOLOGIC MAPS OF THE BEVERLY HILLS AND VAN NUYS (SOUTH ½) QUADRANGLE BY T.W. DIBBLEE, 1991.



DIVISION OF MINES AND GEOLOGY
JAMES F. DAVIS, STATE GEOLOGIST

STATE OF CALIFORNIA-PETE WILSON, GOVERNOR
THE RESOURCES AGENCY-DOUGLAS P. WHEELER, SECRETARY
DEPARTMENT OF CONSERVATION-LAWRENCE J. GOLDZBAND, DIRECTOR

SEISMIC HAZARDS ZONES MAP
Van Nuys Quadrangle



Base Map prepared by U.S. Geological Survey, 1956, photorevised 1972

SCALE 1:24,000

PURPOSE OF MAP

This map will assist cities and counties in fulfilling their responsibilities for protecting the public safety from the effects of earthquake-triggered ground failure as required by the Seismic Hazards Mapping Act (Public Resources Code Sections 2690-2699.6).

For information regarding the scope and recommended methods to be used in conducting the required site investigations, see DMG Special Publication 117, Guidelines for Evaluating and Mitigating Seismic Hazards in California.

For a general description of the Seismic Hazards Mapping Program, the Seismic Hazards Mapping Act and regulations, and related information, please refer to the draft User's Guide (see http://www.consrv.ca.gov/dmg/shezp/userguid.html).

Production of this map was funded by the Federal Emergency Management Agency's Hazard Mitigation Program and the Department of Conservation in cooperation with the Governor's Office of Emergency Services.

IMPORTANT - PLEASE NOTE

1) This map may not show all areas that have the potential for liquefaction, landsliding, strong earthquake ground shaking or other earthquake and geologic hazards. Also, a single earthquake capable of causing liquefaction or triggering landslide failure will not uniformly affect the entire area zoned.

2) Liquefaction zones may also contain areas susceptible to the effects of earthquake-induced landslides. This situation typically exists at or near the toe of existing landslides, downslope from rockfall or debris flow source areas, or adjacent to steep stream banks.

3) This map does not show Alquist-Priolo earthquake fault zones, if any, that may exist in this area. Please refer to the latest official map of earthquake fault zones for disclosures and other actions that are required by the Alquist-Priolo Earthquake Fault Zoning Act. For more information on this subject and and index to available maps, see DMG Special Publication 42.

4) Landslide zones on this map were determined, in part, by adapting methods first developed by the U.S. Geological Survey (USGS). A new generation of landslide hazard maps being prepared by the USGS (Jibson and Harp, in preparation) uses an experimental approach designed to explore new methods to assess earthquake-induced landslide hazards. Although aspects of this new methodology may be incorporated in future seismic hazard zone maps, the experimental USGS maps should not be used as substitutes for these official earthquake-induced landslide zone maps.

5) U.S. Geological Survey base map standards provide that 90 percent of cultural features be located within 40 feet (horizontal accuracy) at the scale of this map. The identification and location of liquefaction and earthquake-induced landslide zones are based on available data. However, the quality of data used is varied. The zone boundaries depicted have been drawn as accurately as possible at this scale.

6) Information on this map is not sufficient to serve as a substitute for the geologic and geotechnical site investigations required under Chapters 7.5 and 7.8 of Division 2 of the Public Resources Code.

7) DISCLAIMER: The State of California and the Department of Conservation make no representations or warranties regarding the accuracy of the data from which these maps were derived. Neither the State nor the Department shall be liable under any circumstances for any direct, indirect, special, incidental or consequential damages with respect to any claim by any user or any third party on account of or arising from the use of this map.

STATE OF CALIFORNIA

SEISMIC HAZARD ZONES

Delineated in compliance with Chapter 7.8, Division 2 of the California Public Resources Code (Seismic Hazards Mapping Act)

VAN NUYS QUADRANGLE

OFFICIAL MAP

Released: February 1, 1998

STATE GEOLOGIST

MAP EXPLANATION

Zones of Required Investigation:

Liquefaction
Areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslides
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

DATA AND METHODOLOGY USED TO DEVELOP THIS MAP ARE PRESENTED IN THE FOLLOWING:

Seismic Hazard Evaluation of the Van Nuys 7.5-minute quadrangle, Los Angeles County, California: California Department of Conservation Division of Mines and Geology Open-File Report 97-15.

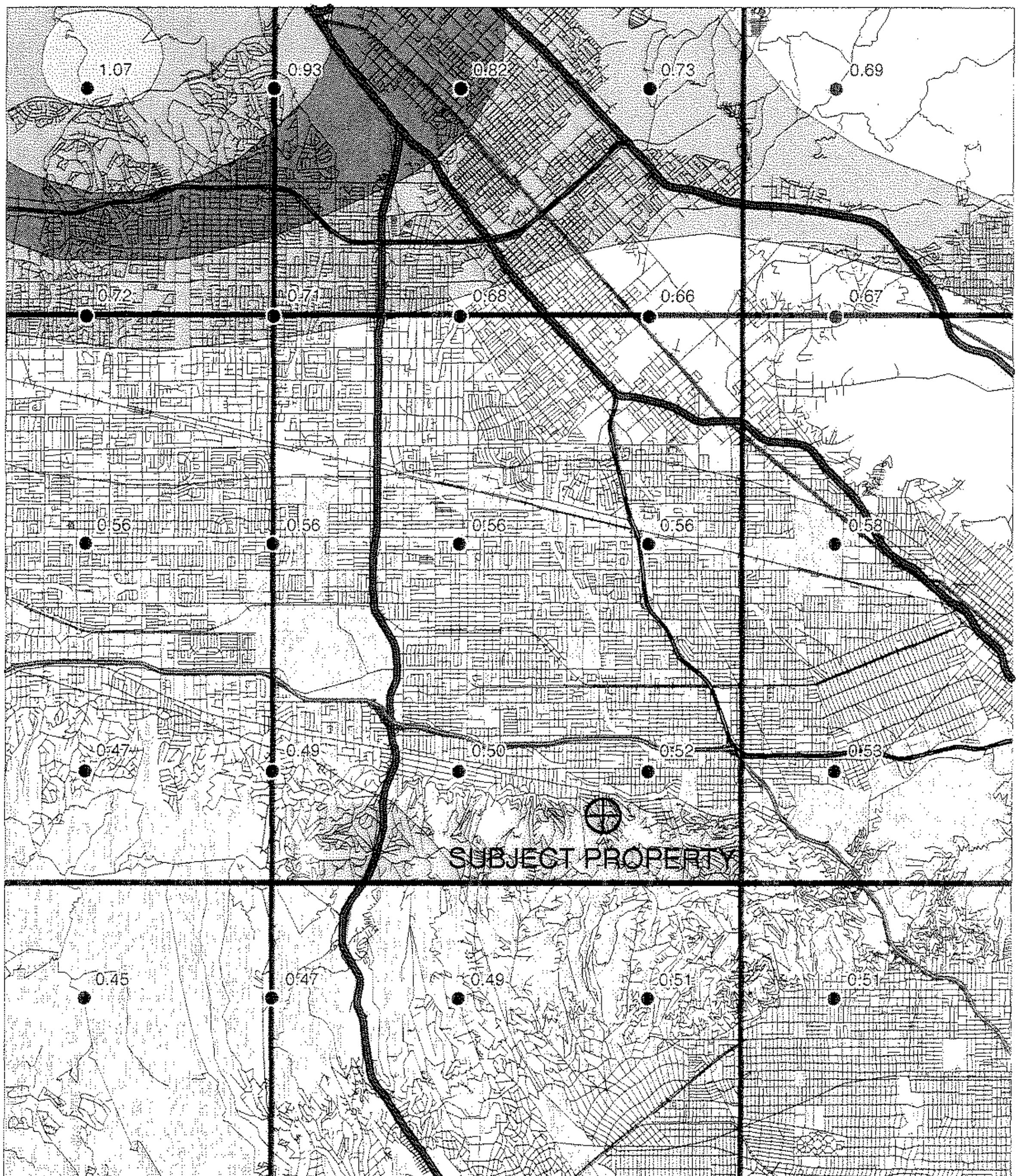
For additional information on seismic hazards in this map area, the rationale used for zoning, and additional references consulted, refer to DMG's World Wide Web site (http://www.consrv.ca.gov/dmg/).

VAN NUYS 7.5 MINUTE QUADRANGLE AND PORTIONS OF ADJACENT QUADRANGLES

10% EXCEEDANCE IN 50 YEARS PEAK GROUND ACCELERATION (g)

1998

ALLUVIUM CONDITIONS



Base map modified from MapInfo Street Works ©1998 MapInfo Corporation

0 2.5 5
Kilometers

Department of Conservation
California Geological Survey

Figure 3.3

VAN NUYS 7.5 MINUTE QUADRANGLE AND PORTIONS OF ADJACENT QUADRANGLES

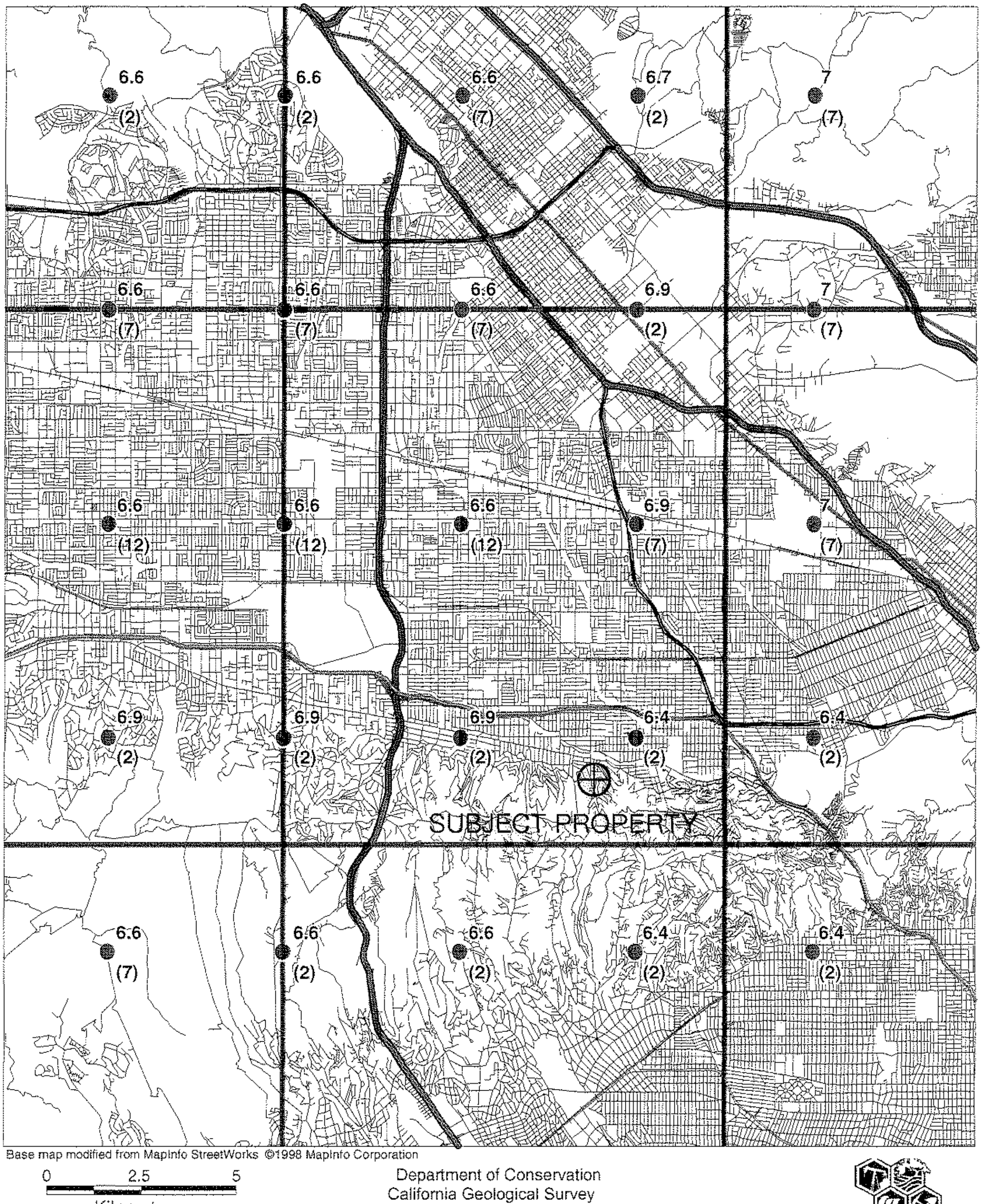
10% EXCEEDANCE IN 50 YEARS PEAK GROUND ACCELERATION

1998

PREDOMINANT EARTHQUAKE

Magnitude (Mw)

(Distance (km))



Base map modified from MapInfo StreetWorks ©1998 MapInfo Corporation

Department of Conservation
California Geological Survey

Figure 3.4

THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
1461 E. CHEVY CHASE DRIVE, SUITE 200, GLENDALE, CA 91206
818•549•9959 Tel 818•543•3747 Fax

LOG OF TEST PITS

JB: 17866-B
CLIENT: HARVARD-WESTLAKE SCHOOL
GEOLOGIST: PK
DATE LOGGED: 9/22/06
REPORT DATE: 10/9/06

TEST PIT #1 Surface Conditions: Dirt Elevation: 680 Feet

DEPTH INTERVAL (feet)	EARTH MATERIAL	LITHOLOGIC DESCRIPTION
0 - 1½	**FILL:**	Gravelly Clay, mottled gray and black, moist, dense, asphalt, brick debris
1½ - 4	**ALLUVIUM:**	Silty Clay, dark gray, moist, firm
4 - 6½		medium to dark gray, less moist

End at 6½ Feet; No Water; No Caving; Fill to 1½ Feet.

TEST PIT #2 Surface Conditions: Dirt Elevation: 682 Feet

DEPTH INTERVAL (feet)	EARTH MATERIAL	LITHOLOGIC DESCRIPTION
0 - 2	**FILL:**	Sandy Clay, mottled gray/black, moist, dense
2 - 5	**ALLUVIUM:**	Clayey Silt, dark brown, moist, dense
5 - 6½		medium brown

End at 6½ Feet; No Water; No Caving; Fill to 2 Feet.

TEST PIT #3 Surface Conditions: Grass Elevation: 678 Feet

DEPTH INTERVAL (feet)	EARTH MATERIAL	LITHOLOGIC DESCRIPTION
0 - 1½	**FILL:**	Silty Sand, brown, moist, dense
1½ - 6½	**ALLUVIUM:**	Silty Clay, medium brown, moist, firm

End at 6½ Feet; No Water; No Caving; Fill to 1½ Feet.

NOTE: The stratification depths shown on the Log of Test Pits are approximate and are based upon visual classification of samples and cuttings. The actual depths may vary. Variations between test pits may also occur.

JB No: 17866-B

Log of Boring: 1

Client: HARVARD-WESTLAKE SCHOOL

Logged By: PK

Site Location: 3700 Coldwater Canyon Avenue, Studio City

The J. Byer Group, Inc.
1461 E. Chevy Chase Dr., Ste 200
Glendale, CA. 91206
(818) 549-9959

SUBSURFACE PROFILE				SAMPLE							
Elevation	Depth	Description	Symbol	USCS	Type	Blow Count	Moisture Content (%)	Dry Density	Saturation %	Remarks
		FILL: Silty Sand, brown, moist, dense								
678.0	1									
677.0	2									
676.0	3	*ALLUVIUM:* Sandy Clay, medium brown, moist, dense, rock chips to ¼ inch								
675.0	4									
674.0	5			---	R	50	11.1	102.0	47.3	Rock in Bit
673.0	6									
672.0	7									
671.0	8	Clayey Silt, medium brown, moist, moderately firm, slightly porous								
670.0	9									
669.0	10			---	R	11	20.3	90.8	65.7	
668.0	11									
667.0	12									
666.0	13									
665.0	14	Sandy Clay, medium brown, moist, firm, rock fragments to 1 inch								
664.0	15			---	R	27	232.3	98.9	87.9	
663.0	16									
662.0	17									
661.0	18									
660.0	19									
659.0	20	Clayey Silt, medium brown, moist, dense, shale chips to ¼ inch		---	R	18	19.7	98.9	77.6	
658.0	21									
657.0	22									
656.0	23									
655.0	24									
654.0	25			---	R	17	23.9	98.3	92.8	
653.0	26									

Surface: Grass | Size: 8 Inch Diameter | Elevation: 679 Feet

Drill Method: Hollow-Stem Auger Drill Rig | Sheet: 1 of 2 | Drill Date: September 22, 2006

JB No: 17866-B

Log of Boring: 1

Client: HARVARD-WESTLAKE SCHOOL

Logged By: PK

Site Location: 3700 Coldwater Canyon Avenue, Studio City

The J. Byer Group, Inc.
1461 E. Chevy Chase Dr., Ste 200
Glendale, CA. 91206
(818) 549-9959

SUBSURFACE PROFILE				SAMPLE							
Elevation	Depth	Description	Symbol	USCS	Type	Blow Count	Moisture Content (%)	Dry Density	Saturation %	Remarks
652.0	27									
651.0	28	water encountered								
650.0	29	Clayey Sand, light brown, saturated, medium dense								
649.0	30			----	R	11	24.9	94.7	88.4	
648.0	31									
647.0	32									
646.0	33									
645.0	34									
644.0	35	Clayey Sand, light brown, saturated, medium dense, rock fragments to 1 inch		----	R	16	29.4	92.3	98.6	
643.0	36									
642.0	37									
641.0	38	Silty Clay, light brown, saturated, firm, slightly oxidized								
640.0	39									
639.0	40			----	R	20	48.5	71.3	97.3	
638.0	41									
637.0	42	*BEDROCK:* Shale, light gray, moderately hard, very weathered								
636.0	43									
635.0	44									
634.0	45			----	R	31	42.3	67.1	76.6	
633.0	46									
632.0	47	Diatomaceous Shale, gray-brown, moderately hard to hard, bedded, dips approximately 30 degrees								
631.0	48									
630.0	49									
629.0	50	End at 50 Feet; Water at 28 Feet; Fill to 4 Feet.		----	R	50 11"	47.8	70.1	93.2	
628.0	51									
627.0	52									

Surface: Grass

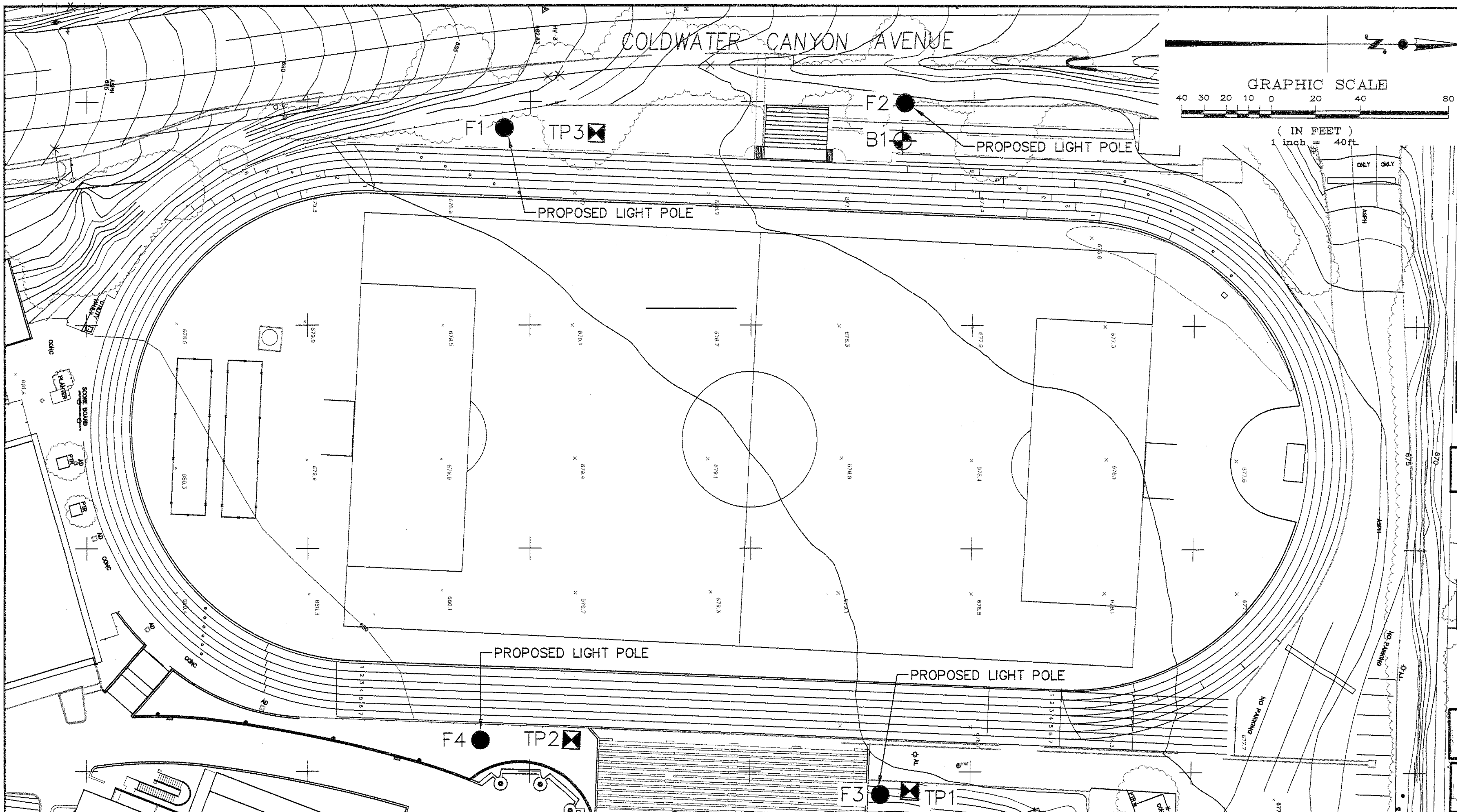
Drill Method: Hollow-Stem Auger Drill Rig

Size: 8 Inch Diameter

Sheet: 2 of 2

Elevation: 679 Feet

Drill Date: September 22, 2006



LEGEND

TP3 LOCATION AND NUMBER OF HAND DUG TEST PIT

B1 LOCATION AND NUMBER OF HOLLOW STEM AUGER BORING

OCTOBER 9, 2006

REFERENCE: LAND SURVEY PROVIDED BY DIACOBELLIS & ASSOCIATES, INC. DATED: 1/23/2001.

THE J. BYER GROUP, INC.
A GEOTECHNICAL CONSULTING FIRM
1461 E. Chevy Chase Dr. Suite 200, Glendale, CA 91206
(818) 549-9959 Tel (818) 543-3747 Fax

SITE PLAN

JB: 17866-B HARVARD WESTLAKE

CONSULTANT: PK

SCALE: 1" = 40'

CITY OF LOS ANGELES
CALIFORNIA

ANTONIO R. VILLARAIGOSA
MAYOR

DEPARTMENT OF
BUILDING AND SAFETY
201 NORTH FIGUEROA STREET
LOS ANGELES, CA 90012

ANDREW A. ADELMAN, P.E.
GENERAL MANAGER

RAYMOND CHAN
EXECUTIVE OFFICER

GEOLOGY/SOIL REPORT APPROVAL LETTER

January 23, 2007

Log # 56969
SOILS/GEOLOGY FILE - 2

LADBS approval letter log #56969 1/23/07

Harvard Westlake
3700 Coldwater Canyon Avenue
Studio City, Ca 91604

TRACT: 1000
LOT: 1111
LOCATION: 3700 N. Coldwater Canyon Avenue

CURRENT REFERENCE REPORT/LETTER(S)	REPORT NO.	DATE(S) OF DOCUMENT	PREPARED BY
Geology/Soil Report	JB 17866-B	10/09/2006	The J. Byer Group

The above referenced report concerning the proposed installation of four light poles up to 80 feet high to be located across the tract field has been reviewed by the Grading Division of the Department of Building and Safety. According to the report, the site is relatively flat. It is proposed to support the light poles on friction pile foundation system.

The report is acceptable, provided the following conditions are complied with during site development:

(Note: Numbers in parenthesis ( ) refer to applicable sections of the 2002 City of LA Building Code. P/BC numbers refer the applicable Information Bulletin. Information Bulletins can be accessed on the internet at LADBS.ORG.)

1. The geologist and soils engineer shall review and approve the detailed plans prior to issuance of any permits. This approval shall be by signature on the plans which clearly indicates that the geologist and soils engineer have reviewed the plans prepared by the design engineer and that the plans include the recommendations contained in their reports.

2. All recommendations of the report which are in addition to or more restrictive than the conditions contained herein shall be incorporated into the plans.

3. A copy of the subject and appropriate referenced reports and this approval letter shall be attached to the District Office and field set of plans. Submit one copy of the above reports to the Building Department Plan Checker prior to issuance of the permit. (7006.1)

4. Existing uncertified fill shall not be used for support of footings, concrete slabs or new fill. (7011.3 & 1806.1)

5. The proposed poles shall be supported on footings embedded into competent alluvium soil, as recommended.

6. All loose foundation excavation material shall be removed. Slopes disturbed by construction activities shall be restored. (7005.3)

7. The applicant is advised that the approval of this report does not waive the requirements for excavations contained in the State Construction Safety Orders enforced by the State Division of Industrial Safety. (3301.1)

8. The LABC Soil Type underlying the site is S_D. The minimum horizontal distance to known seismic sources shall be in accordance with the "Maps of Known Active Fault Near Source Zones" published by ICBO. (1636A)

9. If groundwater is encountered during excavation for piles, the water shall be pumped out, as specified on page 9 of the above current referenced report.

10. When water over 3 inches in depth is present in drilled pile holes, a concrete mix with a strength of 1000 p.s.i. over the design p.s.i. shall be tremied from the bottom up; an admixture that reduces the problem of segregation of paste/aggregates and dilution of paste shall be included.

11. Prior to the pouring of concrete, a representative of the consulting soils engineer shall inspect and approve the footing excavations. He shall post a notice on the job site for the LADBS Building Inspector and the Contractor stating that the work so inspected meets the conditions of the report, but that no concrete shall be poured until the City Building Inspector has also inspected and approved the footing excavations. A written certification to this effect shall be filed with the Grading Division of the Department upon completion of the work. (108.9 & 7008.2)

Negisti Girmay

NEGISTI GIRMAY
Engineering Geologist Associate II

Dana Prevost

DANA PREVOST
Engineering Geologist III

Log #56969
(213) 482-0480

cc: The J. Byer Group
Electro Construction, Corp.
VN District Office

PRELIMINARY
GEOTECHNICAL INVESTIGATION
PROPOSED PARKING STRUCTURE
HARVARD-WESTLAKE SCHOOL
3700 COLDWATER CANYON AVENUE
NORTH HOLLYWOOD, CALIFORNIA

Prepared for:
Innovative Design Group
17848 Sky Park Circle, Suite D
Irvine, California 92614

Prepared by:
Geotechnical Professionals Inc.
5736 Corporate Avenue
Cypress, California 90630
(714) 220-2211

Project No. 2270.I

July 27, 2010

TABLE OF CONTENTS

LIST OF FIGURES

APPENDIX A

APPENDIX B

1.0 INTRODUCTION

1.1 GENERAL

This report presents the results of a geotechnical investigation performed by Geotechnical Professionals Inc. (GPI) for the proposed parking structure to be located on Coldwater Canyon Avenue at the Harvard-Westlake School in North Hollywood, California. The geographical site location is shown on the Site Location Map, Figure 1.

This report is not intended as a stand-alone document for submittal to the City as a final design document. As discussed in the report, additional information is needed to finalize the detailed design.

1.2 PROJECT DESCRIPTION

Based on a site plan prepared by Innovative Design Group, the proposed development will consist of a new parking structure located within an undeveloped parcel across Coldwater Canyon Avenue from the school's athletic facilities. The parking structure will encroach into an ascending slope adjacent to Coldwater Canyon Avenue. The proposed site configuration is shown on the Site Plan, Figure 2.

The parking structure will be four-levels (3 suspended decks) covering a footprint of approximately 82,047 square feet (sf). We have assumed maximum column loads of up to 700 kips.

The proposed cut adjacent to the parking structure will be supported on three sides with a retaining system independent of the parking structure. The walls are planned to be constructed using top down methods, probably soil nails. The walls will range in height from approximately 10 to 60 feet with a total length on the order of 800 feet.

Based on preliminary grading information provided, the finished floor of the parking structure is planned to range from approximately 5 to 60 feet below the existing site grades. The finished floor of the parking structure is planned to be approximate 5 feet above the grade of Coldwater Canyon Avenue.

Our recommendations are based upon the above structural and grading information. We should be notified if the actual loads and/or grades change during the project design to either confirm or modify our recommendations. Also, when the project grading and foundation plans become available, we should be provided with copies for review and comment.

1.3 PURPOSE OF INVESTIGATION

The primary purpose of this investigation and report is to provide an evaluation of the existing geotechnical and seismic conditions at the site, as they relate to the design and construction of the proposed construction. More specifically, this investigation was aimed at providing geotechnical recommendations for earthwork, and design of foundations and pavements.

It should be noted that detailed grading and soil nail wall plans will need to be reviewed by GPI prior to confirmation of final design parameters.

2.0 SCOPE OF WORK

Our scope of work for this investigation consisted of field exploration, laboratory testing, engineering analysis, and the preparation of this report.

Our field exploration consisted of ten exploratory borings. The field locations and designations of the subsurface explorations are shown on the Site Plan, Figure 2. The exploratory borings were drilled using truck-mounted, bucket auger equipment to depths ranging from 21 to 71 feet below existing site grades. All borings were downhole logged by a certified engineering geologist. Details of the drilling and Logs of Borings are presented in Appendix A.

Laboratory tests were performed on selected representative soil samples as an aid in soil classification and to evaluate the engineering properties of the soils. The geotechnical laboratory testing program included determinations of moisture content and dry density, grain size distribution, shear strength, compressibility, maximum density/optimum moisture, expansion index, and soil corrosivity. Laboratory testing procedures and results are summarized in Appendix B.

Soil corrosivity testing was performed by Schiff Associates under subcontract to GPI. Their test results are presented at the end of Appendix B.

Geologic evaluations were performed to assess geologic conditions at the site. Engineering evaluations were performed to provide earthwork criteria, foundation and slab design parameters, preliminary pavement sections, and assessments of seismic hazards. The results of our evaluations are presented in the remainder of the report.

3.0 SITE CONDITIONS

3.1 SURFACE CONDITIONS

The proposed area to be developed is located within sloped lots extending upwards from Coldwater Canyon Avenue. The lots contain two residential homes on graded building pads, a larger graded area, driveways, and vacant sloped land. A retaining wall with a height of up to approximately 8 feet runs along a portion of the driveway to the upper vacant building pad. The site is heavily vegetated outside the graded lots with grasses, chaparral, and trees.

The site is bounded on the north by the undeveloped slopes, on the east by Coldwater Canyon Avenue, on the south and west by slopes with residences at the top.

The east facing natural slope extending upward from Coldwater Canyon Avenue has a height of greater than 200 feet. The north facing natural slope has a height of approximately 100 feet to the residence near the top. In general, the slopes have an inclination of steeper than 2:1 (horizontal:vertical). In between these slopes, there exist drainage valleys or fills within former drainage valleys. The topography at the site is shown in Figure 2.

3.2 SUBSURFACE SOILS

Our field investigation disclosed a subsurface profile consisting of undocumented fills underlain by soils and/or bedrock. Fills of less than 5 feet were encountered in our explorations though deeper fills are anticipated at the site. Detailed descriptions of the subsurface conditions encountered are shown on the Logs of Borings in Appendix A. A brief summary is provided below.

The natural soils encountered within the site consist primarily of silty clay, sandy silt, silt, and clayey silt. These soils were encountered within the areas of the current or former drainage valleys. The thickness of native soils in our explorations extended to depths of up to 23 feet below existing grade. These materials range from dry to wet and generally exhibit low strength and high compressibility characteristics.

Bedrock consisting of diatomaceous siltstone was encountered under the undocumented fill and natural soils extending to the depth of the borings. These materials are very moist to wet. These materials generally exhibit moderate to high strength and low to moderate compressibility characteristics.

Expansion Index testing of the siltstone within the parking structure footprint indicated the materials are moderately expansive. Atterberg limits testing of the siltstone indicates a high expansion potential.

3.3 SITE GEOLOGIC CONDITIONS

The project site is located in the Santa Monica Mountains on the west canyon wall of Coldwater Canyon, one of many north-flowing canyons that drain toward the San Fernando Valley. The area is within moderate to steep hillside terrain on the north flank of the east-west trending Santa Monica Mountains.

As shown on Figure 3, a Regional Geologic Map (Reference 1), the site and surrounding area are underlain by sedimentary bedrock of an unnamed shale (Modelo Formation of previous authors) that is typically diatomaceous. The geologic structure of the area is relatively simple, with bedding striking nearly east-west and dipping steeply (60 to 70 degrees) to the north. The geologic map by Dibblee (Reference 1) indicates that a contact between highly diatomaceous shale to the north and thin-bedded siltstone to the south is a depositional sedimentary contact. An AEG Geologic Map (Reference 2) indicates that the contact is a fault contact. Since no shearing or other evidence of faulting was observed in the borings, it is our opinion that the contact is depositional.

Our subsurface investigation consisted of ten large diameter borings that were downhole logged by a registered geologist. The locations of the borings, as well as the geologic data collected, are indicated on the attached Site Plan, Figure 2.

Our geologic investigation generally confirmed the published geology as shown by Dibblee. Bedding generally strikes nearly east-west and dips steeply to the north, except in the extreme southerly portion of the site, where bedding generally steepens, overturns, and dips to the south, as found in Borings B-9 and B-10. No evidence of faulting, such as shearing, was observed in the borings. The geologic map by Dibblee (Reference 1) shows several areas of overturned bedding in areas to the immediate south and east of the site. The bedding reversal is most likely due to simple overturning of steeply dipping bedding.

In general, bedding is favorably oriented with respect to proposed cuts at the toes of east and south facing existing natural slopes. Along a portion of the north facing slope on the south side of the proposed parking structure, steeply dipping bedding will be day-lighted by the proposed cut for the parking structure wall.

The AEG Geologic Map (Reference 2) also indicates a questioned landslide encompassing the ridgeline on the southern portion of the property. Borings B-1, B-2, B-9 and B-10 were drilled specifically to determine whether or not a landslide exists in the area. No evidence of landsliding was found.

Bedrock underlying the site is overlain by clayey, native residual soils and colluvium on the natural hillsides, and fine-grained alluvium, virtually indistinguishable from the colluvial soils, in the east flowing drainage in the southern portion of the site. The maximum thickness of alluvium observed is approximately 23 feet in Boring B-7.

Fill deposits, placed during a previous grading operation of unknown purpose, are present within two east flowing drainages, as shown on the Site Plan, Figure 2. The fill deposits are undocumented and have an estimated maximum thickness of approximately 20 feet. The fills will be removed by the planned cuts for the parking structure.

The interpreted geologic conditions expected to be encountered in the slope areas are indicated on the attached Geologic Cross Sections, Figures 4 to 6.

3.4 GROUNDWATER AND CAVING

Groundwater was not encountered in our exploratory borings to depths of 71 feet below the existing ground surface. Perched groundwater may be encountered within excavations at the bottom of the drainage valleys. A historical depth to groundwater has been determined for the site to be greater than 40 feet below existing grades (Reference 3).

Caving was not observed within the large diameter borings.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 GENERAL

Based on the results of our investigation, it is our opinion that from a geotechnical viewpoint it is feasible to develop the site as proposed. The proposed parking structure can be supported on shallow foundations following remedial grading to mitigate the geotechnical constraints discussed below. The most significant geotechnical issues that will affect the design and construction of the proposed structures are as follows:

- The recommendations provided herein are based on very preliminary design concepts for the project. Until details of the proposed project are available, the recommendations provided herein are subject to revision.

- Due to moderate to high potential for expansion, we recommend that the upper 2 feet of the subgrade soils consisting of siltstone be removed and replaced with imported, non-expansive sandy soils.

- Undocumented fills and compressible soils not removed by the proposed cuts should be removed and replaced as properly compacted fill. At the southeast area of the parking structure, we anticipate deeper excavations to remove compressible alluvium/colluvium will extend to a depth of approximately 20 feet below the finished floor.

- The majority of footings will be supported on competent bedrock. At the southeast area of the parking structure, a limited number of footings will be supported on properly compacted fill. The fill material should be derived from on-site siltstone or suitable import soils. These footings will need to be designed with a reduced bearing capacity relative to footings supported on competent bedrock.

- In order to limit the total and differential settlement of footings, we recommend 2 feet of crushed aggregate base be placed underneath the spread footings with fill depths of 10 feet or greater from the building pad elevation.

- The anticipated locations of footings with reduced bearing capacity should be identified by the Geotechnical Engineer after a foundation plan has been developed and confirmed during pad grading. The anticipated locations of footings required to be underlain with crushed aggregate base should be identified by the Geotechnical Engineer during pad grading.

- As noted in the geologic assessment of the site, the bedding structure in the bedrock encountered in our explorations was noted to be favorably oriented with respect to proposed excavations for the majority of the proposed wall. As such, the stability of excavations extending into the bedrock material is not anticipated to be adversely affected by bedding. We recommend that our geologist be on-site during the excavation to confirm the actual subsurface conditions encountered.

- Steeply sloping bedding may be exposed in the cuts for the soil nail wall. Adverse effects for this condition will be mitigated by the soil nail wall.

- Alluvium/colluvium soils are anticipated to be exposed in the soil nail wall cuts along a portion of the walls for the parking structure. We recommend the soils nails along the wall areas as identified in Section 4.7 of the report utilize the design parameters for alluvium/colluvium.

- The on-site soils are severely corrosive to metals. This should be considered in the design of soil nails and other buried metal. Portland cement products in contact with the on-site soils should be designed for severe levels of soluble sulfate exposure for soil.

Our recommendations related to the geotechnical aspects of the development of the site are presented in the subsequent sections of this report.

4.2 SEISMIC CONSIDERATIONS

4.2.1 General

The site is located in a seismically active area of Southern California and is likely to be subjected to strong ground shaking due to earthquakes on nearby faults.

We assume the seismic design of the proposed development will be in accordance with the California Building Code, 2007 edition. For the 2007 CBC, a Soil Class C may be used. The seismic code values can be obtained directly from the tables in the building code using the above values and appropriate United States Geological Survey web site (Reference 4). The seismic design method should be determined by the Project Structural Engineer.

4.2.2 Strong Ground Motion Potential

Based on published information presented in Reference 5, the most significant fault in the proximity of the site is the Hollywood Fault, which is located approximately 6 kilometers from the site.

During the life of the project, the site will likely be subject to strong ground motions due to earthquakes on nearby faults. Based on our probabilistic ground motion analysis using FRISKSP (Reference 5), the site could be subjected to a peak ground acceleration of 0.56g. This acceleration has a 10 percent chance of being exceeded in 50 years. The ground accelerations are averages of those calculated using attenuation relationships given by Boore, et al (1997), Campbell and Bozorgnia (1997) and Sadigh, et al (1997). The structural design will need to incorporate measures to mitigate the effects of strong ground motion.

4.2.3 Ground Rupture

The site is not located within an Alquist-Priolo Earthquake Fault Zone and there are no known faults crossing or projecting toward the site. Therefore, ground rupture due to faulting is considered unlikely at this site.

4.2.4 Liquefaction

Liquefaction is a phenomenon in which saturated, cohesionless soils undergo a temporary loss of strength during severe ground shaking and acquire a degree of mobility sufficient to permit ground deformation. In extreme cases, the soil particles can become suspended in groundwater, resulting in the soil deposit becoming mobile and fluid-like. Liquefaction is generally considered to occur primarily in loose to medium dense deposits of saturated sandy soils. Thus, three conditions are required for liquefaction to occur: (1) a sandy soil of loose to medium density; (2) saturated conditions; and (3) rapid, large strain, cyclic loading, normally provided by earthquake motions.

The majority of the site is not located within an area identified by the State as having a potential for soil liquefaction. Within this area, soil liquefaction is not likely to occur at the project site because the majority of the soils encountered are sedimentary bedrock and groundwater is deep.

A small portion of the parking structure is located within an area mapped by the State of California as having a potential for soil liquefaction (Reference 3). Groundwater was not encountered to the depth of the bedrock at our exploration (Boring B-7) within the liquefaction zone. Any potentially liquefiable soils within the alluvium and colluvium under the foundations of parking structure will be removed during remedial grading.

4.2.5 Seismic Ground Subsidence

Seismic ground subsidence (not related to liquefaction), occurs when loose, granular (sandy) soils above the groundwater are densified during strong earthquake shaking. Significant subsidence during a strong earthquake is not expected to occur if the recommended earthwork is performed.

4.3 EARTHWORK

The earthwork anticipated at the project site will consist of clearing and grubbing, excavation of undocumented fills, excavation of compressible soils, excavation to pad grade, subgrade preparation, and the placement and compaction of fill.

4.3.1 Clearing and Grubbing

Prior to grading, the areas to be developed should be stripped of any vegetation and cleared of all debris, slabs, and pavements. All buried obstructions, such as footings, underground storage tanks, utilities and tree roots, should be removed.

All deleterious material generated during the clearing operation should be removed from the site. Inert demolition debris, such as concrete and asphalt, may be crushed for re-use in engineered fills in accordance with the criteria presented in the "Material for Fill" section of this report.

Although none were encountered, any cesspools or septic systems encountered during grading should be removed in their entirety. The resulting excavation should be backfilled as recommended in the "Subgrade Preparation" and "Placement and Compaction of Fill" sections of this report. As an alternative, cesspools can be backfilled with a lean sand-cement slurry. At the conclusion of the clearing operations, the representative of the geotechnical engineer should observe and accept the site prior to any further grading.

4.3.2 Excavations

Excavations at this site will include removals of undocumented fill soils, removals of compressible soils, cuts to finish grade, removals of siltstone under the concrete slab, footing excavations, and trenching for proposed utility lines.

Prior to placement of fills, or construction of floor slabs and foundation supported structures, undocumented fills, compressible soils, soils disturbed during demolition, and a portion of the relatively expansive siltstone occurring under the proposed parking structure should be removed and replaced as properly compacted fill. Compressible soils include alluvium, colluvium and residual soils.

Due to their moderate to high expansion potential, we recommend that the siltstone bedrock be excavated to at least 2 feet below proposed finish subgrade under the proposed parking structure and replaced with non expansive fill as described below.

At the southeast area of the parking structure, deeper excavations to remove the compressible alluvium and colluvium soils will be required. We anticipate these excavations to extend to a depth of approximately 20 feet below the finished floor along the eastern wall of the parking structure.

The actual depths of removals should be determined in the field during grading by the Geotechnical Engineer.

The base of the overexcavation for the structures should extend laterally at least 5 feet beyond the building line or perimeter foundations, or a minimum distance equal to the depth of overexcavation/compaction below finish grade (i.e., a 1:1 projection below the top outside edge of footings), whichever is greater. Building lines include all canopies or other foundation supported improvements associated with the parking structure. The corners of the areas to be overexcavated should be accurately staked in the field by the Project Surveyor.

Where not removed by the aforementioned excavations, existing utility trench backfill should be removed and replaced as properly compacted fill. This is especially important for deeper fills associated with existing sewers and storm drains. For planning purposes, removals over the utilities should extend to within 1-foot of the top of the pipe. For utilities, which are 5 feet or shallower, the removal should extend laterally 1-foot beyond both sides of the pipe. For deeper utilities, the removals should include a zone defined by a 1:1 projection upward (and away from the pipe) from each side of the pipe. The actual limits of removal will need to be confirmed in the field. We recommend that all known utilities be shown on the grading plan.

Temporary construction excavations may be made vertically without shoring to a depth of 5 feet below adjacent grade. For cuts up to 10 feet deep within the siltstone, the slopes should be properly shored or sloped back to at least ¾:1 or flatter. For cuts up to 20 feet deep within the native soils, the slopes should be properly shored or sloped back to at least 1½:1 or flatter. No surcharge loads should be permitted within a horizontal distance equal to the height of cut from the top of the excavation or 5 feet from the top of the slopes, whichever is greater, unless the cut is properly shored. Excavations that extend below an imaginary plane inclined at 45 degrees below the edge of any adjacent existing site facilities should be properly shored to maintain support of adjacent elements. All excavations and shoring systems should meet the minimum requirements given in the most current State of California Occupational Safety and Health Standards.

4.3.3 Subgrade Preparation

After the recommended removals are performed and prior to placing any fills, the exposed subgrade soils exhibiting near-optimum moisture conditions should be scarified to a depth of 8 inches, moisture-conditioned, and compacted to at least 90 percent of maximum dry density in accordance with ASTM D-1557.

In areas to receive pavements, the upper 12 inches below the pavement base should be scarified, moisture-conditioned, and compacted to dry densities equal to at least 90 percent (95 percent for granular soils) of maximum dry density (ASTM D-1557).

Subgrade processing should not be performed at the bottom of excavations if moist, undisturbed siltstone bedrock conditions are exposed, as determined by GPI in the field during grading. Where siltstone is exposed, care should be taken to prevent it from drying out during construction. Moisture conditioning should be performed on any subgrade soils allowed to dry. Disturbing and recompacting the materials will increase their potential for future expansion.

4.3.4 Material for Fill

The on-site soils are, in general, suitable for use as compacted fill provided they are dried back to near optimum moisture conditions. Beneath the influence of the foundations for the parking structure, the compacted fill should be derived from on-site siltstone or suitable import soils. The on-site siltstone, silts, or clay soils are not suitable for use as retaining wall backfill or under concrete slabs/pavements. We recommend that a minimum of 2 feet of imported, non-expansive, granular fill be used under the slabs for the parking structure. If heaving of exterior flatwork is not tolerable, the same zone of non-expansive materials should be placed under the flatwork.

Retaining wall backfill and select fill below flatwork and slabs should consist of imported granular (containing no more than 40 percent fines – portion passing the No. 200 sieve) and relatively non-expansive (Expansion Index of 20 or less) soils. Moisture conditioning (extensive drying) will be required prior to re-using some of the on-site soils to permit compaction to the recommended degree.

From a geotechnical engineering standpoint, asphalt concrete or portland cement concrete can be incorporated into fills placed outside the building areas provided that they are crushed to the consistency of aggregate base. Such material should not be placed within landscape areas. Provided it is acceptable to the reviewing governmental agencies and owner, crushed, inert demolition debris derived from the existing pavements, may be used in fills with the following processing requirements:

- If the inert debris is crushed to a well graded mixture with maximum particle size of 1½ inches, the crushed material may be used directly in the fill without further blending.
- Inert debris up to a maximum size of 6 inches may also be used in fills, provided it is thoroughly blended with imported sandy soils to form a well graded mixture.

Imported fill material should be predominately granular (contain no more than 40 percent fines - portion passing No. 200 sieve) and non-expansive (E.I. less than 20). The Geotechnical Engineer should be provided with a sample (at least 50 pounds) and notified of the location of any soils proposed for import at least 72 hours in prior to importing. Each proposed import source should be sampled, tested and accepted for use prior to delivery of the soils to the site. Soils imported prior to acceptance by the Geotechnical Engineer may be rejected if not suitable. Both imported and existing on-site soils to be used as fill should be free of debris and should not contain material larger than 6 inches in any dimension.

Both imported and existing on-site soils to be used as fill should be free of debris and should not contain material larger than 6 inches in any dimension.

4.3.5 Placement and Compaction of Fills

Granular (sands and gravels) fill soils should be placed in horizontal lifts, moisture-conditioned, and mechanically compacted to densities equal to at least 95 percent of the maximum dry density, determined in accordance with ASTM D1557. Fills comprised of clayey soils should be compacted to at least 90 percent. Crushed aggregate base beneath the footings to limit settlement should be compacted to at least 98 percent of the maximum dry density in accordance with ASTM D 1557.

The optimum lift thickness will depend on the compaction equipment used and can best be determined in the field. The following uncompacted lift thickness can be used as preliminary guidelines.

Plate compactors	4-6 inches
Small vibratory or static rollers (5-ton)	6-8 inches
Scrapers, heavy loaders, and large vibratory rollers	8-12 inches

The on-site soils include diatomaceous siltstone exhibiting high moisture contents. The grading contractor should anticipate these soils to be moisture sensitive and difficult to compact. The moisture content of the on-site soils is well above optimum, and will require drying. The moisture content of the fill materials should be at least 2 to 3 percent over optimum conditions at the time of compaction. Discing of soils to accelerate drying should be anticipated, if these materials will be used as fill.

The maximum lift thickness should not be greater than 12 inches and each lift should be thoroughly compacted and accepted prior to subsequent lifts.

During backfill of excavations, the fill should be properly benched into the construction slopes as it is placed in lifts.

4.3.6 Shrinkage and Subsidence

Shrinkage is the loss of soil volume caused by compaction of fills to a higher density than before grading. Subsidence is the settlement of in-place subgrade soils caused by loads generated by large earthmoving equipment. For earthwork volume estimating purposes, an average shrinkage value of 10 to 15 percent may be assumed for the surficial soils (upper 5 feet) and alluvium/colluvium soils within the drainage valleys. Subsidence is expected to be less than 0.1 feet. These values are estimates only and exclude losses due to removal of vegetation or debris. Actual shrinkage and subsidence will depend on the types of earthmoving equipment used and should be determined during grading.

4.3.7 Trench/Wall Backfill

Utility trench and wall backfill material should be mechanically compacted in lifts. The clayey soils and siltstone at the site should not be used as retaining wall backfill. Lift thickness should not exceed those values given in the "Compacted Fill" section of this report. Jetting or flooding of backfill materials should not be permitted. The Geotechnical Engineer should observe and test all trench and wall backfills as they are placed.

In backfill areas where mechanical compaction of soil backfill is impractical due to space constraints, sand-cement slurry may be substituted for compacted backfill. The slurry should contain one sack of cement per cubic yard and have a maximum slump of 5 inches. When set, such a mix typically has the consistency of compacted soil. We also recommend that slurry be used as bedding material for trenches containing multiple lines.

4.3.8 Observation and Testing

A representative of GPI should observe all excavations, subgrade preparation, and fill placement activities. Sufficient in-place field density tests should be performed during fill placement and in-place compaction to evaluate the overall compaction of the soils. Soils that do not meet minimum compaction requirements should be reworked and tested prior to placement of any additional fill.

4.4 SLOPES

Natural slopes of varying heights exist above the proposed parking structure and the proposed retaining wall system. The slopes to the south side of the site extend to heights on the order of 100 feet. The slopes to the west and north side of the site extend to heights on the order of 200 feet. The natural slopes above the proposed retaining wall system, as shown in our cross-sections (Figures 4 to 6), have inclinations, in general, of approximately 1.6:1 or flatter.

Preliminary gross stability analysis was performed for the existing slopes using the computer program STABL5M and the Modified Bishop Method of analysis. The surficial stability of the slopes was determined using the method of infinite slope. The soil parameters used were based on direct shear testing of undisturbed and deformed samples.

Existing slopes with favorable bedrock bedding inclined at 1.5:1 were determined to exhibit the minimum generally accepted factors of safety for gross and surficial stability under static and pseudostatic conditions (1.5 and 1.1, respectively).

Existing slopes consisting of colluvium and alluvium at the surface do not have the generally accepted factors of safety for surficial stability under static and pseudostatic conditions (1.5 and 1.1, respectively). This is consistent with observations of creep of the colluvium on the natural soils.

The existing slopes will be modified as part of the construction of the soil nail walls. Details regarding the length of the soil nails will be completed by the wall designer. In addition to internal stability, the wall designer should evaluate the global stability of the slopes as the length of the nails determines the stability of the slopes. The modified slopes should be evaluated as part of the review of the wall and grading plans.

Construction within the slopes should be observed by our geologist to confirm the subsurface conditions, especially with respect to adverse bedding, are consistent with our findings.

Fill slopes may be constructed at inclinations of 2:1 (horizontal:vertical) or flatter.

4.5 FOUNDATIONS

4.5.1 Foundation Type

The proposed structure may be supported on conventional isolated and/or continuous shallow footings, provided the subsurface soils are prepared in accordance with the recommendations given in this report. All footings for the parking structure should be supported on competent bedrock and/or properly compacted fill. Footing bottoms should be moistened immediately prior to placement of concrete.

4.5.2 Allowable Bearing Pressures

Based on the shear strength and elastic settlement characteristics of the natural and recompacted on-site soils, static allowable net bearing pressures of up to 6,000 pounds per square foot (psf) may be used for both continuous footings and isolated column footings for the proposed parking structure. These bearing pressures are for dead-load-plus-live-load, any may be increased one-third for short-term, transient, wind and seismic loading. The actual bearing pressure used may be less than the value presented above and can be based on economics and structural loads to determine the minimum width for footings as discussed below. The maximum edge pressures induced by eccentric loading or overturning moments should not be allowed to exceed these recommended values.

4.5.3 Minimum Footing Widths and Embedments

The following minimum footing widths and embedments are recommended for the corresponding allowable bearing pressure.

MINIMUM FOOTING WIDTHS AND EMBEDMENTS

STATIC BEARING PRESSURE (psf)	MINIMUM FOOTING WIDTH (inches)	MINIMUM FOOTING* EMBEDMENT (inches)
Footings Supported on Competent Bedrock		
6,000	60	36
4,000	48	24
3,000	24	24
2,500	18	18
Footings Supported on Properly Compacted Fill		
5,000	60	36
3,000	48	24
2,000	24	24
1,500	18	18

* Refers to minimum depth below lowest adjacent grade at the time of foundation construction.

A minimum footing width of 18 inches should be used even if the actual bearing pressure is less than 1,500 psf.

To achieve a bearing pressure of 6,000 psf, deepening of footings locally to bedrock may be required.

The majority of footings will be supported on competent bedrock. The locations of footings anticipated to be supported on properly compacted fill should be identified by the Geotechnical Engineer after a foundation plan has been developed and should be confirmed during the grading of the building pad.

Footings adjacent to the descending slope along Coldwater Canyon Avenue should be deepened to allow for a lateral distance of at least one-half of the slope height, but not less than 10 feet, between the base of the footing and the face of the slope. We should be provided with the foundation and grading plans to review the footing conditions relative to the proposed adjacent grades prior to bidding the project.

4.5.4 Estimated Settlements

For the parking structure, total static settlement of the column footings (700 kips maximum column load) is expected to be less than 1.5 inches provided the footings are supported on competent bedrock or properly compacted fills.

In order to limit the total settlement to 1.5 inches, we recommend 2 feet of crushed aggregate base be placed underneath the spread footings with fill depths of 10 feet or greater from the building pad elevation. The crushed aggregate base beneath footings should extend beyond the edge of footings at least a distance equal to the thickness of the base. The crushed aggregate base should be placed as recommended in the "Placement and Compaction of Fills" section of this report.

The actual footings requiring to be underlain with crushed aggregate base to limit settlements should be determined in the field during grading by the Geotechnical Engineer.

Provided the above recommendations concerning the placement of crushed aggregate base under footings supported on deeper fills are incorporated into the project plans and placed during construction, the maximum differential settlements between similarly loaded adjacent footings or along a 60-foot span are expected to be less than ¾-inch.

The above estimates are based on the assumption that the recommended earthwork will be performed and that the footings will be sized in accordance with our recommendations.

4.5.5 Lateral Load Resistance

Soil resistance to lateral loads will be provided by a combination of frictional resistance between the bottom of footings and underlying soils and by passive soil pressures acting against the embedded sides of the footings. For frictional resistance, a coefficient of friction of 0.35 may be used for design. In addition, an allowable lateral bearing pressure equal to an equivalent fluid weight of 300 pounds per cubic foot may be used for footings. The allowable lateral bearing pressure values provided are based on the footings being poured tight against compacted fill or competent bedrock. The friction and lateral bearing values may be used in combination without reduction.

4.5.6 Foundation Concrete

Laboratory testing by Schiff Associates (Appendix B) on a samples provided by GPI indicates soluble sulfate content of 1,080 and 5,220 mg/kg (0.11 and 0.52 percent by weight). Foundation concrete should conform to the requirements outlined in ACI 318, Section 4.3, for severe levels of soluble sulfate exposure for soil.

4.5.7 Footing Excavation Observation

Prior to placement of steel and concrete, a representative of GPI should observe and approve all footing excavations.

4.6 BUILDING FLOOR SLABS

Slab-on-grade floors should be supported on a minimum of 24 inches of granular non-expansive (Expansion Index less than 20), compacted soils as discussed in the "Placement and Compaction of Fill" section. The on-site siltstone, silt, or clay should not be permitted within 24 inches of the concrete slab.

While not anticipated over the majority of the parking structure floor, a vapor/moisture retarder should be placed under any slabs that are to be covered with moisture-sensitive floor coverings (parquet, vinyl tile, etc.). Currently, common practice is to use 10-mil polyethylene as a vapor retarder placed either directly on the subgrade or over a thin layer of sand. Recently, other types of vapor retarders with much lower permanence and higher puncture resistance have become available and should be considered as an alternative. Polyolefin in 10-mil or 15-mil thickness is such a material and could be considered for this project. This material should be covered by a layer of clean sand (less than 5 percent by weight passing the No. 200 sieve) having a minimum thickness of 2 inches. The function of the sand layer is to protect the vapor retarder during construction and to aid in the uniform curing of the concrete. This layer should be nominally compacted using light equipment. The sand placed over the vapor barrier should be only slightly moist. If the sand gets wet (for example, as a result of rainfall) it must be allowed to dry prior to placing concrete.

It should be noted that the material used as a vapor retarder is only one of several factors affecting the prevention of moisture accumulation under floor coverings. Other factors include effective sealing of joints edges (particularly at pipe penetration) as well as excess moisture in the concrete. The manufacturer of floor coverings should be consulted for establishing acceptable criteria for the condition of floor surface prior to placing moisture-sensitive floor coverings.

For lateral resistance design, a coefficient of friction of 0.4 can be used for concrete in direct contact with sandy fill. For slabs constructed over a visqueen or polyolefin moisture barrier, a friction coefficient of 0.1 should be used.

4.7 RETAINING STRUCTURES

At the time this report was prepared, building basement walls were not planned for the project. The cut behind the parking structure is planned to be supported by an independent retaining system. The following recommendations are provided for soil nail walls, the planned retaining wall system outside of the parking structure, and conventional retaining walls for ramp walls and small site walls.

We should be provided with the design plans retaining systems prior to finalizing to confirm suitable geotechnical design parameters have been used.

4.7.1 Conventional Retaining Walls

Active earth pressures can be used for designing cantilevered walls up to 15 feet in height that can yield at least ½-inch laterally in 10 feet under the imposed loads. For cantilever walls with level backfill comprised of granular soils, the magnitude of active pressures are equivalent to the pressures imposed by a fluid weighing 35 pounds per cubic foot (pcf). For sloping backfill with a 2:1 inclination, the active pressure would be about 52 pcf.

For restrained walls that remain rigid enough to be essentially non-yielding, an at-rest lateral earth pressure should be used for design. For restrained walls with level backfill comprised of granular soils, the magnitude of at-rest pressure is equivalent to the pressure imposed by a fluid weighing 52 pounds per cubic foot (pcf).

Walls subject to surcharge loads should be designed for an additional uniform lateral pressure equal to one-third and one-half the anticipated surcharge pressure for unrestrained and restrained walls, respectively. We can provide more specific lateral earth pressures resulting from surcharge loads when further details on the surcharge load are available.

The recommended pressures are based on the assumption that the supported earth will be fully drained, preventing the build-up of hydro-static pressures. For traditional backfilled retaining walls, a drain consisting of perforated pipe surrounded by gravel and wrapped in filter fabric should be used. As a minimum, one cubic foot of rock should be used for each lineal foot of drain. The fabric (non-woven filter fabric, Mirafi 140N or equivalent) should be lapped at the top.

The Structural Engineer should specify the use of select, granular wall backfill on the plans for conventional retaining walls. Wall footings should be designed as discussed in the "Foundations" section.

4.7.2 Soil Nail Walls

We understand that soil nail walls will probably be used for retaining the cuts up to 60 feet outside of the parking structure. The soil nail walls consist of steel bar encased in grout constructed from the top down in increments and completed with a wire mesh and shotcrete surface.

We expect that a specialty contractor will be retained to develop a soil nail wall design and construction plan on a design-build basis. A soil nail wall should be designed using soil strengths that reflect the condition of the retained materials behind the wall. Based on our explorations, it appears that the wall will retain mainly siltstone materials and to a lesser extent alluvium/colluvium and existing fill. The actual conditions should be observed in the field during construction by a representative of GPI to confirm the actual conditions.

Provisions should be made by the soil nail design engineer to modify the nail lengths as needed during construction to accommodate changes in ground conditions. For design of the nails, we recommend the following design parameters:

Preliminary Soil Nail Design Parameters

Moist Unit Weight, pcf	Cohesion, psf	Phi angle, degrees
Siltstone Bedrock		
90	200	30
Alluvium/Colluvium/Existing Fill		
100	100	28

We anticipate alluvium/colluvium soils to be exposed in the soil nail wall cuts along a portion of the wall on the west side of the parking structure and along the diagonal wall facing the southeast. We anticipate the alluvium/colluvium will be exposed from the southwest corner of the parking structure for approximately 100 feet to the north. We anticipate the alluvium/colluvium will be exposed along the entire portion of the diagonal wall facing the southeast. We recommend the alluvium/colluvium be assumed to extend a depth of 20 feet from the top of the proposed wall in these areas.

The areas where alluvium/colluvium design parameters should be used in the wall design is shown on Figure 2. We recommend that our geologist be on-site during the excavation to confirm the actual subsurface conditions encountered during the excavations for the soil nail walls.

The soil nail wall should be designed for seismic conditions. We recommend a pseudostatic coefficient of one-half of the peak ground acceleration provided in the "Strong Ground Motion" section of this report be used in the design of the soil nail wall.

The design should include consideration of global stability of the cut as well as internal stability. The retaining wall designer should confirm the global stability of the cut by evaluating potential failures beyond the soil nails. **The nails should have sufficient length whereas potential failure surfaces extending beyond the soil nails and the toe**

of the planned wall have an adequate factor of safety for the global stability. The global stability should have a factor of safety of at least 1.5 and 1.1 for static and seismic conditions, respectively.

We should review the soil nail plans and analyses for global stability. During our review, we will only confirm soil strength design parameters.

For design of soil nail walls, a design bond stress between the soil nail grout and the surrounding soil is needed to perform internal wall stability calculations. An ultimate bond stress values of 12 psi in the siltstone and 10.0 psi in the colluvium/alluvium soils may be used, assuming that the average depth to the grouted portion of the soil nail is at least 20 feet below finish grades. The values may be increase if the average depth to the grouted portion of the soil nail is significantly deeper than 20 feet. These conditions can be evaluated at the time of the final design of the wall. Considering the large number of soil nails to be installed, we recommend installation and load testing of several pre-production test nails (in alluvium/colluvium and siltstone) in order to confirm/refine the bond stresses listed above. Details of soil nail testing are presented in the subsequent section of this report.

The upper 10 feet of the wall adjacent to streets or drives should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pound per square foot surcharge behind the wall due to normal street traffic. If traffic is kept at least 10 feet from the wall, the traffic surcharge may be neglected.

The soil nail contractor should evaluate the potential drilling conditions when planning the installation methods. Caving was not encountered during our explorations at the upper portion of the site in the area of the planned cut. However, some loose, dry materials may be encountered in the near-surface alluvium/colluvium and may be prone to local caving.

The soil nails should be designed for soils severely corrosive to metals. The grout in the soil nails should be designed for severe levels of soluble sulfate exposure for soil.

The permanent walls should be drained full-height using a suitable drainage composite. The drainage composite should be placed between the soil nails prior to applying the shotcrete surface to allow for perched groundwater seepage within the height of the cut to be collected and discharged without building up hydrostatic pressures behind the wall. We recommend that the continuous drainage panels be installed at the same spacing as the soil nails. Sufficient drainage should be provided to accommodate existing outlet drains from the backdrain of the slope stabilization fill.

We recommend performing a detailed survey of the improvements supported above the planned cut prior to and during the soil nail installation. The survey should include topographic data and a video account of the condition of the existing improvements, including cracks or signs of distress. During construction, the monitoring should consist of periodic surveying of the lateral and vertical locations of the top of the soil nail wall. We suggest weekly readings for the first four weeks after installation. After that time, the readings should be performed twice-monthly until the completion of the construction.

4.7.3 Soil Nail Testing

We recommend the contractor perform proof and verification testing on the soil nails. The following soil nail testing procedures are in general accordance with FHWA guidelines (Reference 6).

Proof tests should be performed on production nails at locations approved by the Geotechnical Engineer. We recommend at least 5 percent of the total nails in each row should be selected for proof testing. This should include at least 1 nail per row and 1 nail per distinct soil/rock unit for proof testing. We recommend pre-production verification tests should be performed on at least two sacrificial test nails in each different soil/rock unit and for each different drilling/grouting method.

The test nails should have both bonded and temporary unbonded lengths. Prior to testing only the bonded length of the test nail shall be grouted. The temporary unbonded length of the test nail should be at least 3 feet.

We recommend the verification tests be performed by incrementally loading the test nail to a maximum test load of 200 percent of the Design Test Load (DTL). The DTL is determined by multiplying the as-built bonded test length (feet) by the allowable pullout resistance of the nail (kips per foot of grouted nail length). After loading the nail to an alignment load (0.10DTL), the loads should be increased to 0.25DTL and subsequent load increments of 0.25DTL. At load increments below 1.5DTL, the load shall be held a sufficient time increment to obtain a stable reading. At 1.5DTL, the load shall be held for 60 minutes for a creep test. The nail movement during the creep test shall be measured and recorded at 1, 2, 3, 5, 6, 10, 20, 30, 40, 50, and 60 minutes. After the creep test, the nail shall be loaded to 1.75 DTL and 2.0 DTL for a sufficient time increment to obtain a stable reading.

For verification tests, the test nail may be considered acceptable when a total creep movement of less than 0.08 inch per log cycle of time between the 6 and 60 minute readings is measured during creep testing and the creep rate is linear or decreasing throughout the creep test load hold period.

We recommend the proof tests be performed by incrementally loading the test nail to a maximum test load of 150 percent of the DTL. After loading the nail to an alignment load (0.10DTL), the loads should be increased to 0.25DTL and subsequent load increments of 0.25DTL. At load increments below 1.5DTL, the load shall be held a sufficient time increment to obtain a stable reading. At 1.5DTL, the load shall be held for 60 minutes for a creep test. The nail movement during the creep test shall be measured and recorded at 1, 2, 3, 5, 6, and 10 minutes. If the nail movement between 1 minute and 10 minutes exceeds 0.04 inches, the maximum test load shall be maintained an additional 50 minutes and the movements shall be recorded at 20, 30, 50, and 60 minutes.

The test nails during proof testing may be considered acceptable when the following have been achieved:

- A total creep movement of less than 0.04 inch measured between the 1 and 10 minute readings or a total creep movement of less than 0.08 inch is measured between the 6 and 60 minute readings and the creep rate is linear or decreasing throughout the creep test load hold period.

- The total measured movement at the maximum test load exceeds 80 percent of the theoretical elastic elongation of the test nail unbonded length.

- A pullout failure does not occur at the maximum test load. Pullout failure is defined as the load at which attempts to further increase the test load simply result in continued pullout movement of the test nail. The pullout failure load shall be recorded as part of the test data.

Nails meeting the above proof-testing acceptance criteria may be incorporated as production nails after being completed by grouting the unbonded test length.

If a test nail does not meet the acceptance criterion, the Contractor should determine the cause of the problem. The Geotechnical Engineer may require the installation and testing of additional proof test nails to verify that adjacent previously installed production nails have sufficient load carrying capacity. The Contractor may be required to modify design or construction procedures. The modifications may include the installation of additional proof test nails, increasing the drillhole diameter, modifying the installation or grouting methods, reducing the production nail spacing, or installing longer production nails. Lengthening of the nails may be limited by the temporary construction easements or the permanent right-of-way.

Nail testing should be performed by the Contractor and observed by GPI. The Contractor should provide all necessary test equipment, including an independent fixed reference point (i.e., tripod) for placement of the digital or dial gauge for measuring nail deflections during testing. Prior to testing, the Contractor should supply current calibration records of the hydraulic jack and pressure gauge to be used for testing. Calibration records should be signed by a California registered professional engineer and be within 9 months prior of the start of testing.

We recommend that a representative of GPI observe the installation and testing of all soil nails to confirm that the recommendations provided in our report are applicable during construction.

4.8 CORROSIVITY

Resistivity testing of a sample of the on-site soils indicates that the on-site soils and bedrock are severely corrosive to metals. We do not practice corrosion protection engineering. If buried metal pipe is to be used, a corrosion engineer such as Schiff Associates should be consulted.

4.9 DRAINAGE

Positive surface gradients should be provided adjacent to all structures so as to direct surface water run-off and roof drainage away from foundations and slabs toward suitable discharge facilities. Long-term ponding of surface water should not be allowed on pavements or in planters adjacent to buildings.

4.10 EXTERIOR CONCRETE AND MASONRY FLATWORK

If heaving of exterior flatwork is not tolerable, diatomaceous siltstone, silt, or clay within 24-inches of the flatwork or concrete pavements adjacent to the parking structure should not be permitted and the exterior flatwork should be supported on non-expansive, compacted fill. Prior to placement of concrete, the subgrade should be prepared as recommended in "Subgrade Preparation" section.

4.11 PAVED AREAS

Although significant paved areas are not anticipated for the project, preliminary pavement sections are provided below based upon an assumed R-value of 20 and conventional Traffic Indices (TI's) typically used for commercial developments. The California Division of Highways Design Method was used for design of the recommended preliminary pavement sections. These recommendations are based on the assumption that the pavement subgrades will consist of existing near-surface soils. Final pavement design should be based on R-value testing performed near the conclusion of rough grading. The following pavement sections are recommended for planning purposes only.

PAVEMENT AREA	TRAFFIC INDEX	SECTION THICKNESS (inches)	
		ASPHALT CONCRETE	AGGREGATE BASE COURSE
Asphalt Concrete Auto Parking Stalls	4	3	5
Circulation Drives (no trucks)	5	3	8
Truck Driveways	6	3	11
Portland Cement Concrete Auto Parking Stalls	4	**Portland Cement Concrete** 7	----
Circulation Drives (no trucks)	5	7	----
Truck Driveways	6	7½	----

The concrete used for paving should have a modulus of rupture of at least 550 psi (equivalent to an approximate compressive strength of 3,700 psi at the time the pavement is subjected to traffic).

The pavement subgrade underlying the aggregate base should be properly prepared and compacted in accordance with the recommendations outlined under "Subgrade Preparation".

The pavement base course should be compacted to at least 95 percent of the maximum dry density (ASTM D 1557). Aggregate base should conform to the requirements of Section 26 of the California Department of Transportation Standard Specifications for Class II aggregate base (three-quarter inch maximum) or Section 200-2 of the Standard Specifications for Public Works Construction (Green Book) for untreated base materials (except processed miscellaneous base).

The above recommendations are based on the assumption that the upper 24-inches of expansive soils below concrete pavements have been removed and replaced with non-expansive material.

The above recommendations are based on the assumption that the base course and compacted subgrade will be properly drained. The design of paved areas should incorporate measures to prevent moisture build-up within the base course, which can otherwise lead to premature pavement failure. For example, curbing adjacent to landscaped areas should be deep enough to act as a barrier to infiltration of irrigation water into the adjacent base course.

4.12 GEOTECHNICAL OBSERVATION AND TESTING

We recommend that a representative of GPI observe all earthwork during construction to confirm that the recommendations provided in our report are applicable during construction. The earthwork activities include soil nail wall construction, grading, compaction of fills, subgrade preparation, pavement construction and foundation excavations. If conditions are different than expected, we should be afforded the opportunity to provide an alternate recommendation based on the actual conditions encountered.

5.0 LIMITATIONS

The report, exploration logs, and other materials resulting from GPI's efforts were prepared exclusively for use by Innovative Design Group and their consultants in designing the proposed development. The report is not intended to be suitable for reuse on extensions or modifications of the project or for use on any project other than the currently proposed development, as it may not contain sufficient or appropriate information for such uses. If this report or portions of this report are provided to contractors or included in specifications, it should be understood that they are provided for information only. This report cannot be utilized by another entity without the express written permission of GPI. This report is an instrument of our services and remains the property of GPI.

Soil deposits may vary in type, strength, and many other important properties between points of exploration due to non-uniformity of the geologic formations or to man-made cut and fill operations. While we cannot evaluate the consistency of the properties of materials in areas not explored, the conclusions drawn in this report are based on the assumption that the data obtained in the field and laboratory are reasonably representative of field conditions and are conducive to interpolation and extrapolation.

Furthermore, our recommendations were developed with the assumption that a proper level of field observation and construction review will be provided during grading, excavation, and foundation construction by GPI. If field conditions during construction appear to be different than is indicated in this report, we should be notified immediately so that we may assess the impact of such conditions on our recommendations. If construction phase services are performed by others they must accept full responsibility (as Project Geotechnical Engineer) for all geotechnical aspects of the project including this report.

Our investigation and evaluations were performed using generally accepted engineering approaches and principles available at this time and the degree of care and skill ordinarily exercised under similar circumstances by reputable Geotechnical Engineers practicing in this area. No other representation, either expressed or implied, is included or intended in our report.

Respectfully submitted,
Geotechnical Professionals Inc.

REGISTERED PROFESSIONAL ENGINEER DONALD A. CORDS No. GE 2529 Exp. 6-30-13 GEOTECHNICAL STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER DONALD A. CORDS No. GE 2529 Exp. 6-30-11 GEOTECHNICAL STATE OF CALIFORNIA

Donald A. Cords, G.E.
Associate

James E. Harris, G.E.
Principal

REGISTERED PROFESSIONAL ENGINEER JAMES E. HARRIS, IV No. GE 2100 Exp. 12-31-11 GEOTECHNICAL STATE OF CALIFORNIA

Thomas G. Hill, C.E.G.
Consulting Geologist
DAC/JEH/TGH:sph

REGISTERED GEOLOGIST THOMAS G. HILL No. 1100 CERTIFIED ENGINEERING GEOLOGIST STATE OF CALIFORNIA

Exp 9-30-10
Exp 9-30-14

REGISTERED PROFESSIONAL ENGINEER JAMES E. HARRIS, IV No. GE 2100 Exp. 12-31-13 GEOTECHNICAL STATE OF CALIFORNIA

2/13/13

REFERENCES

1. Dibblee, T.W., Jr., 1991, Geologic Map of the Beverly Hills and Van Nuys (south ½) Quadrangles. Dibblee Geological Foundations Map #DF-31; Scale, 1:24,000.

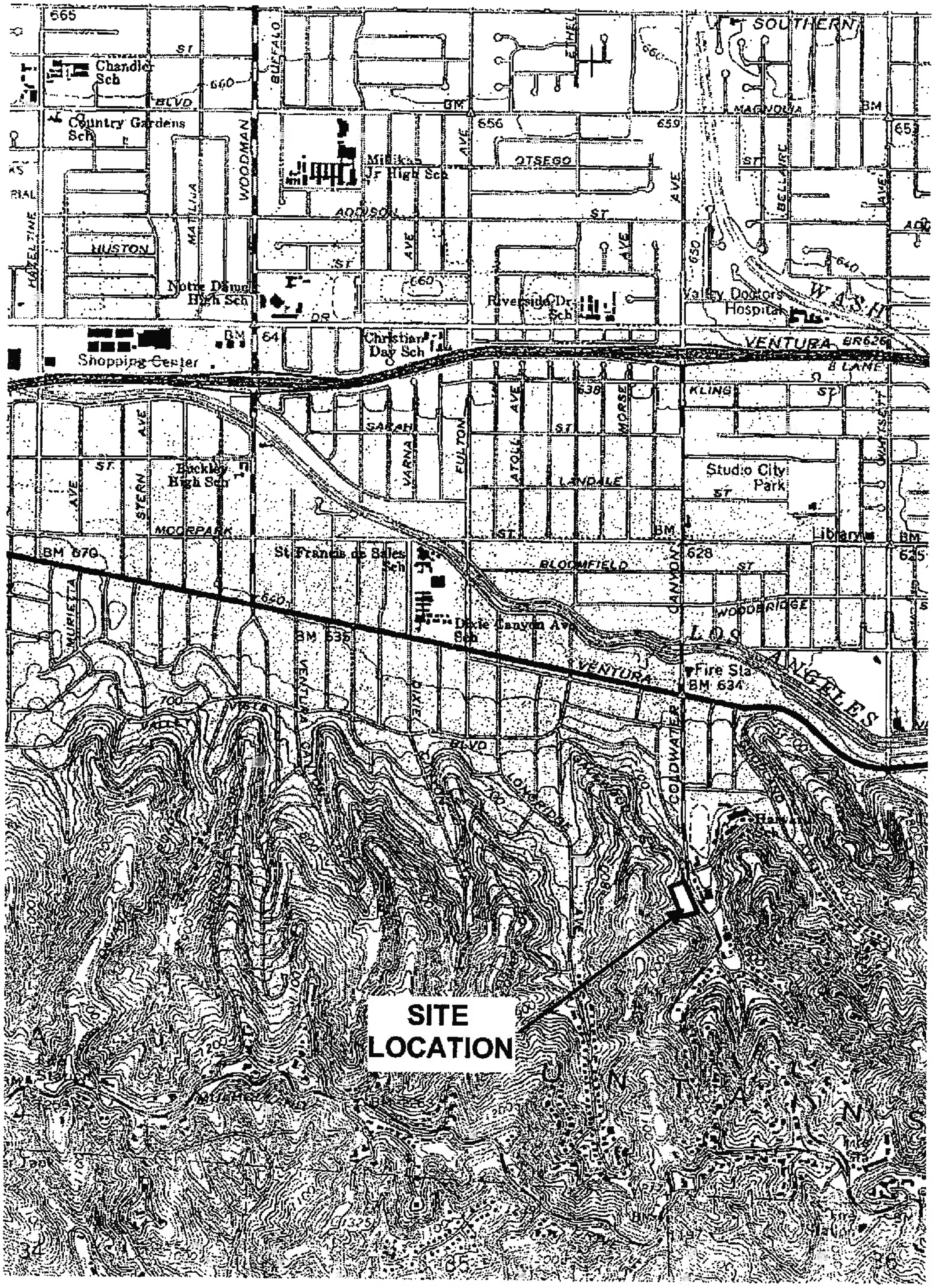
2. AEG, 1982, Geologic Maps, Santa Monica Mountains, Compiled by Bureau of Engineering, Department of Public Works, City of Los Angeles, Book of Geologic Maps; Scale 1:48,000.

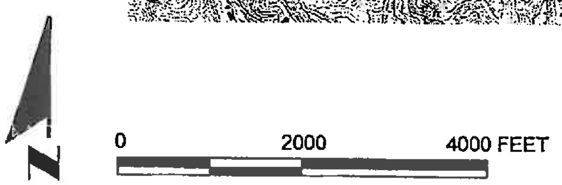
3. California Department of Conservation, Division of Mines and Geology (1997), "Seismic Hazard Zone Map for the Van Nuys 7.5-Minute Quadrangle, Los Angeles County, California," Updated June 2005.

4. United States Geological Survey, "Seismic Design Values for Buildings, Seismic Hazard Mapping, Research and Monitoring, Website Address: http://earthquake.usgs.gov/research/hazmaps/design/index.php.

5. Blake, T.F. (2000), "FRISKSP, A Computer Program for the Probabilistic Estimation of Uniform-Hazard Spectra Using Faults as Earthquake Sources," Version, 4.00.

6. FHWA, "Appendix B1, FHWA Guide Specification for Permanent Soils Nails and Wall Excavation, Manual for Design and Construction Monitoring of Soil Nail Walls," FHWA Publication No. FHWA-SA-96-069, November 1996.





BASE MAP REPRODUCED FROM VAN NUYS QUADRANGLE FROM USGS MAPS

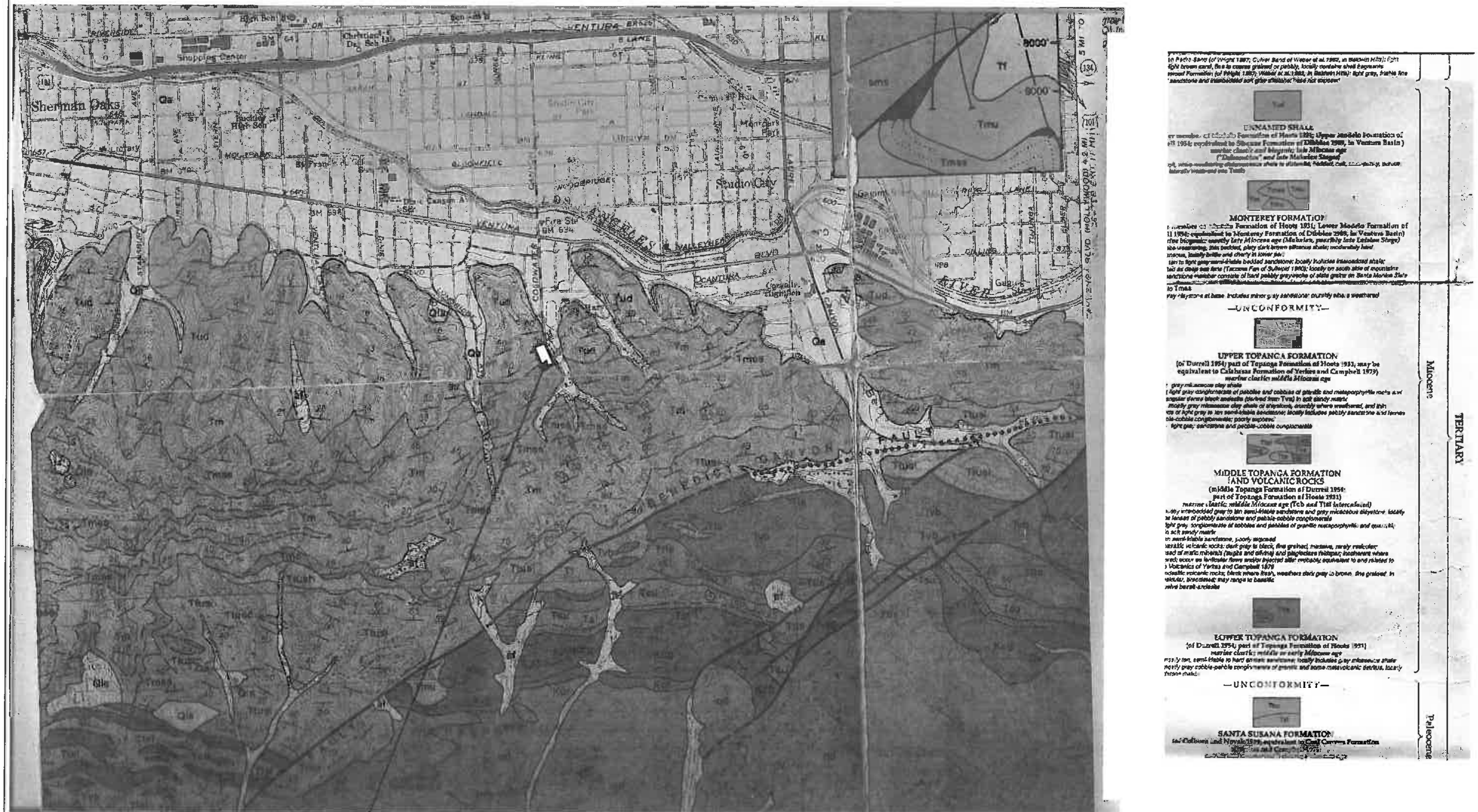
GPI GEOTECHNICAL PROFESSIONALS, INC.

HARVARD WESTLAKE

GPI PROJECT NO. 2270.I | SCALE: 1" = 2000'

SITE LOCATION

FIGURE 1



GEOLOGY MAP FROM DIBBLEE (REFERENCE 1)

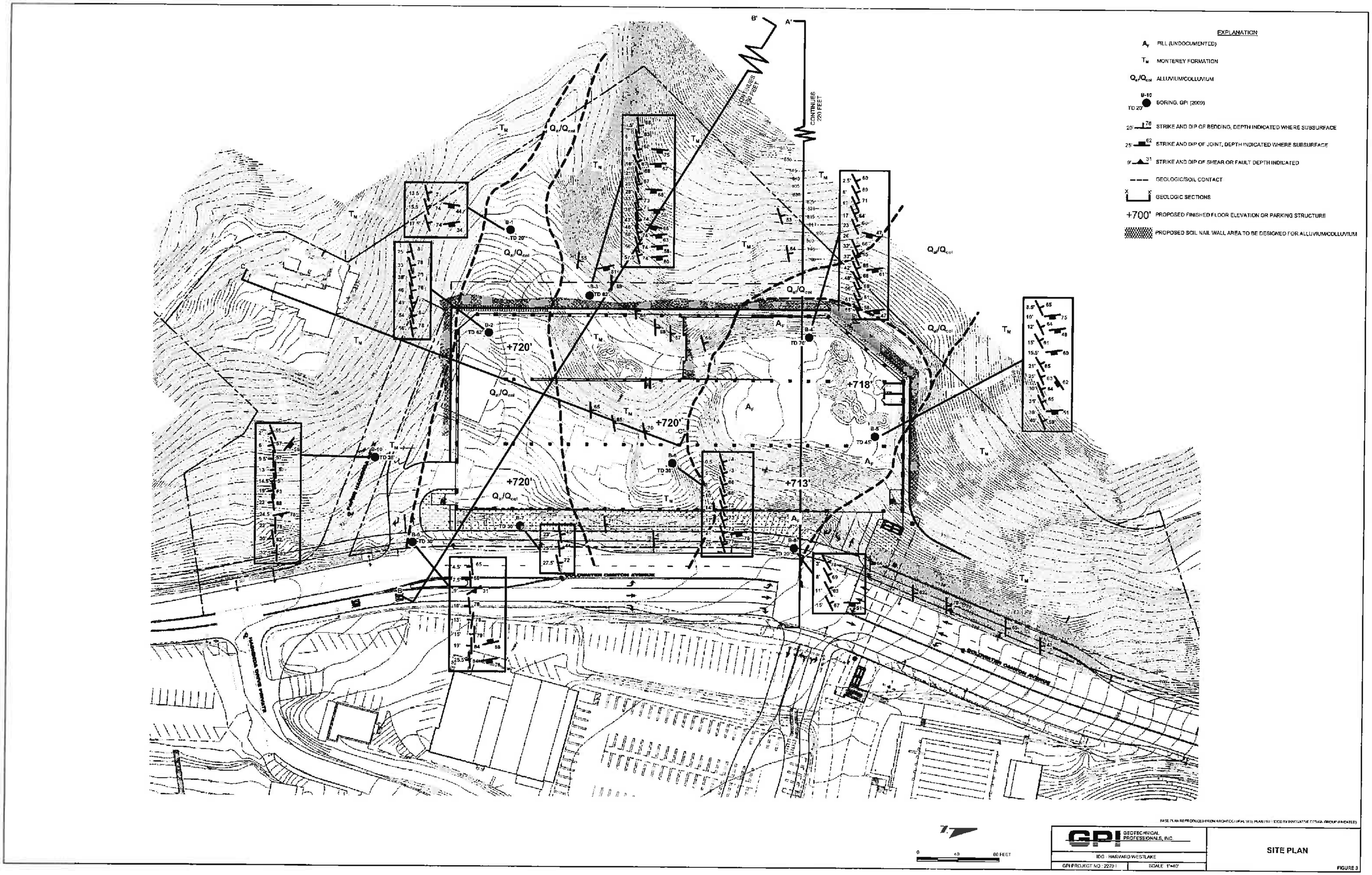
GPI GEOTECHNICAL PROFESSIONALS, INC.

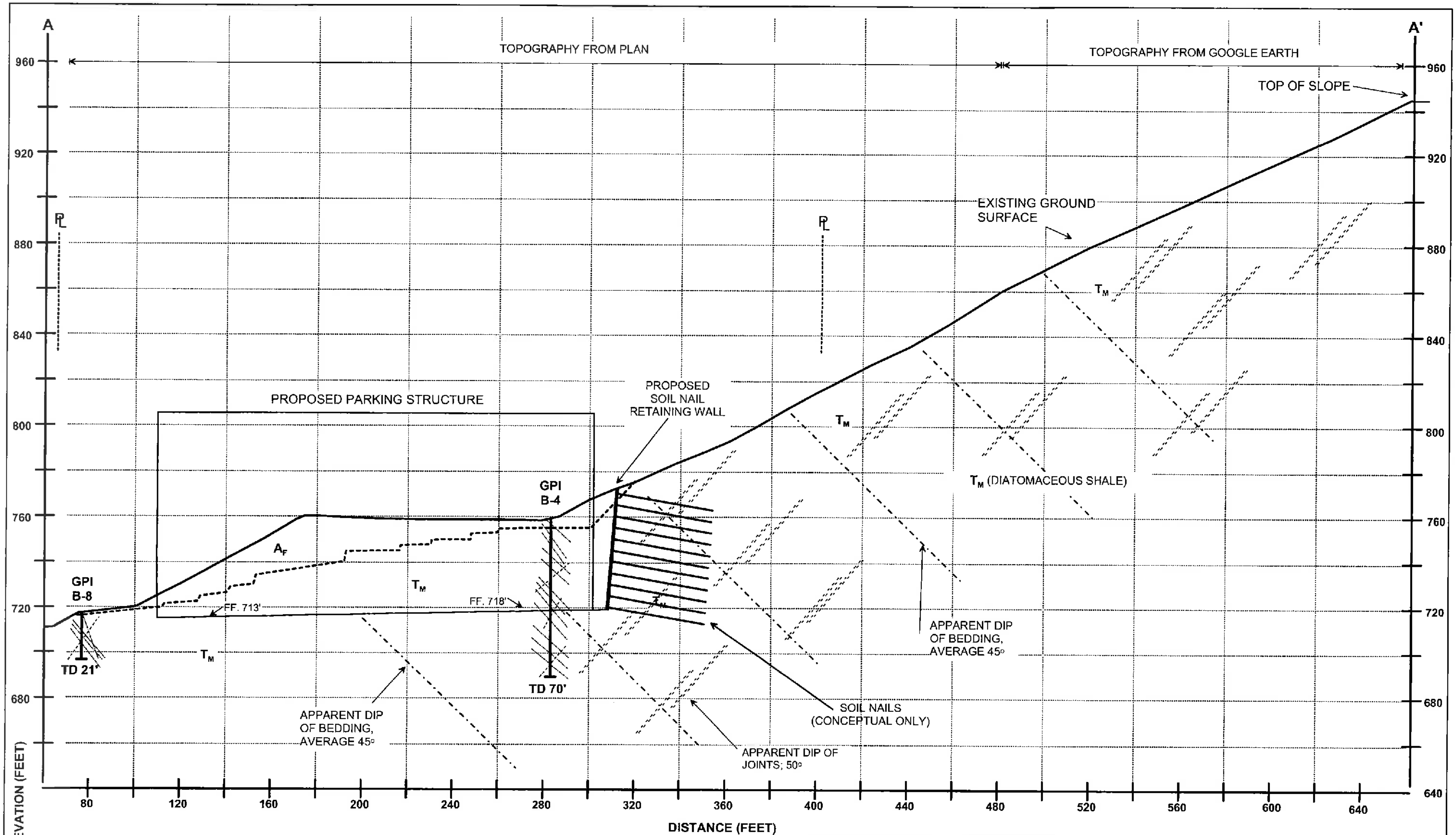
HARVARD WESTLAKE

GPI PROJECT NO.: 2270.I | SCALE: 1" = 1500'

REGIONAL GEOLOGIC MAP

FIGURE 2


EXPLANATION
A_F FILL (UNDOCUMENTED)
T_M MONTEREY FORMATION
Q_{al}/Q_{col} ALLUVIUM/COLLUVIUM
BORING, GPI (2009)
STRIKE AND DIP OF BEDDING, DEPTH INDICATED WHERE SUBSURFACE
STRIKE AND DIP OF JOINT, DEPTH INDICATED WHERE SUBSURFACE
STRIKE AND DIP OF SHEAR OR FAULT DEPTH INDICATED
GEOLOGIC/SOIL CONTACT
GEOLOGIC SECTIONS
+700' PROPOSED FINISHED FLOOR ELEVATION OR PARKING STRUCTURE
PROPOSED SOIL NAIL WALL AREA TO BE DESIGNED FOR ALLUVIUM/COLLUVIUM
CONTINUES 220 FEET
+720'
+718'
+713'
60 FEET
GPI GEOTECHNICAL PROFESSIONALS, INC.
IDG - HARVARD WESTLAKE
GPI PROJECT NO.: 2270.I
SCALE: 1"=40'
SITE PLAN
FIGURE 3



GPI GEOTECHNICAL PROFESSIONALS, INC.

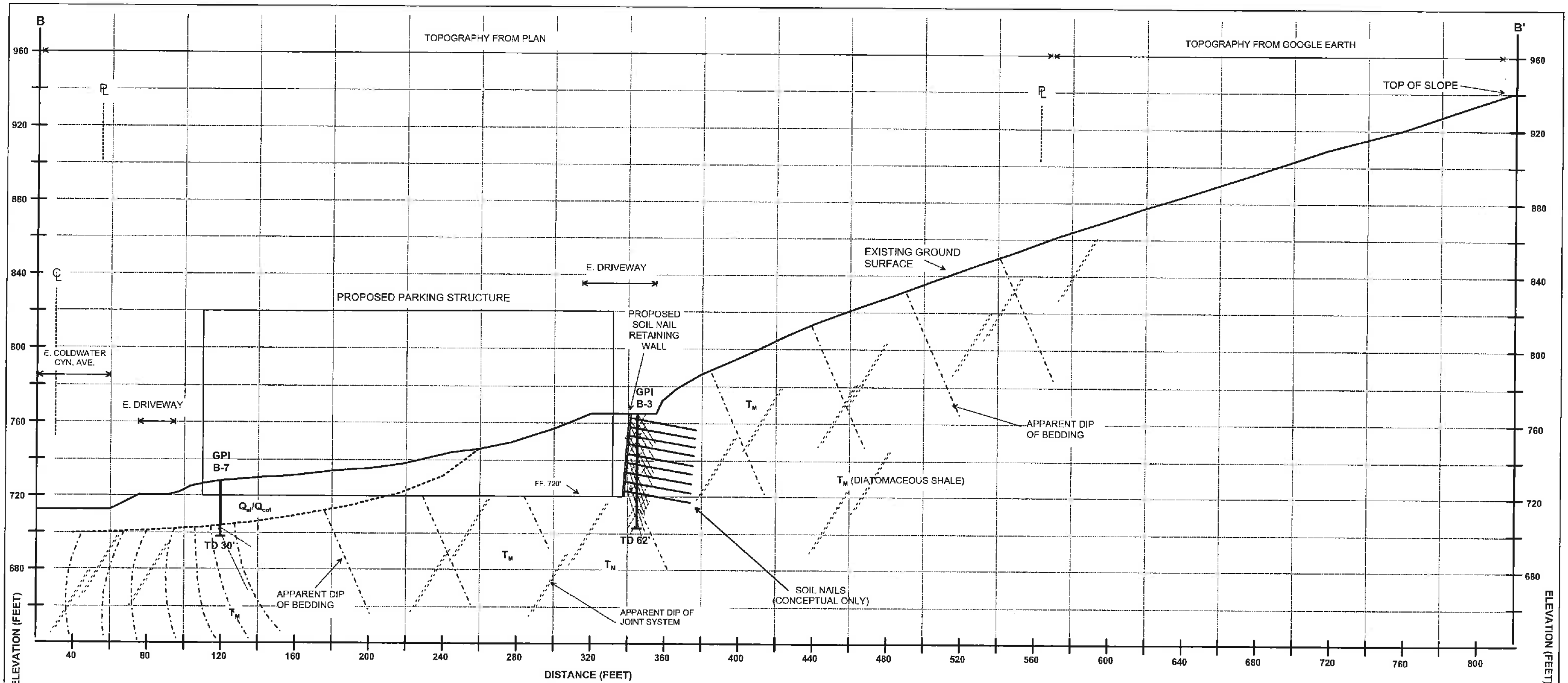
IDG - HARVARD WESTLAKE

GPI PROJECT NO.:2270.I

SCALE: 1" = 40'

CROSS SECTION A-A'

FIGURE 4



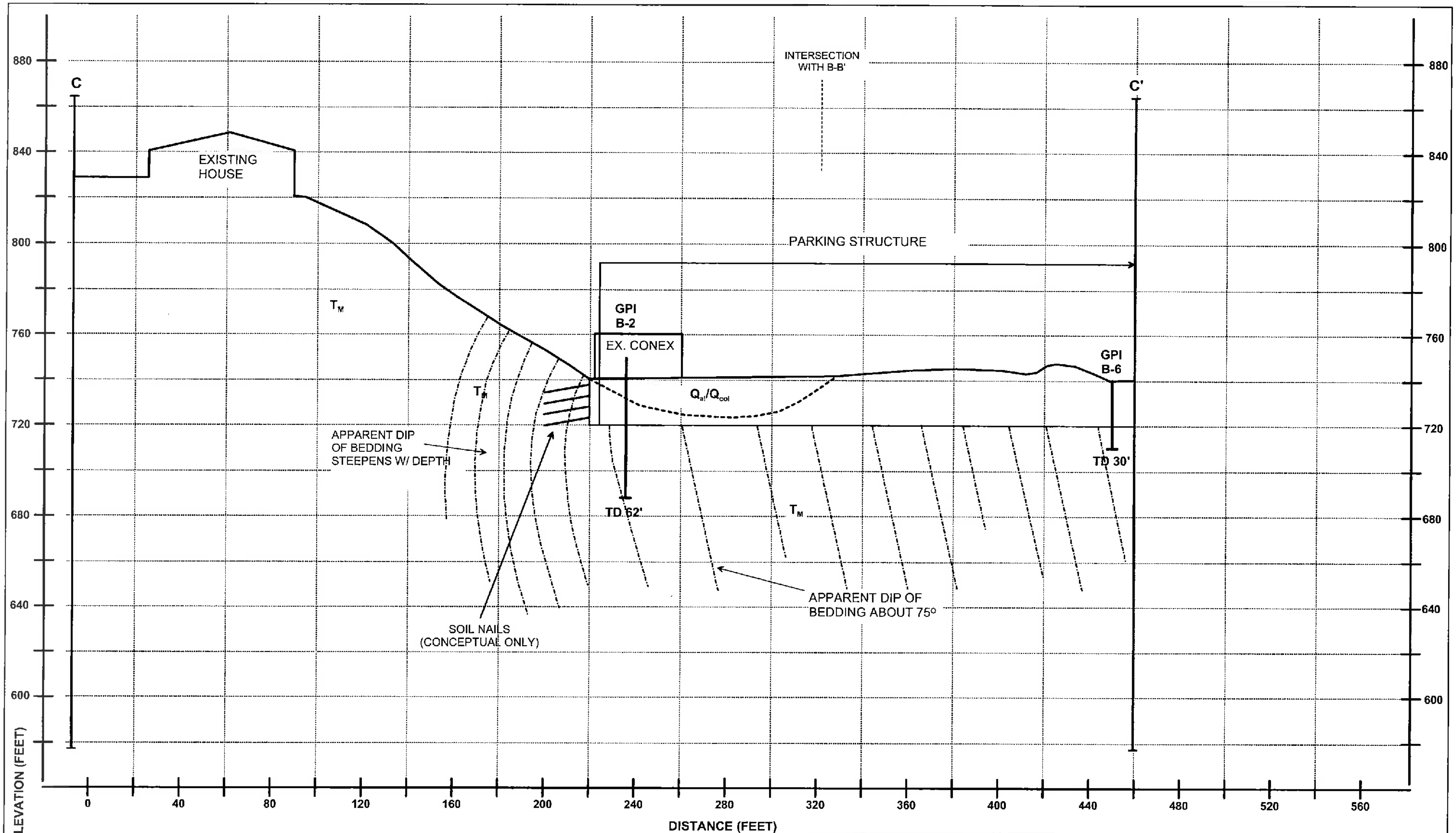
GPI GEOTECHNICAL PROFESSIONALS, INC.

IDG - HARVARD WESTLAKE

GPI PROJECT NO.:2270.I | SCALE: 1" = 40'

CROSS SECTION B-B'

FIGURE 5



GPI GEOTECHNICAL PROFESSIONALS, INC.

CROSS SECTION C-C'

IDG - HARVARD WESTLAKE

GPI PROJECT NO.:2270.I | SCALE: 1" = 40'

FIGURE 6

APPENDIX A

APPENDIX A

EXPLORATORY BORINGS

The subsurface conditions at the site were investigated by drilling and sampling ten exploratory borings. The borings were advanced to depths of 21 to 71 feet below the existing ground surface. The location of the exploration is shown on the Site Plan, Figure 2.

The borings were drilled using truck-mounted bucket auger equipment. Relatively undisturbed samples were obtained using a brass-ring lined sampler (ASTM D 3550). The brass-rings have an inside diameter of 2.42 inches. The ring samples were driven into the soil by a 2400-pound hammer dropping 12 inches. At depths from 24 to 43 feet, the ring samples were driven into the soil by a 1550-pound hammer dropping 12 inches. At depths below 43 feet, the ring samples were driven into the soil by an 850-pound hammer dropping 12 inches. The number of blows needed to drive the sampler into the soil was recorded as the penetration resistance. One blow with a 2400-pound Kelly bar (upper 25 feet) typically provides an equivalent penetration of 8 to 10 blows with the drive sampler using the hollow-stem rig.

The field explorations for the investigation were performed under the continuous technical supervision of GPI's representative, who visually inspected the site, maintained detailed logs of the borings, classified the soils encountered, and obtained relatively undisturbed samples for examination and laboratory testing. The soils encountered in the borings were classified in the field and through further examination in the laboratory in accordance with the Unified Soils Classification System. Detailed logs of the borings are presented in Figures A-1 to A-10 in this appendix.

The boring location was laid out in the field by measuring from existing site features. The ground surface elevations at the boring locations were estimated from a preliminary site plan prepared by Innovative Design Group (not dated) and should be considered approximate.

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	ELEVATION (FEET)
					0	
						760
	18.4	61	3	D	5	
						755
	19.3	65	3	D	10	
						750
	25.7	82	9	D	15	
						745
	37.7	72	11	D	20	

DESCRIPTION OF SUBSURFACE MATERIALS

This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Alluvium/Colluvium:
SILTY CLAY (CL) dark brown, slightly moist, soft, porous, with 20-30% white, gravel-cobble size shale fragments, with many roots

SANDY SILT (ML) brown, very moist, very stiff
@ 6' and 10'-6", thick gravel beds of shale fragments, irregular

Monterey Formation:
SILTSTONE gray to light brown, very moist, hard, highly weathered, fractured, diatomaceous shale
No continuous or coherent bedding
@ 13 feet, hard, intact diatomaceous shale with continuous bedding.
Mod-highly fractured with open fractures 1/8" to 1/4" wide
@ 13.5 feet, B: N78E, 71NW
@ 15.5 feet, B: N76E, 74NW
J: N10E, 44SE
Gypsum filled joints at 6"-12" spacing
@ 17.5 feet, B: N72E, 74NW
J: N10W, 34NE
As above, 6"-12" spacing, gypsum filled
Total Depth 21 feet
No water or caving
Backfilled and tamped with drill cuttings

SAMPLE TYPES
C Rock Core
S Standard Split Spoon
D Drive Sample
B Bulk Sample
T Tube Sample

DATE DRILLED:
11-18-09
EQUIPMENT USED:
24" Bucket Auger
GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2270.I
HARVARD-WESTLAKE

LOG OF BORING NO. B-1

FIGURE A-1

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
				0	Alluvium/Colluvium: **SANDY SILT (ML)** dark brown, dry to slightly moist, soft, porous, with white diatomaceous shale fragments, roots to 2-3" diameter, massive	745
18.3	63	2	D	5	@ 5 feet, stiff	
						740
					@ 8 feet, 6"-8" thick, poorly defined gravel bed of shale fragments	
20.4	64	2	D	10		
29.0	81					
25.0	85					735
25.0	85					
					@ 14 feet, poorly defined gravel bed of shale fragments	
19.4	72	3	D	15	**SILT (MH)** brown, wet, very stiff	
						730
22.8	63	3	D	20		
						725
					Monterey Formation: **SILTSTONE** gray to light brown, very moist, hard, high weathered with soil pockets, no continuous bedding	
32.7	77	10	D	25	@ 25 feet, highly fractured but hard shale with gypsum filled fractures B: N74E, 81SE	
						720
				30		
						715
					@ 33 feet, B: N71E, 78NW, shale continues highly fractured with filled and partially filled gypsum seams	
41.4	69	6/6"	D	35		
						710
					@ 38 feet, B: N72E, 78NW, shale is very hard with gypsum filled fractures	

This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 11-18-09

EQUIPMENT USED: 24" Bucket Auger

GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 2270.I

HARVARD-WESTLAKE

LOG OF BORING NO. B-2

FIGURE A-2

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
				40	@ 40 feet, all joints tight, filled with gypsum, shale is very hard	
						705
42.0	70	30/9"	D	45	@ 45 to 46 feet, start of unoxidized shale in irregular patches	
					@ 46 feet, B: N74E, 76NW	700
					@ 49 feet, B71E, 72NW	
				50	@ 50 feet, dark grey, unoxidized shale, very hard, few gypsum filled fractures	
					@ 52 feet, unfractured, no gypsum	695
					@ 54 feet, B: N71E, 84NW	
		50/5"	D	55		
					@ 56 feet, B: N80E, 78NW	
						690
				60		
32.9	75	50/5"	D			685
					Total Depth 62.5 feet No water or caving Backfilled with cuttings and tamped	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
11-18-09

EQUIPMENT USED:
24" Bucket Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2270.I
HARVARD-WESTLAKE

LOG OF BORING NO. B-2

FIGURE A-2

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
				0	Fill: SILT (ML) yellow brown and white, diatomaceous silt, with shale debris	
					Monterey Formation: SILTSTONE gray to light brown, very moist, hard, moderately fractured, diatomaceous shale	760
46.2	64	8	D	5	@ 1.5 feet, B: N85E, 68NW	
					@ 6 feet, B: N74E, 63NW, hard, slightly fractured	755
33.6	82	5	D	10	@ 10 feet, joint set @ 12" spacing J: NS, 75E B: N72E, 68NW	750
34.0	84	7	D	15	@ 16 feet, B: N62E, 67NW J: N5W, 67NE	745
28.4	87	4	D	20	@ 21 feet, B: N71E, 68NW very hard, few joints, very tight	740
37.7	73	14	D	25	@ 25 feet, B: N72E, 67NW	
					@ 28 feet, J: N8E, 68SE (tight)	735
84.0	49	20	D	30		730
					@ 33 feet, B: N70E, 73NW	
44.4	71	11	D	35	@ 36 feet, J: N8W, 58NE B: N70E, 71NW	725

This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 11-17-09
EQUIPMENT USED: 24" Bucket Auger
GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 2270.I
HARVARD-WESTLAKE

LOG OF BORING NO. B-3

FIGURE A-3

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
44.7	75	13	D	40		
					@ 43 feet, B: N75E, 74NW	720
33.4	83	30/9"	D	45		
					@ 46 feet, J: N18E, 70SE	
						715
51.6	70	35	D	50	@ 50 feet, B: N72E, 74NW J: N12W, 63NE	
						710
43.6	72	50/11"	D	55		
					@ 56 feet, J: N8W, 58NE B: N71E, 74NW @ 57.5 feet, B: N73E, 75NW J: N7W, 60NE	705
50.3	69	50/10"	D	60		
52.7	64	50/7"	D			
					Total Depth 63 feet No water or caving Backfilled with cuttings and tamped	700

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 11-17-09

EQUIPMENT USED: 24" Bucket Auger

GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 2270.I

HARVARD-WESTLAKE

LOG OF BORING NO. B-3

FIGURE A-3

Moisture (%)	Dry Density (PCF)	Penetration Resistance (Blows/Foot)	Sample Type	Depth (Feet)	*Description of Subsurface Materials*	Elevation (Feet)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
				0	Fill: **SILT (ML)** brown, dry, soft, gravelly	
					Colluvium: **SILT (ML)** dark brown, moist, firm, with white diatomaceous shale fragments	755
11.2	90	8/5"	D	5	Monterey Formation: **SILTSTONE** grey to light brown, moist, hard, carbonate bed, blocky fracturing, loose @ 2.5 feet, B: N60E, 60NW	
					@ 3 to 11 feet, moderately-highly fractured/weathered, diatomaceous shale	750
					@ 8 feet, B: N61E, 69NW	
48.3	62	4	D	10	@ 10 feet, very moist	
					@ 12 feet, diatomaceous shale, very tight, very few fractures/joints B: N68E, 71NW	745
68.6	55	5	D	15		
					@ 17 feet, B: N70E, 64NW	740
67.7	55	4	D	20		
					@ 23 feet, B: N68E, 60NW	735
88.2	42	20	D	25		
					@ 26 feet, J: N10E, 47SE	730
104.0	42	28/10"	D	30		
					@ 32 feet, B: N68E, 66NW	725
98.6	41	10/7"	D	35		
					@ 37 feet, B: N66E, 65NW	720

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 11-16-09
EQUIPMENT USED: 24" Bucket Auger
GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 2270.I
HARVARD-WESTLAKE

LOG OF BORING NO. B-4

FIGURE A-4

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
109.6	38	10/6"	D	40		
					@ 42 feet, B: N67E, 65NW J: N11W, 61NE (tight)	715
104.3	40	26/6"	D	45		
					@ 48 feet, B: N70E, 68NW change to dark grey, unoxidized shale @ 49 feet, B: N70E, 68NW	710
43.0	74	25/6"	D	50		
						705
31.1	83	50/6"	D	55		
					@ 56 feet, B: N68E, 66NW	
						700
37.3	72	50/7"	D	60		
					@ 61 feet, B: N68E, 61NW	
						695
63.0	57	20/7"	D	65		
					@ 66 feet, B: N65E, 58NW J: N9W, 47NE (tight)	690
83.7	48	20/7"	D	70		
					Total Depth 71 feet No water or caving Backfilled with cuttings and tamped	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
11-16-09

EQUIPMENT USED:
24" Bucket Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2270.I
HARVARD-WESTLAKE

LOG OF BORING NO. B-4

FIGURE A-4

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
				0	Fill: **SILT (ML)** brown, dry, soft, horizontal contact with soil, with 3/4" crushed gravel	760
					Residual Soil/Colluvium: **CLAYEY SILT (ML)/SILTY CLAY (CL)** dark brown,	
16.1	74	4	D	5	moist, firm, porous, with 10%-20% shale fragments, with roots to 1/2" diameter	755
					Monterey Formation: **SILTSTONE** whitish and yellow brown, very moist, hard, diatomaceous shale, very few fractures	
58.4	49	12	D	10	@ 8.5 feet, B: N72E, 65NW @ 10 feet, J: N10W, 75NE	
					@ 12 feet, B: N71E, 4NW J: N5W, 48NE	750
46.4	58	7/8"	D	15	@ 15 feet, B: N64E, 61NW @ 15.5 feet, J: N12W, 60NE partially open to 1/4" with roots	745
60.5	56	10/10"	D	20	@ 21 feet, B: N64E, 65NW	740
45.3	72	13	D	25	@ 25 feet, B: N68E, 63NW J: N65E, 62SE (tight)	735
50.9	67	12	D	30	@ 30 feet, B: N68E, 64NW	730
84.0	48	15	D	35	@ 35 feet, B: N70E, 65NW	725
					@ 38 feet, J: N8W, 51NE	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 11-17-09

EQUIPMENT USED: 24" Bucket Auger

GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 2270.I

HARVARD-WESTLAKE

LOG OF BORING NO. B-5

FIGURE A-5

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
						This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
	74.5	54	11	D	40	@ 40 feet, B: N69E, 59NW	
							720
	83.6	47	30/6"	D	45		
						Total Depth 46 feet No water or caving Backfilled	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
11-17-09

EQUIPMENT USED:
24" Bucket Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2270.I
HARVARD-WESTLAKE

LOG OF BORING NO. B-5

FIGURE A-5

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
				0	Monterey Formation: **SILTSTONE** grey and yellowish white, moist to very moist, hard, diatomaceous shale, laminated to thin bedded, few fractures	740
					@ 0.5 feet, B: N71E, 74NW	
29.0	75	4	D	5	@ 5 feet, B: N68E, 73NW	735
					@ 8 feet, B: N69E, 68NW	
38.3	76	9	D	10	@ 10 feet, B: N69E, 68NW	730
					@ 14 feet, B: N71E, 72NW	
				15		725
					@ 17 feet, B: N70E, 73NW	
90.7	45	8/7"	D	20		720
					@ 22 feet, B: N71E, 72NW	
				25	@ 25 feet, B: N71E, 77NW	715
76.2	47	9	D	30		710
					Total Depth 31 feet No water or caving Backfilled with cuttings and tamped	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 11-18-09
EQUIPMENT USED: 24" Bucket Auger
GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 2270.I
HARVARD-WESTLAKE

LOG OF BORING NO. B-6

FIGURE A-6

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
				0	AC Pavement	720
					Fill: **CLAYEY SILT (ML)** brown, slightly moist, firm, white shale fragments	
18.5	55	2	D	5	Alluvium/Colluvium: **CLAYEY SILT (ML)** brown, moist, with sand to gravel size white shale fragments, soft, very porous to about 10 feet then less, roots to 1" diameter	715
20.3	54	3	D	10		710
27.0	73	5	D	15		705
43.9	69	3	D	20	Monterey Formation: **SILTSTONE** grey to light brown, very moist, very stiff, highly weathered and fractured shale, no continuous bedding @ 23 feet, grey to light brown, diatomaceous shale, hard B: N80E, 45NW	700
48.4	71	8	D	25	@ 25 feet, B: N85E, 44NW @ 27.5 feet, B: N75E, 72NW hard, coherent shale	695
50.0	68	6	D	30		690
					Total Depth 31 feet No water or caving	

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED:
11-19-09

EQUIPMENT USED:
24" Bucket Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2270.I
HARVARD-WESTLAKE

LOG OF BORING NO. B-7

FIGURE A-7

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
				0	Fill: **CLAYEY SILT (ML)** yellow brown, dry, soft, with diatomaceous, shale fragments, roots to 1" diameter, horizontal lower contact	720
					Monterey Formation: **BEDROCK** white-yellow brown, very moist, hard, diatomaceous shale, thin bedded, few tight fractures/joints	
90.7	43	6/7"	D	5	@ 3 feet, B: N53E, 73NW	715
					@ 8 feet, B: N58E, 69NW	
67.1	59	7/10"	D	10		710
					@ 11 feet, B: N61E, 63NW J: N5, 51E	
				15	@ 15 feet, B: N63E, 67NW	705
16.1	108	8/7"	D	20		700
					Total Depth 21 feet No water or caving Backfilled with cuttings	

This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 11-18-09
EQUIPMENT USED: 24" Bucket Auger
GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 2270.I
HARVARD-WESTLAKE

LOG OF BORING NO. B-8

FIGURE A-8

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
				0	AC Pavement	715
					Fill: **SILTY CLAY (CL)** dark brown, very moist, firm, with whitish shale fragments, sloping contact with bx below dips to N (parallel to hillside)	
				5	Monterey Formation: **SILTSTONE** yellow brown, shaly siltstone, moist, hard, moderately-highly fractured to 8-9 feet then hard, little fractured @ 4.5 feet, N: N85E, 65SE @ 7.5 feet, B: N87E, 66SE less fractured	710
				10	@ 9 feet, discontinuous shear, paper thin S: N33W, 31NE @ 10 feet, B: N86E, 78SE Hard, few fractures @ 13 feet, B: N81W, 79SW very hard, shaly siltstone	705
				15	@ 15 feet, B: EW, 79S hard, diatomaceous shale, very few irregular fractures	700
				20	@ 19 feet, B: EW, 84S J: N10W, 88SW	695
					@ 22 feet, Darker in color, medium brown, very hard B: EW, 89S J: N10W, 60NE	
				25	@ 22.5 feet, start of dark grey, unoxidized siltstone in irregular patches @ 25.5 feet, B: N88E, 84SE J: N7W, 78NE @ 26 to 30 feet, very hard, dark grey, unoxidized shaly siltstone	690
				30	Total Depth 30 feet No samples collected Backfilled with cuttings	685

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 12-16-09

EQUIPMENT USED: 24" Bucket Auger

GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 2270.I

HARVARD-WESTLAKE

LOG OF BORING NO. B-9

FIGURE A-9

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
					0	Natural: Residual soil **CLAYEY SILT (ML)** dark brown, with shale rock fragments, very moist, soft, porous with roots	740
						Monterey Formation: **SILTSTONE** white chalky diatomaceous shale, laminated/thin bedded, moderately fractured with roots along fractures, hard	
					5	@ 2 feet, B: N71E, 55SE @ 6 feet, as above, light yellowish brown, shaly siltstone, diatomaceous in part B: N79E, 57SE J: N70W, 56NE	735
					10	@ 9.5 feet, B: N85E, 57SE hard shale, very tight, few fractures	730
						@ 13 feet, polished, paper thin clay, parallel bedding, grooves, parallel dip	
					15	S/B: N89E, 57SE	725
					20	@ 19 feet, B: N84E, 63SE hard diatomaceous shale, little fractures J: N80W, 45NE @ 22 feet, B: N83E, 68SE	720
					25	@ 24.5 feet, 1/2" wide shear zone, disrupts bedding S/F: N15W, 78NE @ 26 feet, B: N87W, 78SW	715
					30	@ 30 feet, hard grey shale, unfractured B: 84E, 82SE @ 32 feet, B: EW, 83S, hard (tight) @ 32 to 35 feet, patches of dark grey, unoxidized shale	710
					35	Total Depth 35 feet No water or caving Backfilled; No samples collected	705

SAMPLE TYPES
- C Rock Core
- S Standard Split Spoon
- D Drive Sample
- B Bulk Sample
- T Tube Sample

DATE DRILLED: 12-16-09

EQUIPMENT USED: 24" Bucket Auger

GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 2270.I

HARVARD-WESTLAKE

LOG OF BORING NO. B-10

FIGURE A-10

APPENDIX B

APPENDIX B

LABORATORY TESTS

INTRODUCTION

Representative undisturbed soil samples and bulk samples were carefully packaged in the field and sealed to prevent moisture loss. The samples were then transported to our Cypress office for examination and testing assignments. Laboratory tests were performed on selected representative samples as an aid in classifying the soils and to evaluate the physical properties of the soils affecting foundation design and construction procedures. Detailed descriptions of the laboratory tests are presented below under the appropriate test headings. Test results are presented in the figures that follow.

MOISTURE CONTENT AND DRY DENSITY

Moisture content and dry density were determined from a number of the ring samples. The samples were first trimmed to obtain volume and wet weight and then were dried in accordance with ASTM D 2216. After drying, the weight of each sample was measured, and moisture content and dry density were calculated. Moisture content and dry density values are presented on the boring logs in Appendix A.

ATTERBERG LIMITS

Liquid and plastic limits were determined for a selected sample in accordance with ASTM D4318. Results of the Atterberg Limits test are summarized on Figure B-1.

DIRECT SHEAR

Direct shear tests were performed on undisturbed and remolded bulk samples in accordance with ASTM D 3080. The bulk sample was remolded to approximately 90 percent of the maximum dry density (ASTM D 1557). The samples were placed in the shear machine, and a normal load comparable to the in-situ overburden stress was applied. The samples were inundated, allowed to consolidate, and then were sheared to failure. The tests were repeated on additional test specimens under increased normal loads. Shear stress and sample deformation were monitored throughout the test. The results of the direct shear tests are presented in Figures B-2 to B-4.

A direct shear test was performed on ring samples to determine the residual strength of the soils after repeated deformation of the soil. The samples were sheared up to a deformation at which the shear resistance reached a well defined residual value. The procedure was repeated on additional test specimens from the same soil layer under increased normal loads. The results of the direct shear test to determine the residual value are presented in Figures B-5 to B-9.

CONSOLIDATION

A one-dimensional consolidation test was performed on an undisturbed sample in accordance with ASTM D 2435. After trimming the ends, the sample was placed in the consolidometer and loaded to up to 0.4 ksf. Thereafter, the sample was incrementally loaded to a maximum load of up to 25.6 ksf. The sample was inundated at 1.6 ksf. Sample deformation was measured to 0.0001 inch. Rebound behavior was investigated by unloading the sample back to 0.4 ksf. Results of the consolidation test, in the form of percent consolidation versus log pressure are presented in Figures B-10 and B-11.

EXPANSION INDEX

Expansion index tests were performed on bulk samples and composite ring samples. The tests were performed in accordance with ASTM 4289 to assess the expansion potential of on-site soils. The results of the test are summarized below:

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	EXPANSION INDEX
B-2	5/10/15	Silt (MH)	41
B-2	10-15	Silt (MH)	42
B-3	20-30	Silt (MH)	27

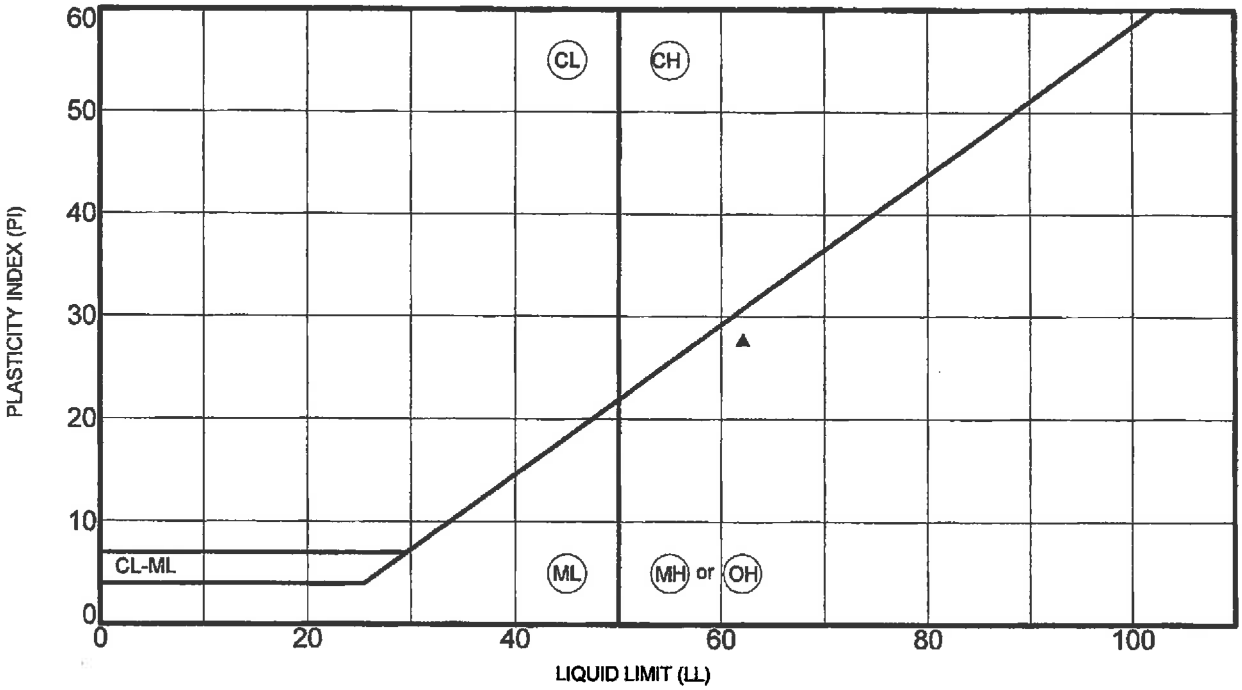
COMPACTION TEST

Maximum dry density/optimum moisture tests were performed in accordance with ASTM D 1557 on representative bulk samples of the surficial soils. The test result is as follows:

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)
B-2	10-15	Silt (MH)	79	40.0
B-3	20-30	Sandy Silt (ML)	99	25.0

CORROSIVITY

Soil corrosivity testing was performed by Schiff Associates on soil samples provided by GPI. The test results are summarized in Table 1 of this appendix.



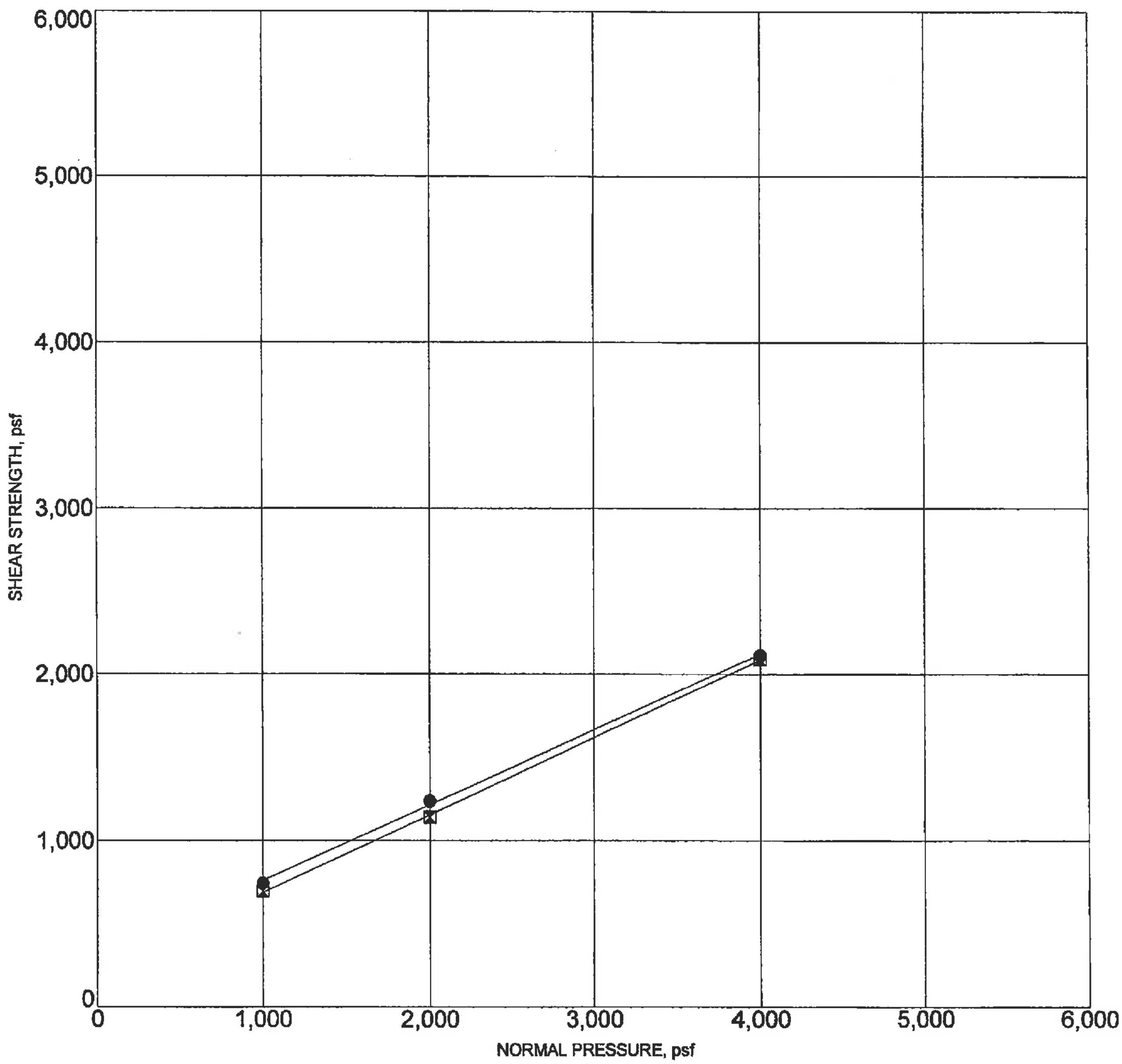
	SAMPLE LOCATION		LL	PL	PI	Fines, %	Classification
▲	B-2	15.0	62	34	28		SILT (MH)

PROJECT: HARVARD-WESTLAKE PROJECT NO. 2270.I

GPI

ATTERBERG LIMITS TEST RESULTS

FIGURE B-1



● **PEAK STRENGTH**
Friction Angle= 24 degrees
Cohesion= 306 psf

☒ **ULTIMATE STRENGTH**
Friction Angle= 25 degrees
Cohesion= 222 psf

Note: Samples remolded to 90% of maximum dry density

Sample Location		Classification	DD,pcf	MC,%
B-2	10-15	SANDY SILT (MH)	81	29.0

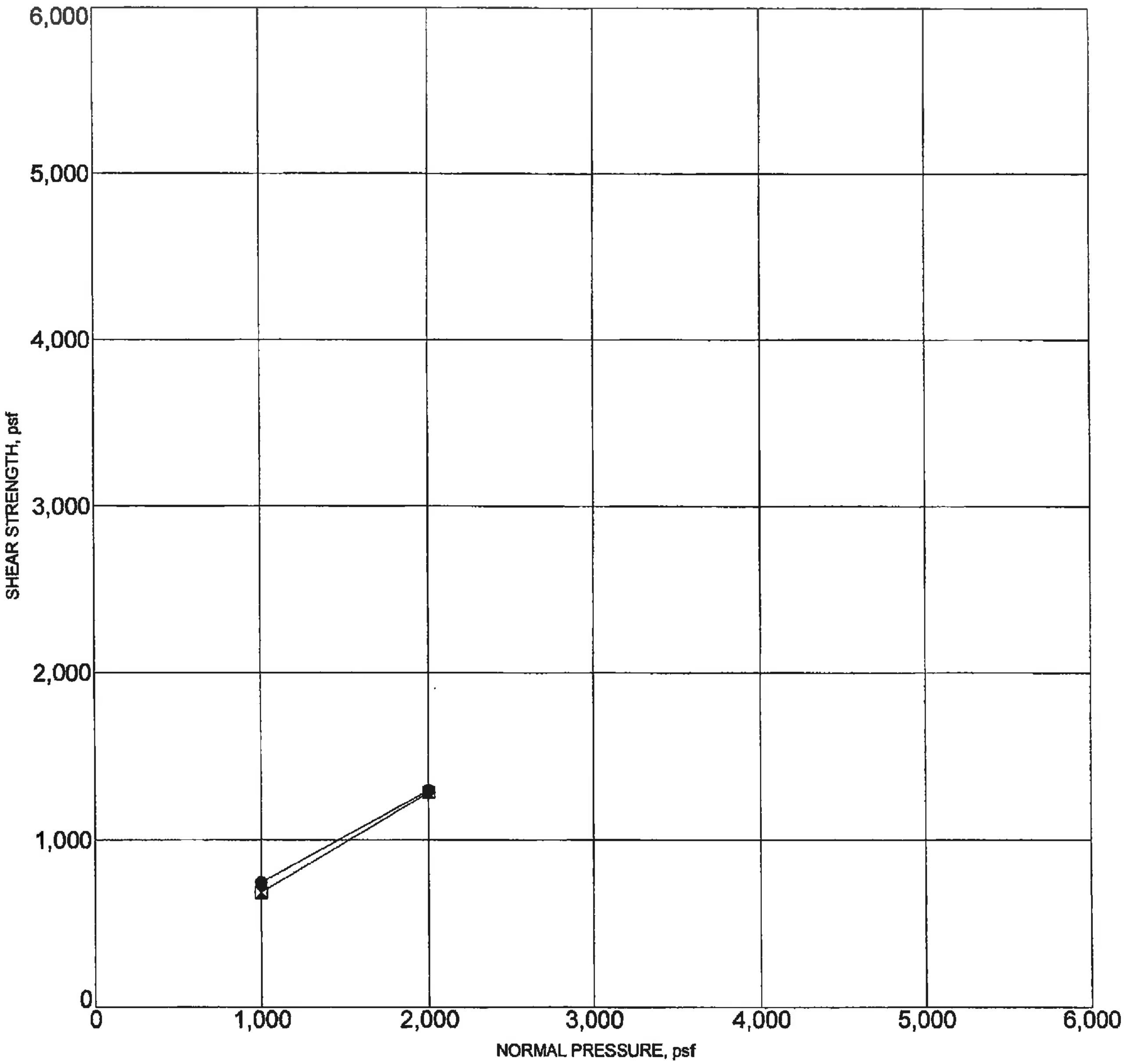
PROJECT: HARVARD-WESTLAKE

PROJECT NO.: 2270.I

GPI

DIRECT SHEAR TEST RESULTS

FIGURE B-2



● PEAK STRENGTH
Friction Angle= 29 degrees
Cohesion= 192 psf

☒ ULTIMATE STRENGTH
Friction Angle= 31 degrees
Cohesion= 84 psf

Sample Location		Classification	DD,pcf	MC,%
B-2	15.0	SILT (MH)	72	19.4

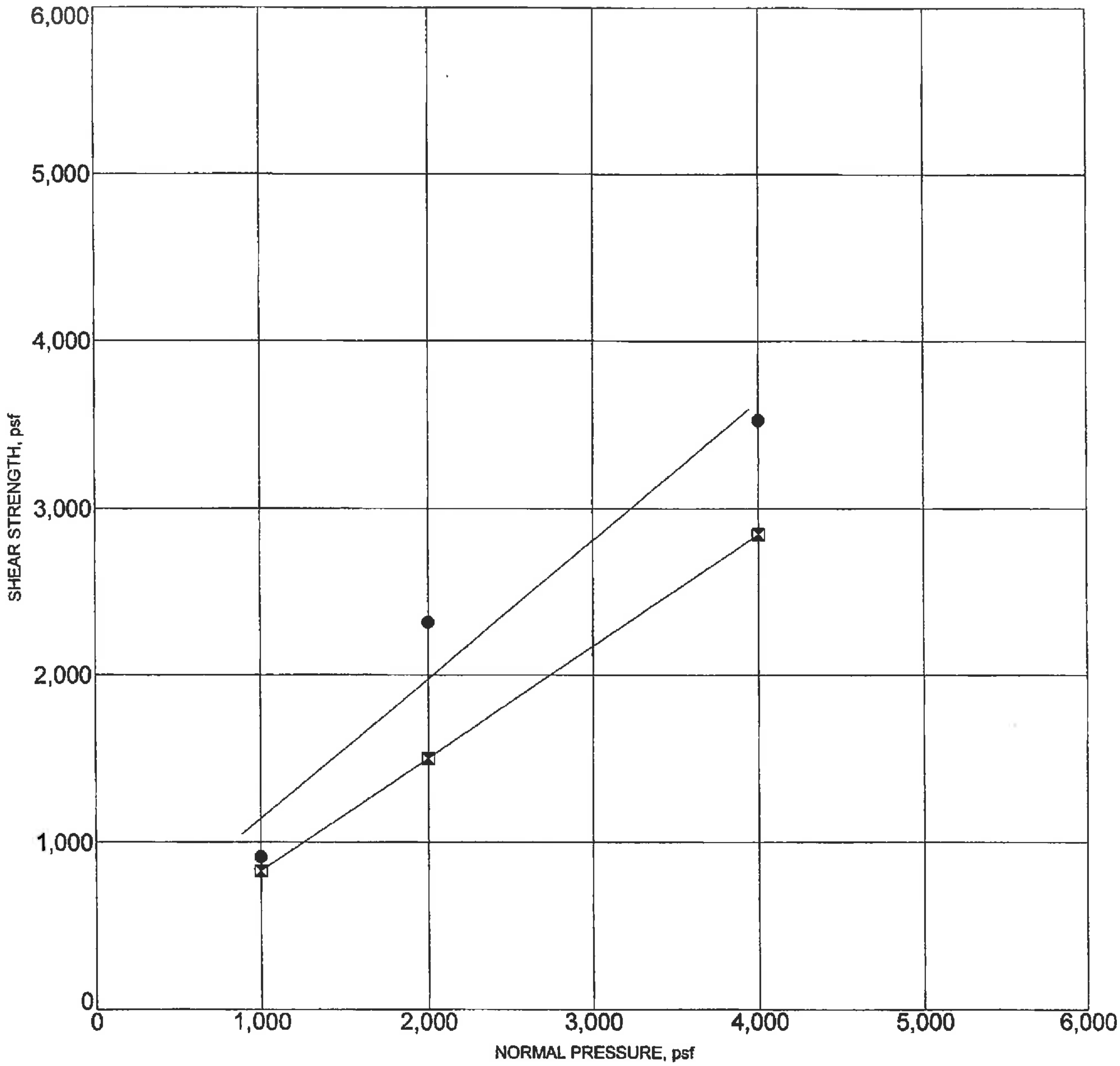
PROJECT: HARVARD-WESTLAKE

PROJECT NO.: 2270.I

GPI

DIRECT SHEAR TEST RESULTS

FIGURE B-3



● **PEAK STRENGTH**
Friction Angle= 40 degrees
Cohesion= 306 psf

☒ **ULTIMATE STRENGTH**
Friction Angle= 34 degrees
Cohesion= 156 psf

Note: Samples remolded to 90% of maximum dry density

Sample Location		Classification	DD,pcf	MC,%
B-3	20-30	SILTSTONE	71	40.5

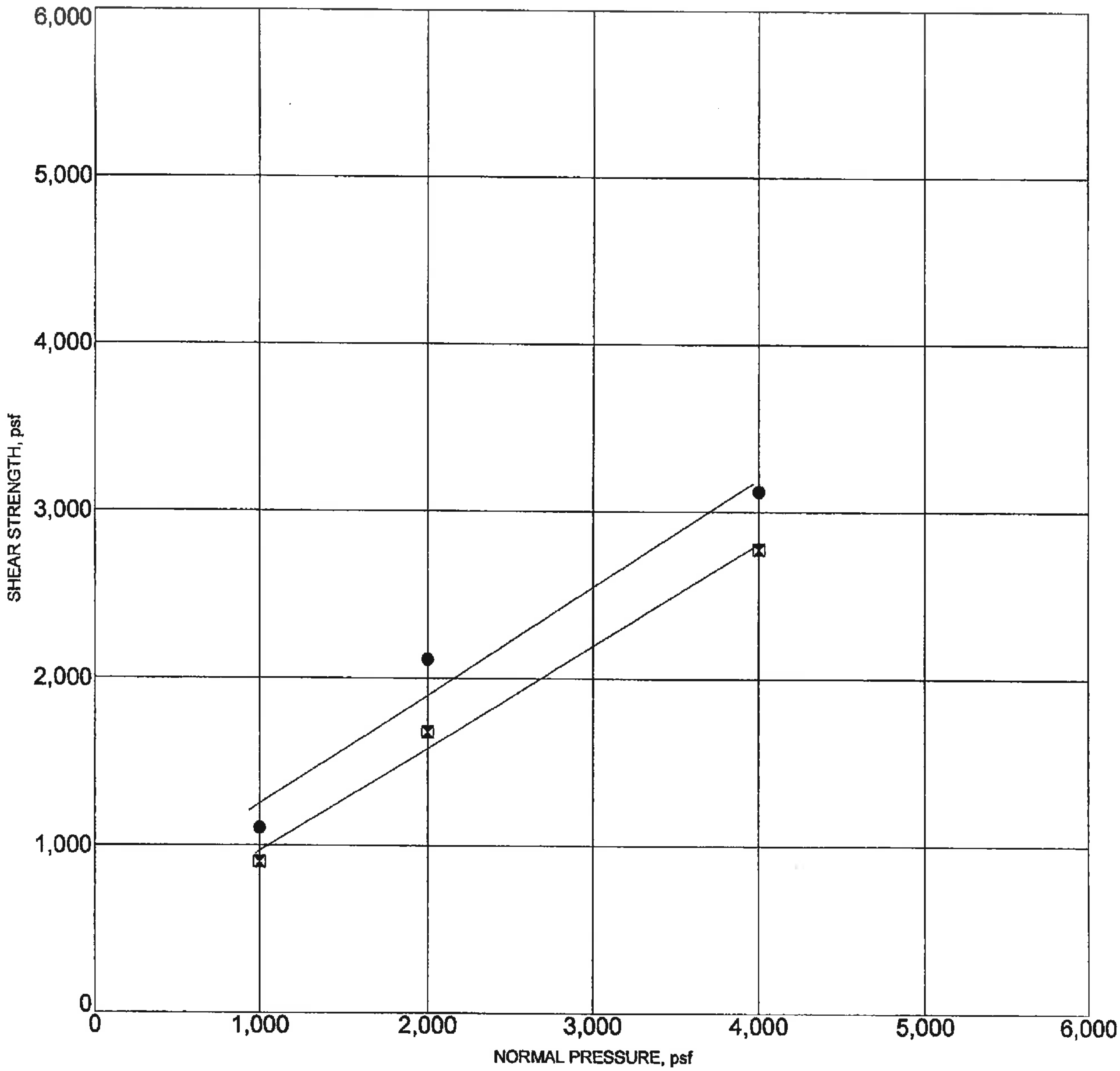
PROJECT: HARVARD-WESTLAKE

PROJECT NO.: 2270.I

GPI

DIRECT SHEAR TEST RESULTS

FIGURE B-4



● **PEAK STRENGTH**
Friction Angle= 33 degrees
Cohesion= 600 psf

⊠ **ULTIMATE STRENGTH**
Friction Angle= 32 degrees
Cohesion= 354 psf

Sample Location		Classification	DD,pcf	MC,%
B-3	35.0	SILTSTONE	71	44.4

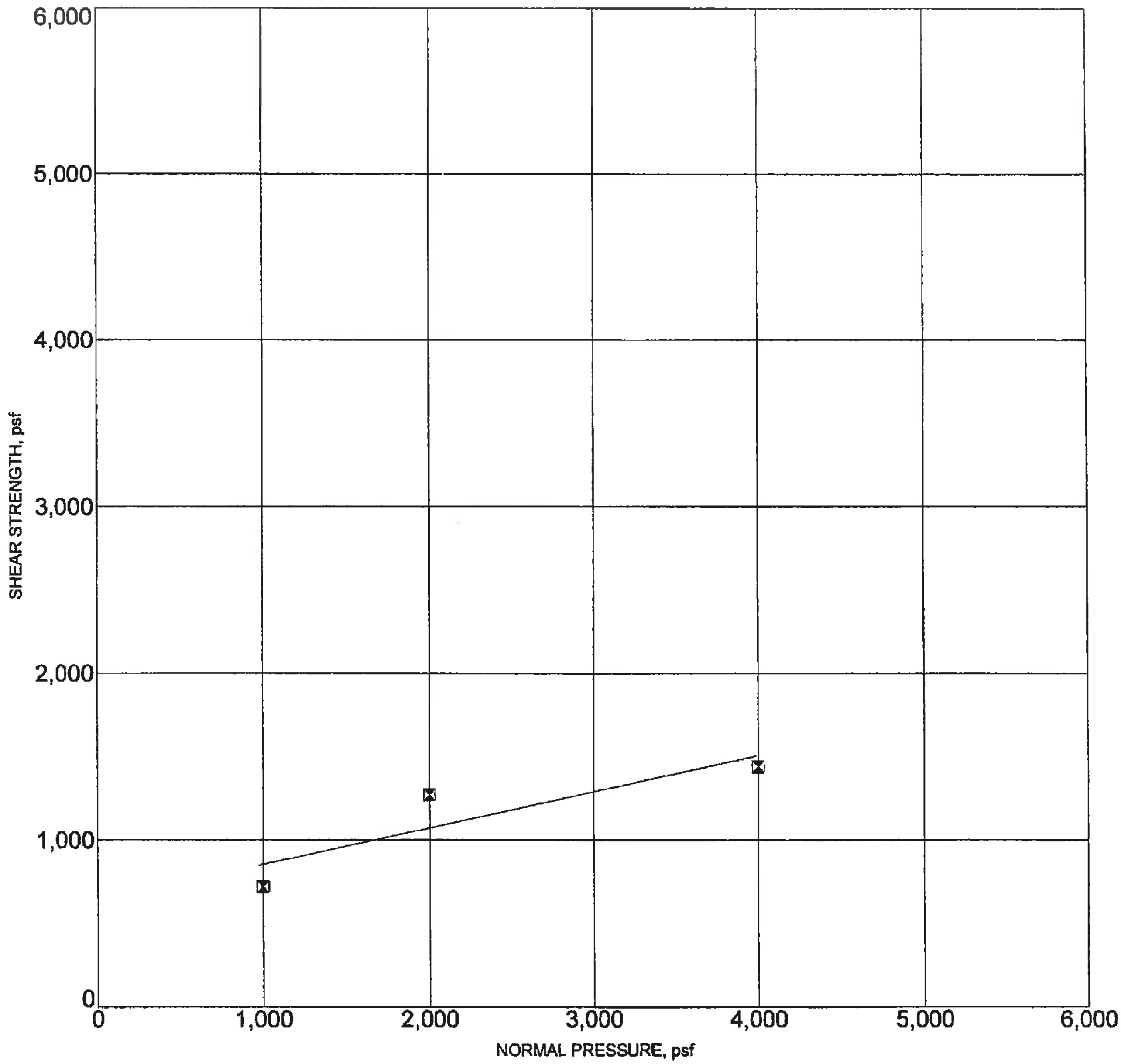
PROJECT: HARVARD-WESTLAKE

PROJECT NO.: 2270.I

GPI

DIRECT SHEAR TEST RESULTS

FIGURE B-5



RESIDUAL STRENGTH
Friction Angle= 12 degrees
Cohesion= 636 psf

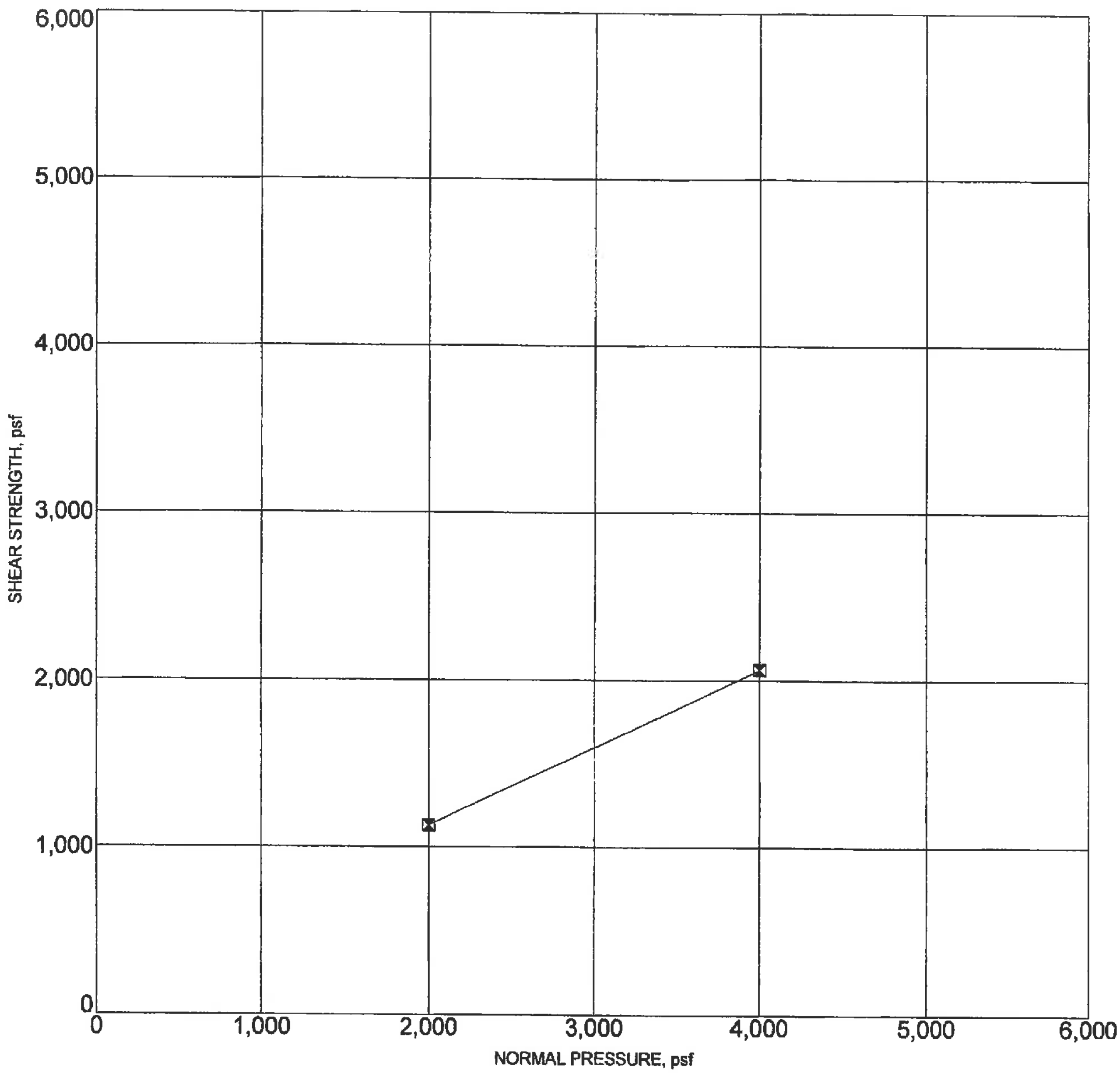
Sample Location		Classification	DD,pcf	MC,%
B-2	25.0	SILTSTONE	77	32.7

PROJECT: HARVARD-WESTLAKE PROJECT NO.: 2270.I

GPI

DIRECT SHEAR TEST RESULTS

FIGURE B-6



RESIDUAL STRENGTH
Friction Angle= 25 degrees
Cohesion= 192 psf

Sample Location		Classification	DD,pcf	MC,%
B-3	20.0	SILTSTONE	87	28.4

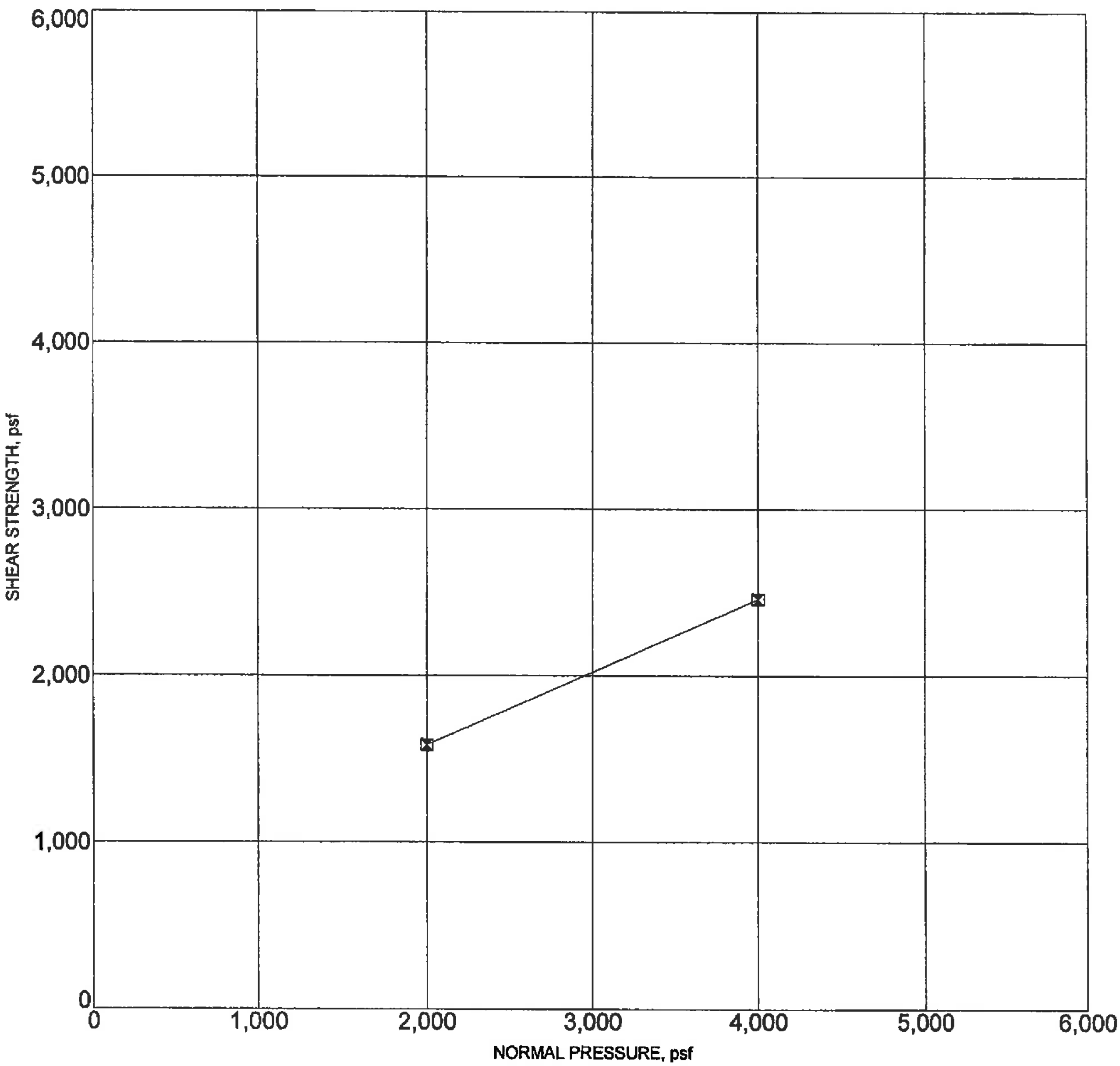
PROJECT: HARVARD-WESTLAKE

PROJECT NO.: 2270.I

GPI

DIRECT SHEAR TEST RESULTS

FIGURE B-7



RESIDUAL STRENGTH
Friction Angle= 24 degrees
Cohesion= 708 psf

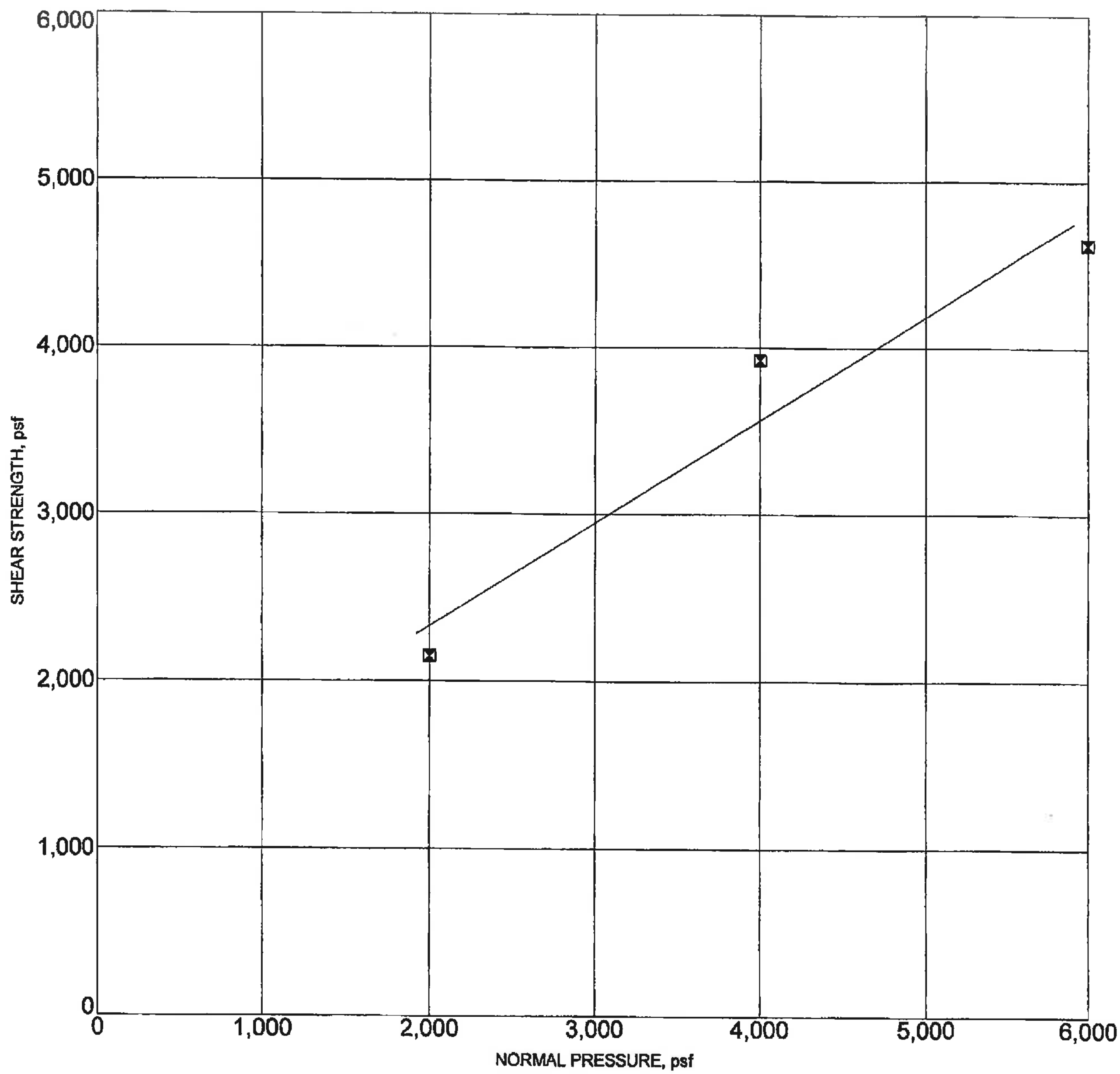
Sample Location		Classification	DD,pcf	MC,%
B-4	20.0	SILTSTONE	55	67.7

PROJECT: HARVARD-WESTLAKE PROJECT NO.: 2270.I

GPI

DIRECT SHEAR TEST RESULTS

FIGURE B-8



RESIDUAL STRENGTH
Friction Angle= 32 degrees
Cohesion= 1092 psf

Sample Location		Classification	DD,pcf	MC,%
B-4	65.0	SILTSTONE	57	63.0

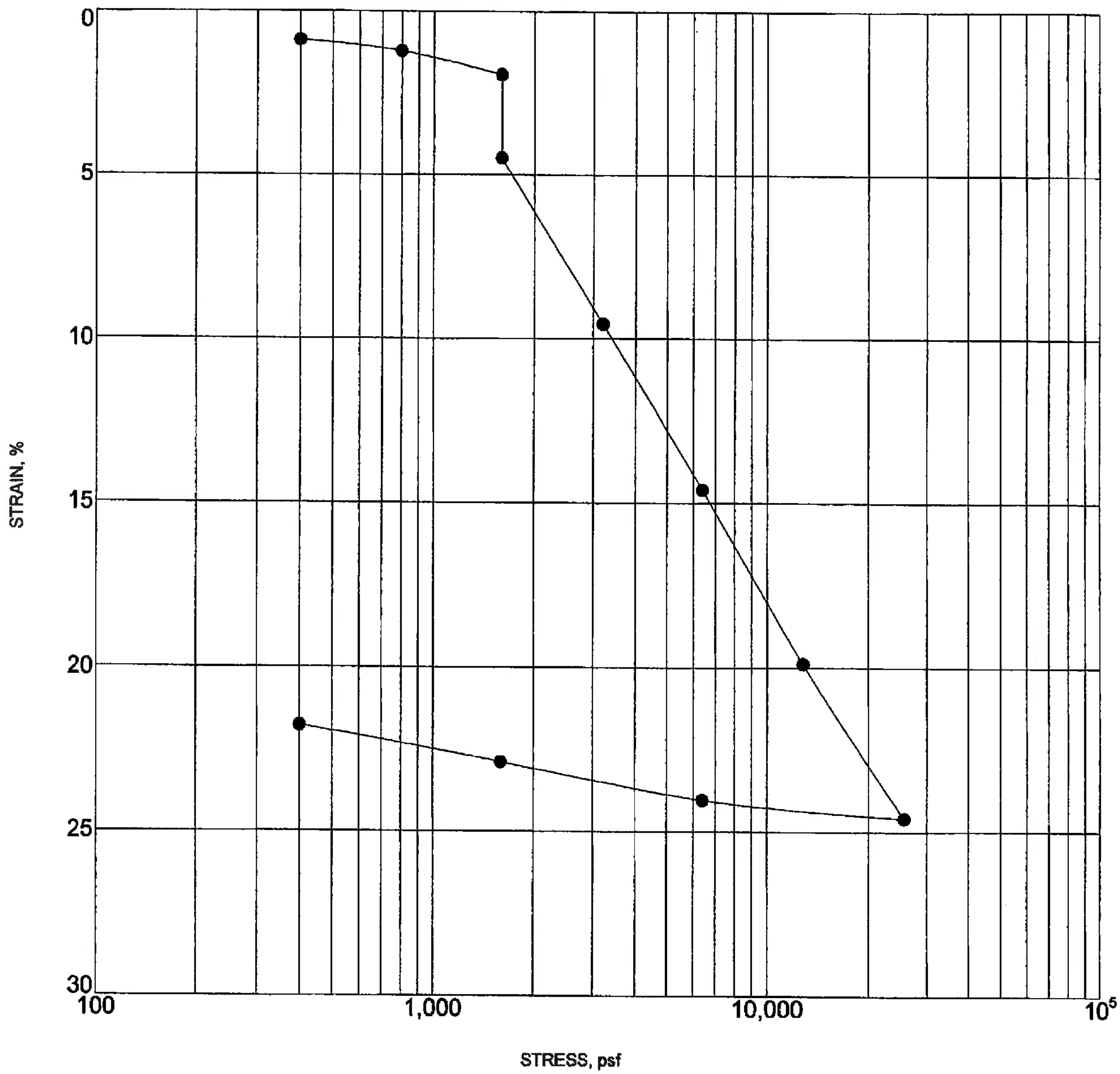
PROJECT: HARVARD-WESTLAKE

PROJECT NO.: 2270.I

GPI

DIRECT SHEAR TEST RESULTS

FIGURE B-9



Sample inundated at 1600 psf

	Sample Location		Classification	DD,pcf	MC,%
●	B-2	10.0	SANDY SILT (MH)	64	20.4

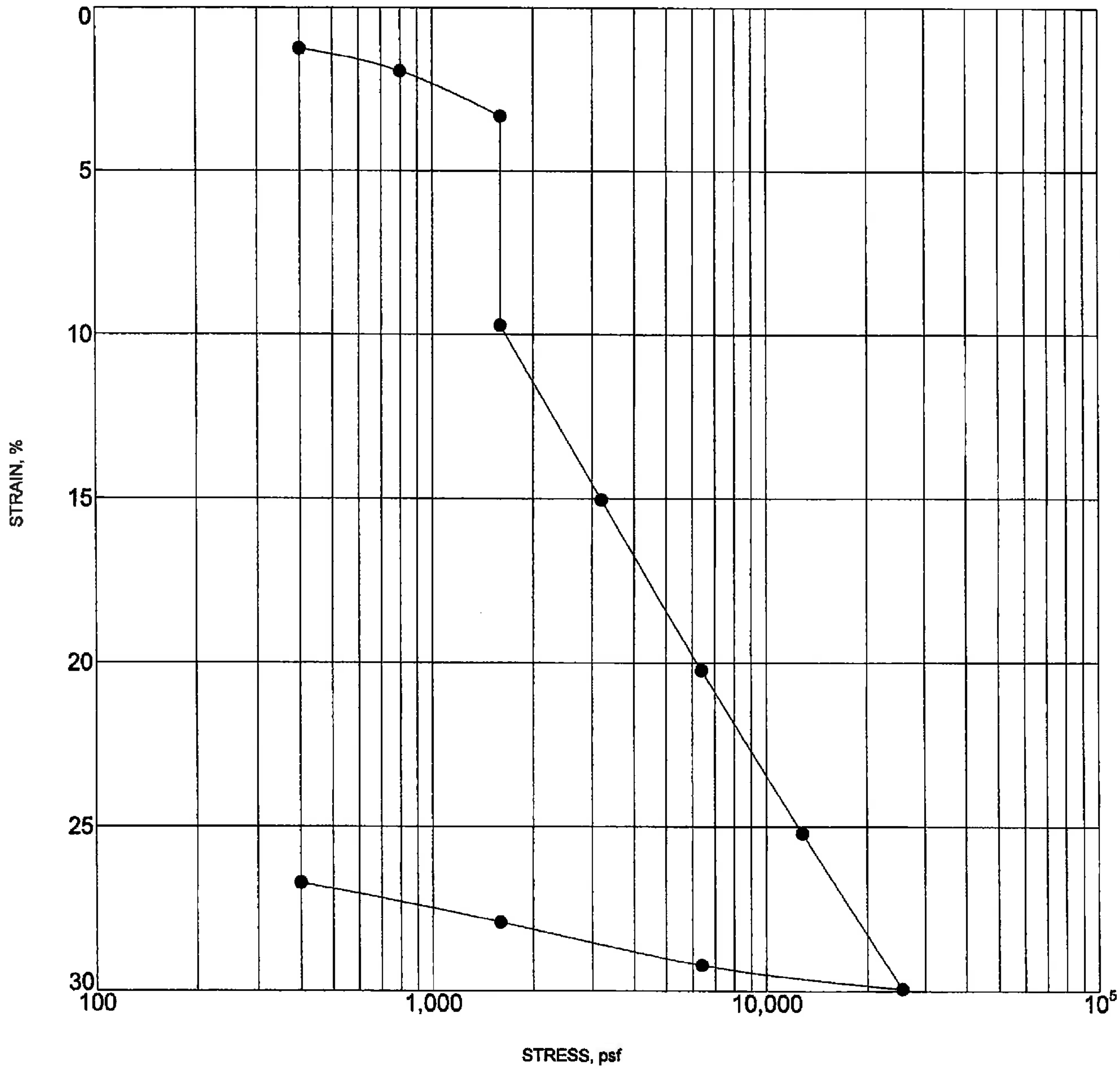
PROJECT: HARVARD-WESTLAKE

PROJECT NO.: 2270.I

GPI

CONSOLIDATION TEST

FIGURE B-10



Sample inundated at 1600 psf

	Sample Location		Classification	DD,pcf	MC,%
●	B-2	20.0	SILT (MH)	63	22.8

PROJECT: HARVARD-WESTLAKE

PROJECT NO.: 2270.I

GPI

CONSOLIDATION TEST

FIGURE B-11

SCHIFF ASSOCIATES
FIFTY YEARS OF PROFESSIONALISM

www.schiffassociates.com
Consulting Corrosion Engineers – Since 1959

Table 1 - Laboratory Tests on Soil Samples

Geotechnical Professionals Inc.
IDG Harvard
Your #2270.I, SA #09-1019LAB
30-Nov-09

Sample ID			B2 @ 5-10'	B3 @ 5-15'
Resistivity		**Units**		
as-received		ohm-cm	3,600	19,600
saturated		ohm-cm	600	760
pH			7.0	7.3
Electrical Conductivity		mS/cm	3.22	0.80
Chemical Analyses				
Cations				
calcium	Ca^{2+}	mg/kg	3,590	649
magnesium	Mg^{2+}	mg/kg	636	54
sodium	Na^{1+}	mg/kg	588	113
potassium	K^{1+}	mg/kg	86	99
Anions				
carbonate	CO_3^{2-}	mg/kg	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	1,135	522
flouride	F^{1-}	mg/kg	3.4	11
chloride	Cl^{1-}	mg/kg	55	264
sulfate	SO_4^{2-}	mg/kg	5,220	1,080
phosphate	PO_4^{3-}	mg/kg	ND	ND
Other Tests				
ammonium	NH_4^{1+}	mg/kg	42	15
nitrate	NO_3^{1-}	mg/kg	104	16
sulfide	S^{2-}	qual	na	na
Redox		mV	na	na

Electrical conductivity in millisiemens/cm and chemical analysis were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed

431 West Baseline Road · Claremont, CA 91711
Phone: 909.626.0967 · Fax: 909.626.3316

BYER GEOTECHNICAL, INC.

December 30, 2010
BG 21256

Harvard-Westlake School
700 North Faring Road
Los Angeles, California 90077

Attention: Mr. Jim DeMatte

Subject

Transmittal of Geologic and Soils Engineering Exploration
Proposed Brendon Kutler Center and Mudd Library Renovation
Arb. 1, Portion of Lot 1111, Tract 1000
3700 North Coldwater Canyon Avenue
North Hollywood, California

Gentlepersons:

Byer Geotechnical has completed our report dated December 30, 2010, which describes the geologic and soils engineering conditions with respect to the proposed project. The reviewing agency for this document is City of Los Angeles, Department of Building and Safety (LADBS). The reviewing agency requires three unbound copies, one with a wet signature, an application form, and a filing fee. Copies of the report have been distributed as follows:

(1) Addressee (E-mail and Mail)
(4) Tobias Architecture (E-mail and Mail)
(1) John A. Martin & Associates, Attention: Kurt Clandening (E-mail)

It is our understanding that Tobias Architecture will file the report with the LADBS. Please review the report carefully prior to submittal to the governmental agency. Questions concerning the report should be directed to the project consultant. Byer Geotechnical appreciates the opportunity to offer our consultation and advice on this project.

Very truly yours,
BYER GEOTECHNICAL, INC.

James E. Tucker
Project Consultant

BYER GEOTECHNICAL, INC.

GEOLOGIC AND SOILS ENGINEERING EXPLORATION

PROPOSED BRENDON KUTLER CENTER AND MUDD LIBRARY RENOVATION

ARB. 1, PORTION OF LOT 1111, TRACT 1000

3700 NORTH COLDWATER CANYON AVENUE

NORTH HOLLYWOOD, CALIFORNIA

FOR HARVARD-WESTLAKE SCHOOL

BYER GEOTECHNICAL, INC., PROJECT NUMBER BG 21256

DECEMBER 30, 2010

GEOLOGIC AND SOILS ENGINEERING EXPLORATION
PROPOSED BRENDON KUTLER CENTER AND MUDD LIBRARY RENOVATION
ARB. 1, PORTION OF LOT 1111, TRACT 1000
3700 NORTH COLDWATER CANYON AVENUE
NORTH HOLLYWOOD, CALIFORNIA
FOR HARVARD-WESTLAKE SCHOOL
BYER GEOTECHNICAL, INC., PROJECT NUMBER BG 21256
DECEMBER 30, 2010

INTRODUCTION

This report has been prepared per our signed Agreement and summarizes findings of Byer Geotechnical, Inc., geologic and soils engineering exploration performed on a portion of the site. The purpose of this study is to evaluate the nature, distribution, engineering properties, relative stability, and geologic structure of the earth materials underlying the site with respect to construction of the proposed Brendon Kutler Center and the Mudd Library renovation. This report is intended to assist in the design and completion of the proposed project and to reduce geotechnical risks that may affect the project. The professional opinions and advice presented in this report are based upon commonly accepted exploration standards and are subject to the AGREEMENT with TERMS AND CONDITIONS, and the GENERAL CONDITIONS AND NOTICE section of this report. No warranty is expressed or implied by the issuing of this report.

PROPOSED PROJECT

The scope of the proposed project was determined from the preliminary plans prepared by Tobias Architecture. Building plans have not been prepared and await the conclusions and recommendations of this report. The project consists of construction of a two-story building in the narrow area between the Mudd Library and Seaver Academic Center buildings. Also, the Mudd Library will be renovated.

EXPLORATION

The scope of the field exploration was determined from our initial site visit and consultation with Jim DeMatte of Harvard-Westlake School. The preliminary plans prepared by Tobias Architecture were a guide to our work on this project. Exploration was conducted using techniques normally applied to this type of project in this setting. This report is limited to the area of the exploration and the proposed project, as shown on the Geologic Map and cross section. The scope of this exploration did not include an assessment of general site environmental conditions for the presence of contaminants in the earth materials and groundwater. Conditions affecting portions of the property outside the area explored are beyond the scope of this report.

Exploration was conducted on November 4 and 24, 2010, with the aid of a bucket-auger drill rig and hand labor. It included excavating three test pits and drilling one boring to depths of 6 to 50 feet. Samples of the earth materials were obtained and delivered to our soils engineering laboratory for testing and analysis. The boring and test pits were visually downhole logged by the project geologist. The boring and test pits were backfilled and tamped, but should not be considered compacted.

Office tasks included laboratory testing of selected soil samples, review of published maps and photos for the area, review of our files, review of agency files, preparation of a cross section, preparation of the Geologic Map, slope stability calculations, engineering analysis, and preparation of this report. Earth materials exposed in the test pits and boring are described on the enclosed Log of Test Pits and Log of Boring. Appendix I contains a discussion of the laboratory testing procedures and results.

The proposed project, surface geologic conditions, and the locations of the test pits and boring are shown on the Geologic Map. Subsurface distribution of the earth materials, projected geologic structure, and the proposed project are shown on Section A. Section A forms the basis for the slope stability calculations.

RESEARCH - PRIOR WORK

Agency records contain the following geotechnical reports, which were prepared for the buildings in the area of the proposed project.

Geology and Soils Consultants, Inc.:

Geologic Engineering Investigation, Proposed Library and Field House, 3700 Coldwater Canyon Avenue, Los Angeles, California, dated January 29, 1973.

Epsilon Engineering & Inspection, Inc.:

Report of Preliminary Soil Investigation, Harvard School, Gallery Basement, 3700 Coldwater Canyon, North Hollywood, California, dated March 14, 1997; and

Report of Preliminary Soil Investigation for Harvard School, Upper Level, 3700 Coldwater Canyon, North Hollywood, California, dated January 10, 1991.

Converse Consultants West:

Geotechnical Investigation, Proposed Science Building, Harvard Westlake School, 3700 Coldwater Canyon Avenue, Studio City, California, dated April 22, 1994.

LeRoy Crandall and Associates:

Inspection of Caisson Excavations, Proposed Subsurface Drainage System, Inspection of Foundation Excavations and Inspection and Testing of Compacted Fill, Proposed Lower School Building - Harvard School, 3700 Coldwater Canyon Drive, Los Angeles, California, dated November 17, 1967;

Placement of Rock Backfill, Academic Center, Harvard School, 3700 Coldwater Canyon Avenue, Los Angeles, California, dated December 29, 1969; and

Review of Foundation Recommendations, Proposed Academic Center, 3700 Coldwater Canyon Drive, Los Angeles, California, dated August 12, 1968.

The J. Byer Group, Inc. (JB 17866-B):

Geologic and Soils Engineering Exploration, Proposed Parking Lot Extension and Gymnasium Addition, Portion of Lot 1111, Tract 1000, 3700 Coldwater Canyon Avenue, North Hollywood, California, dated October 16, 1998; and

Geotechnical Engineering Exploration, Proposed Sports-Field Lighting, Lot 1111, Tract 1000, 3700 Coldwater Canyon Avenue, Studio City, California, dated October 9, 2006.

City of Los Angeles, Department of Building and Safety, conditional approval letter, Log # 27150, dated March 18, 1999, and Geology/Soil Report Approval Letter, Log # 56969, dated January 23, 2007.

The data contained in these reports was reviewed and considered as part of our work on this project.

SITE DESCRIPTION

The subject property consists of a partially-graded hillside parcel on the north flank of the Santa Monica Mountains, in the North Hollywood section of the city of Los Angeles, California (34.1406° N Latitude, 118.4118° W Longitude). It is located south of Ventura Boulevard and just east of Coldwater Canyon Avenue. The site is developed with several single- and multi-story school buildings, an athletic field, an olympic swimming pool, and paved parking lots. Slopes in the area of the proposed project include a 1½:1 to 2:1 slope, which ascends to the south of the access road (south of the Seaver and Mudd Buildings) approximately 110 feet, to Avenida Del Sol. North of the project site, the grade slopes gently to the north. Grade changes to the north are supported by retaining walls and access is via stairs. Past grading on the site has consisted of cut-and-fill operations during site development.

Vegetation on the site is limited to planter areas. Surface drainage is by sheetflow runoff down the contours of the land to the north and collected in area drains.

GROUNDWATER

In *Seismic Hazard Zone Report for the Van Nuys 7.5-Minute Quadrangle, Log Angeles County, California,* 1997, the California Geological Survey (CGS) has estimated the historically-highest groundwater level in the west portion of the site was between 10 and 40 feet. Groundwater was not encountered in the boring to a depth of 50 feet. Seasonal fluctuations in groundwater levels occur due to variations in climate, irrigation, development, and other factors not evident at the time of the exploration. Groundwater levels may also differ across the site. Groundwater can saturate earth materials causing subsidence or instability of slopes.

METHANE ZONES

City of Los Angeles Ordinance No. 175790 established methane mitigation requirements and includes construction standards to control methane intrusion into buildings. The proposed project is not located within a Methane Zone or Methane Buffer Zone.

EARTH MATERIALS

Compacted Fill

Compacted fill, associated with previous site grading, underlies the area of the proposed project to a maximum observed depth of four feet in the test pits. Greater depths of fill may occur. The fill consists of gravelly silt, which is black, dark gray-brown, brown, moist, firm to very firm, with some rock, concrete, and brick fragments up to 10-inches in diameter, and some clay.

Ancient Slide Debris

Ancient slide debris underlies the area of the proposed project and was encountered in the boring and test pits. The ancient slide debris was observed to be 20 feet thick in the boring. The ancient slide debris consists of diatomaceous siltstone, shale, and fine-grained sandstone, which is gray-brown, brown, gray, tan, fractured to very fractured, moderately hard, with some soil-filled fractures up to one inch.

Bedrock

Bedrock underlying the site at depth and encountered in the boring consists of siltstone, shale, and sandstone mapped as part of the Modelo Formation on the City of Los Angeles, Preliminary Geologic Maps, Sheet 70. The bedrock is also exposed in cut slopes on the southwest portion of the site. The bedrock consists of diatomaceous siltstone, shale, and fine sandstone that is light gray to gray, tan, brown, moderately hard to very hard, and well bedded.

GEOLOGIC STRUCTURE

The bedrock described above is common to this area of the eastern Santa Monica Mountains and the geologic structure is consistent with regional trends. The geologic structure of the bedrock is favorably oriented for stability of the site and proposed project. Bedding planes mapped on and adjacent to the site generally strike northeast and dip shallowly to steeply to the north. An east-west-striking fault has been mapped south of the project area (Regional Geologic Map), which forms a boundary with steep dips to the south and shallow dips to the north.

An ancient landslide mass underlies the area of the proposed project, was observed in the boring, and is described in the referenced reports by Geology and Soils Consultants, Inc., and LeRoy Crandall and Associates. The limits of the ancient landslide mass, initially reported by LeRoy Crandall, are shown on the Geologic Map. The slide mass is also shown on Section A.

GENERAL SEISMIC CONSIDERATIONS

The subject property is located in an active seismic region. Moderate to strong earthquakes can occur on numerous local faults. The United States Geological Survey, California Geological Survey (CGS), private consultants, and universities have been studying earthquakes in southern California for several decades. Early studies were directed toward earthquake prediction and estimation of the effects of strong ground shaking. Studies indicate that earthquake prediction is not practical and not sufficiently accurate to benefit the general public. Governmental agencies now require earthquake resistant structures. The purpose of the code seismic-design parameters is to prevent collapse during strong ground shaking. Cosmetic damage should be expected.

Southern California faults are classified as "active" or "potentially active." Faults from past geologic periods of mountain building that do not display evidence of recent offset are considered "potentially active." Faults that have historically produced earthquakes or show evidence of movement within the past 11,000 years are known as "active faults." No known active faults cross the subject property.

The following table lists the applicable seismic coefficients for the project for the 2007 City of Los Angeles Building Code:

SEISMIC COEFFICIENTS (2008 City of Los Angeles Building Code)		
Latitude = 34.1406° N Longitude = 118.4118° W	Short Period (0.2s)	One-Second Period
Earth Materials and Site Class from Table 1613.5.2 and Section 1613.5.2	Bedrock - C	
Spectral Accelerations from Figures 1613.5(3) and 1613.5(4) and USGS	S_s = 1.500 (g)	S_1 = 0.600 (g)
Site Coefficients from Tables 1613.5.3 (1) and 1613.5.3 (2) and USGS	F_A = 1.0	F_V = 1.3
Spectral Response Accelerations from Equations 16-37 and 16-38	S_{MS} = 1.500 (g)	S_{M1} = 0.780 (g)
Design Accelerations from Equations 16-39 and 16-40	S_{DS} = 1.000 (g)	S_{D1} = 0.520 (g)

Reference: U.S. Geological Survey, **Earthquake Hazards Program, Seismic Design Values for Buildings,** http://earthquake.usgs.gov/hazards/design/buildings.php

The mapped spectral response acceleration parameter for the site for a 1-second period (S_1) is less than 0.75g. The design spectral response acceleration parameters for the site for a 1-second period

(S_{D1}) is greater than or equal to 0.20g, and/or the short period (S_{DS}) is greater than or equal to 0.50g. Therefore, the project is considered to be in Seismic Design Category D.

The principal seismic hazard to the proposed project is strong ground shaking from earthquakes produced by local faults. Modern, well-constructed buildings are designed to resist ground shaking through the use of shear panels, moment frames, and reinforcement. Additional precautions may be taken, including strapping water heaters and securing furniture to walls and floors. It is likely that the subject property will be shaken by future earthquakes produced in southern California.

Ground Motion

In *Seismic Hazard Zone Report for the Van Nuys 7.5-Minute Quadrangle, Los Angeles County, California,* 1997, the California Geological Survey (CGS) has assigned ground motion values for this area of North Hollywood. The Design Basis Ground Motion (10 percent exceedance in 50 years) is a peak ground acceleration (PGA) of 0.50 and an earthquake with moment magnitude (M_w) of 6.4. These ground motions could occur at the site during the life of the project.

Liquefaction

The CGS has not mapped the project site within an area where historic occurrence of liquefaction or geological, geotechnical, and groundwater conditions indicate a potential for permanent ground displacement such that mitigation as defined in Public Resources Code Section 2693 (c) would be required.

The subject property is underlain by ancient landslide debris and bedrock, which are not subject to liquefaction.

SLOPE STABILITY

Gross Stability

The CGS has not designated the property within a state zone requiring seismic landslide investigation per Public Resources Code, Section 2693 (c). The 110-foot-high, 1½:1 to 2:1 natural slope has been analyzed for stability. As part of the analysis, the stability of the ancient landslide was determined. A computerized version of Bishop's Simplified Method (*Slide 6.0*, Rocscience, Inc.) was utilized.

The analysis shows that the existing slope is grossly stable with a factor of safety in excess of 1.5. The calculations use the shear tests of samples believed to be representative of the strength of the ancient landslide debris and bedrock encountered during exploration. The cross sections and geologic structure used are the most critical for the slopes analyzed.

The ancient landslide debris has been determined to be grossly stable per the enclosed calculations based on Section A. In addition, the existing buildings (Seaver Academic Center and Mudd Library) surrounding walkways, decking, and walls were observed to be performing well, and did not show signs of distress.

CONCLUSIONS AND RECOMMENDATIONS

General Findings

The conclusions and recommendations of this exploration are based upon review of the preliminary plans, review of published maps, one boring, three test pits, field geologic mapping, research of available records, laboratory testing, engineering analysis, and years of experience performing similar studies on similar sites. It is the finding of Byer Geotechnical, Inc., that development of the proposed project is feasible from a geologic and soils engineering/geotechnical engineering standpoint provided the advice and recommendations contained in this report are included in the plans and are implemented during construction.

The Foundation Plan for Mudd Library indicates that the existing foundation consists of continuous and pad footings. The referenced report by Geology and Soils Consultants, Inc., dated January 29, 1973, recommended that the Mudd Library foundations be into the ancient slide debris. The report determined the ancient slide debris to be grossly stable.

The recommended bearing material is the ancient landslide debris. This will require foundations to be deepened through the existing fill, which is approximately three to four feet thick.

FOUNDATION DESIGN

Spread Footings

Continuous and/or pad footings may be used to support the proposed Brendon Kutler Center building and renovations to the Mudd Library, provided they are founded in firm ancient landslide debris. This will require deepening the foundations through the existing fill. Continuous footings should be a minimum of 12 inches in width. Pad footings should be a minimum of 24-inches square. The following chart contains the recommended design parameters.

Bearing Material	Minimum Embedment Depth of Footing (Inches)	Vertical Bearing (psf)	Coefficient of Friction	Passive Earth Pressure (pcf)	Maximum Earth Pressure (psf)
Ancient Landslide Debris	12	2,000	0.30	300	4,000

Increases in the bearing value are allowable at a rate of 400 pounds-per-square-foot for each additional foot of footing width or depth to a maximum of 4,000 pounds-per-square-foot. For bearing calculations, the weight of the concrete in the footing may be neglected.

The bearing values shown above is for the total of dead and frequently applied live loads and may be increased by one-third for short duration loading, which includes the effects of wind or seismic forces. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

Footings adjacent to retaining walls should be deepened below a 1:1 plane from the bottom of the lower retaining wall, or the footings should be designed as grade beams to bridge from the wall to the 1:1 plane.

All continuous footings should be reinforced with a minimum of four #4 steel bars: two placed near the top, and two near the bottom of the footings. Footings should be cleaned of all loose soil, moistened, free of shrinkage cracks, and approved by the geologist prior to placing forms, steel, or concrete.

Deepened Foundations - Friction Piles

As an alternative to conventional foundations, a friction pile grade-beam foundation system may be utilized. The structural engineer may design piles that are deeper or larger in diameter depending on final loads. Piles should be a minimum of 24 inches in diameter and a minimum of eight feet into ancient landslide debris. Piles may be assumed fixed at three feet into ancient landslide debris. The piles may be designed for a skin friction of 500 pounds-per-square-foot for that portion of pile in contact with the ancient landslide debris. All piles should be tied in two horizontal directions with grade beams.

Lateral Design

The friction value is for the total of dead and frequently applied live loads and may be increased by one-third for short duration loading, which includes the effects of wind or seismic forces. Resistance to lateral loading may be provided by passive earth pressure within the ancient landslide debris.

Passive earth pressure may be computed as an equivalent fluid having a density of 300 pounds-per-cubic-foot. The maximum allowable earth pressure is 4,000 pounds-per-square-foot. For design of isolated piles, the allowable passive and maximum earth pressures may be increased by 100 percent. Piles spaced more than 2½-pile diameters on center may be considered isolated.

Foundation Settlement

Settlement of the foundation system is expected to occur on initial application of loading. A total settlement of one-fourth to one-half of an inch may be anticipated. Differential settlement should not exceed one-fourth of an inch.

FLOOR SLABS

The slabs should be designed to bridge between the foundations. Slabs that will be provided with a floor covering should be protected by a polyethylene plastic vapor barrier. The barrier should be sandwiched between the layers of sand, about two inches each, to prevent punctures and aid in the concrete cure. A low-slump concrete may be used to minimize possible curling of the slab. The concrete should be allowed to cure properly before placing vinyl or other moisture sensitive floor covering.

It should be noted that cracking of concrete slabs is common. The cracking occurs because concrete shrinks as it cures. Control joints, which are commonly used in exterior decking to control such cracking, are normally not used in interior slabs. The reinforcement recommended above is intended to reduce cracking and its proper placement is critical to the performance of the slab. The minor shrinkage cracks, which often form in interior slabs, generally do not present a problem when carpeting, linoleum, or wood floor coverings are used. The slab cracks can, however, lead to surface cracks in brittle floor coverings such as ceramic tile.

DRAINAGE

Control of site drainage is important for the performance of the proposed project. Pad and roof drainage should be collected and transferred to the street in non-erosive drainage devices. Drainage should not be allowed to pond on the pad or against any foundation or retaining wall. Drainage should not be allowed to flow uncontrolled over any descending slope. Planters located within retaining wall backfill should be sealed to prevent moisture intrusion into the backfill. Planters located next to raised-floor-type construction also should be sealed to the depth of the footings. Drainage control devices require periodic cleaning, testing, and maintenance to remain effective.

Irrigation

Control of irrigation water is a necessary part of site maintenance. Soggy ground and perched water may result if irrigation water is excessively applied. Irrigation systems should be adjusted to provide the minimum water needed. Adjustments should be made for changes in climate and rainfall.

PLAN REVIEW

Formal plans ready for submittal to the building department should be reviewed by Byer Geotechnical. Any change in scope of the project may require additional work.

SITE OBSERVATIONS DURING CONSTRUCTION

The building department requires that the geotechnical engineer provide site observations during grading and construction. Foundation excavations should be observed and approved by the geotechnical engineer or geologist prior to placing steel, forms, or concrete. All fill that is placed should be approved by the geotechnical engineer and the building department prior to use for support of structural footings and floor slabs.

Please advise Byer Geotechnical, Inc., at least 24 hours prior to any required site visit. The building department stamped plans, the permits, and the geotechnical reports should be at the job site and available to our representative. The project consultant will perform the observation and post a notice at the job site with the findings. This notice should be given to the agency inspector.

FINAL REPORTS

The geotechnical engineer will prepare interim and final compaction reports upon request. The geologist will prepare reports summarizing pile excavations.

CONSTRUCTION SITE MAINTENANCE

It is the responsibility of the contractor to maintain a safe construction site. The area should be fenced and warning signs posted. All excavations must be covered and secured. Soil generated by foundation excavations should be either removed from the site or placed as compacted fill. Soil should not be spilled over any descending slope. Workers should not be allowed to enter any unshored trench excavations over five feet deep. Water shall not be allowed to saturate open footing trenches.

GENERAL CONDITIONS AND NOTICE

This report and the exploration are subject to the following conditions. Please read this section carefully; it limits our liability.

In the event of any changes in the design or location of any structure, as outlined in this report, the conclusions and recommendations contained herein may not be considered valid unless the changes are reviewed by Byer Geotechnical, Inc., and the conclusions and recommendations are modified or reaffirmed after such review.

The subsurface conditions, excavation characteristics, and geologic structure described herein and shown on the cross section have been projected from test excavations on the site and may not reflect any variations that occur between these test excavations or that may result from changes in subsurface conditions.

Fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, irrigation, and other factors not evident at the time of the measurements reported herein. Fluctuations also may occur across the site. High groundwater levels can be extremely hazardous. Saturation of earth materials can cause subsidence or slippage of the site.

If conditions encountered during construction appear to differ from those disclosed herein, notify us immediately so we may consider the need for modifications. Compliance with the design concepts, specifications, and recommendations requires the review of the engineering geologist and geotechnical engineer during the course of construction.

THE EXPLORATION WAS PERFORMED ONLY ON A PORTION OF THE SITE, AND CANNOT BE CONSIDERED AS INDICATIVE OF THE PORTIONS OF THE SITE NOT EXPLORED.

This report, issued and made for the sole use and benefit of the client, is not transferable. Any liability in connection herewith shall not exceed the Phase I fee for the exploration and report or a negotiated fee per the Agreement. No warranty is expressed, implied, or intended in connection with the exploration performed or by the furnishing of this report.

THIS REPORT WAS PREPARED ON THE BASIS OF THE PRELIMINARY DEVELOPMENT PLAN FURNISHED. FINAL PLANS SHOULD BE REVIEWED BY THIS OFFICE AS ADDITIONAL GEOTECHNICAL WORK MAY BE REQUIRED.

Byer Geotechnical appreciates the opportunity to provide our service on this project. Any questions concerning the data or interpretation of this report should be directed to the undersigned.

Respectfully submitted,
BYER GEOTECHNICAL, INC.

James E. Tucker
James E. Tucker
P. G. 6628

Robert I. Zweigler
E. G. 1210/G. E. 2120

REGISTERED PROFESSIONAL ENGINEER
ROBERT I. ZWEIGLER
No. 2120
Exp. 06-30-12
GEOTECHNICAL
STATE OF CALIFORNIA

JET:RIZ:mh
S:\FINAL\BG\21256_Harvard-Westlake\21256_Harvard-Westlake_Geo_and_Soils.wpd

Enc: Appendix I - Laboratory Testing
Shear Test Diagrams (2 Pages)
Regional Geologic Map
Log of Test Pits 1 - 3
Log of Boring 1 (3 Pages)
Calculation Sheets (11 Pages)
Vicinity Map
Location Map
Section A

In Pocket: Geologic Map

xc: (1) Addressee (E-mail and Mail)
(4) Tobias Architecture (E-mail and Mail)
(1) John A. Martin & Associates, Attention: Kurt Clandening (E-mail)

APPENDIX I

LABORATORY TESTING

Undisturbed and bulk samples of the fill, ancient landslide debris, and bedrock were obtained from the test pits and boring and transported to the laboratory for testing and analysis. The samples were obtained by driving a ring-lined, barrel sampler conforming to ASTM D 3550-01 with successive drops of the sampler. Experience has shown that sampling causes some disturbance of the sample. However, the test results remain within a reasonable range. The samples were retained in brass rings of 2.50 inches outside diameter and 1.00 inches in height. The samples were stored in close fitting, waterproof containers for transportation to the laboratory.

Moisture-Density

The dry density of the samples was determined using the procedures outlined in ASTM D 2937-04. The moisture content of the samples was determined using the procedures outlined in ASTM D 2216-05. The results are shown on the Log of Test Pits and Log of Boring.

Shear Tests

Shear tests were performed on samples of the ancient landslide debris and bedrock using the procedures outlined in ASTM D 3080-04 and a strain controlled, direct-shear machine manufactured by Soil Test, Inc. The rate of deformation was 0.005 (for the ancient landslide debris) and 0.025 (for the bedrock) inches per minute. The ancient landslide debris was repeatedly sheared. The samples were tested in an artificially saturated condition. Following the shear test, the moisture content of the samples was determined to verify saturation. The results are plotted on the Shear Test Diagrams.

BYER GEOTECHNICAL, INC.
1461 E. CHEVY CHASE DR., SUITE 200
GLENDALE, CA 91206
818.549.9959 TEL
818.543.3747 FAX

SHEAR DIAGRAM #1

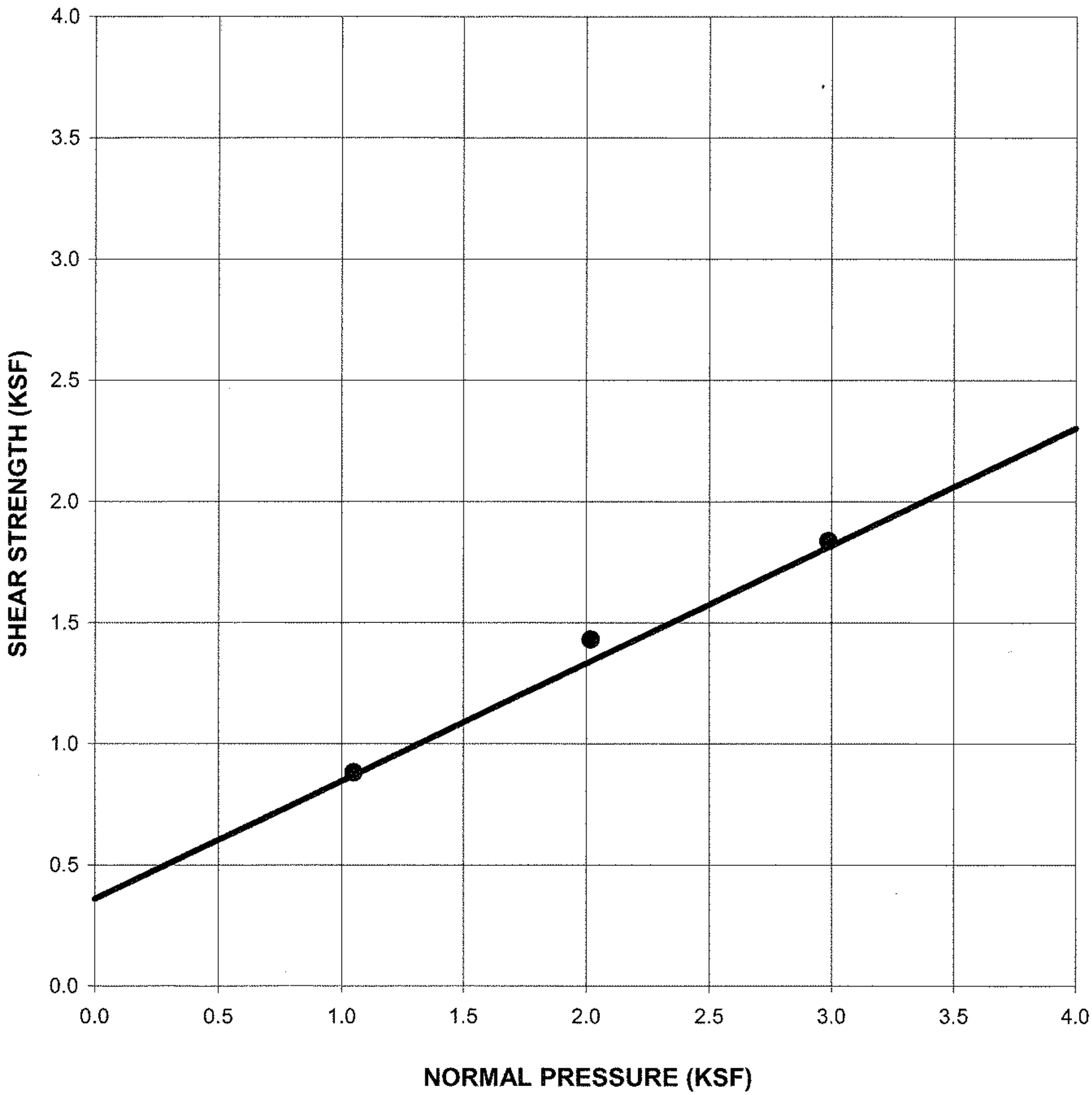
BG: 21256 CONSULTANT: JET
CLIENT: HARVARD WESTLAKE SCHOOL

EARTH MATERIAL: ANCIENT SLIDE DEBRIS

SAMPLE B1-15' REPEATEDLY SHEARED AT 0.005 INCHES PER MINUTE.

			B1-15'
Phi Angle =	**26 degrees**	**Moisture Content**	**70.5%**
Cohesion =	**360 psf**	**Dry Density (pcf)**	**55.8**
		Percent Saturation	**95.2%**

DIRECT SHEAR TEST - ASTM D-3080 (RESIDUAL VALUES)



BYER GEOTECHNICAL, INC.
1461 E. CHEVY CHASE DR., SUITE 200
GLENDALE, CA 91206
818.549.9959 TEL
818.543.3747 FAX

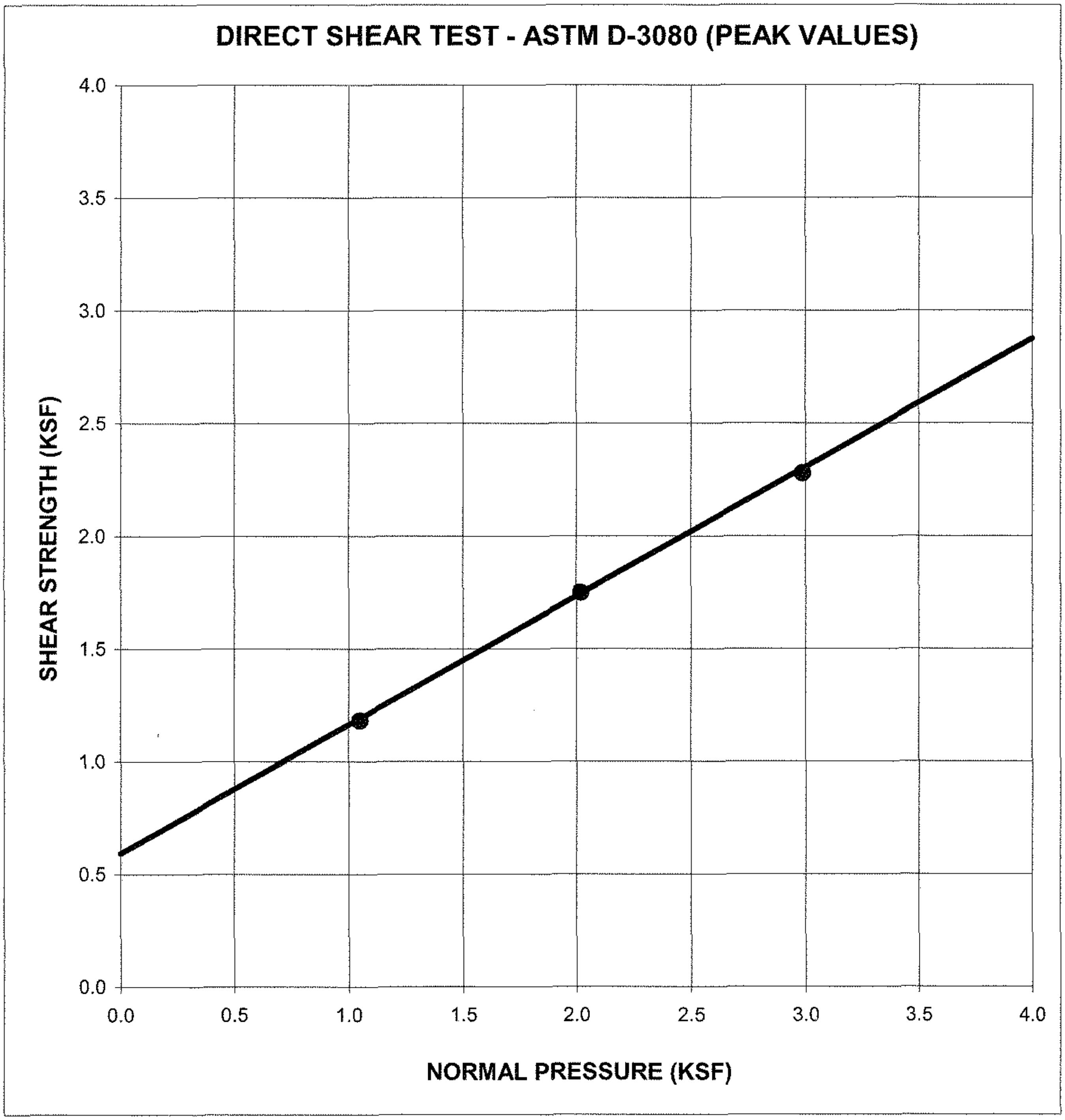
SHEAR DIAGRAM #2

BG: 21256 CONSULTANT: JET
CLIENT: HARVARD WESTLAKE SCHOOL

EARTH MATERIAL: BEDROCK

SAMPLE B1-25'

			B1-25'
Phi Angle =	30 degrees	Moisture Content	63.1%
Cohesion =	590 psf	Dry Density (pcf)	60.0
		Percent Saturation	95.2%



BYER
GEOTECHNICAL
INC.

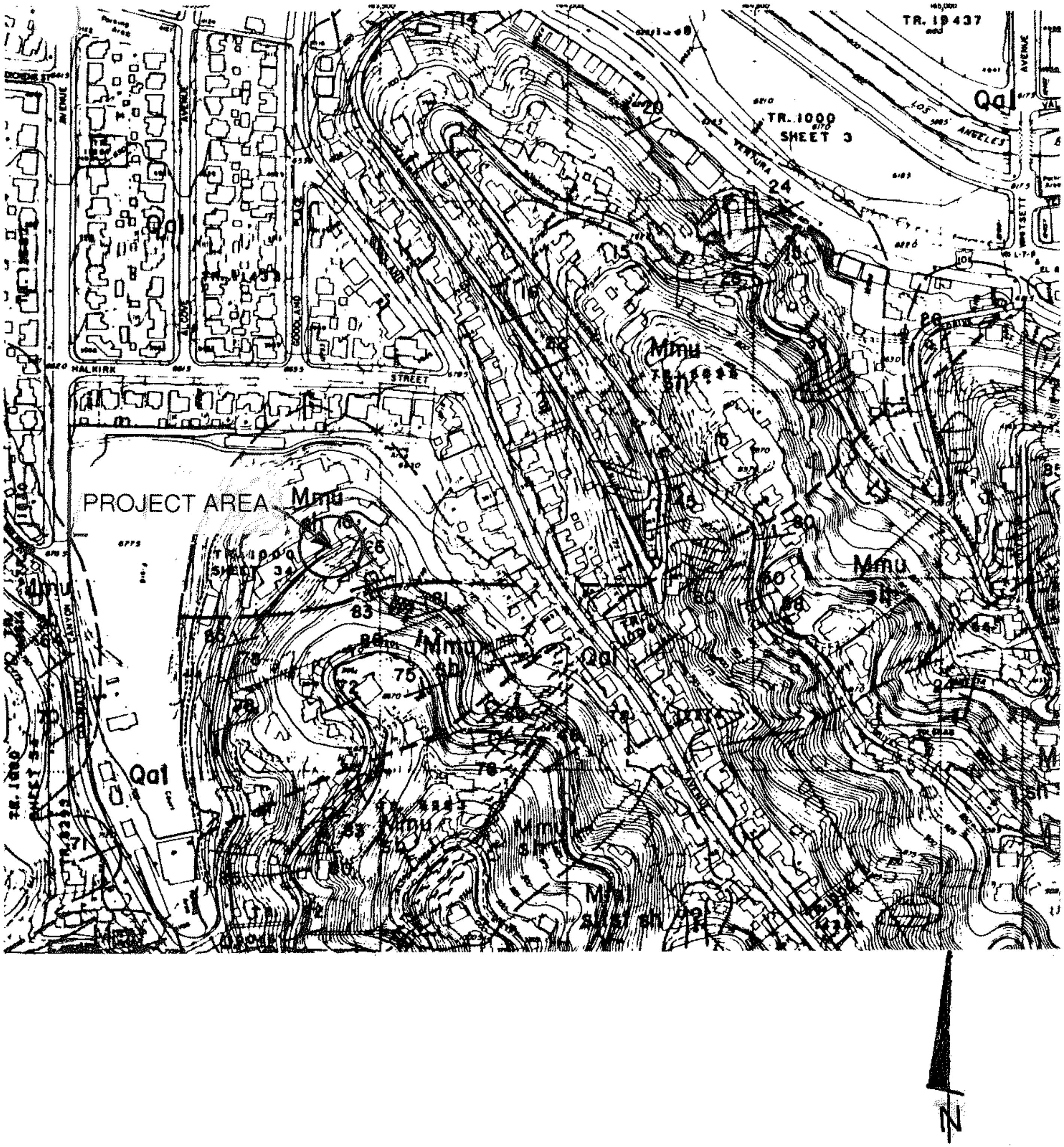
1461 E. CHEVY CHASE DRIVE, # 200, GLENDALE, CA 91206
tel 818.549.9959 fax 818.543.3747

REGIONAL GEOLOGIC MAP

CLIENT: HARVARD WESTLAKE SCHOOL

GEOLOGIST: JET BG: 21256 SCALE: 1" = 400'

Reference: *City of Los Angeles Preliminary Geologic Maps,* Sheet 70.



BYER GEOTECHNICAL INC.

1461 E. CHEVY CHASE DRIVE, SUITE 200, GLENDALE, CA 91206
tel 818.549.9959 fax 818.543.3747

LOG OF TEST PITS

CLIENT: HARVARD-WESTLAKE SCHOOL

GEOLOGIST: JET BG: 21256

REPORT DATE: */*/* DATE LOGGED: 11/4/10

SAMPLE DEPTH (feet)	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	DEPTH INTERVAL (feet)	EARTH MATERIAL	LITHOLOGIC DESCRIPTION
TEST PIT #1					Location and Surface Conditions: Planter
2	21.9	97.6	0 - 4	<u>FILL</u>:	Gravelly Silt, black, dark gray-brown, moist, firm, with some clay at 1 foot, 1-inch diameter PVC irrigation line at 3 feet, 4-inch diameter cast iron pipe
6	43.2	72.4	4 - 6	**ANCIENT LANDSLIDE DEBRIS:**	Siltstone, Shale, tan, moderately hard, weathered
End at 6 Feet; No Water; No Caving; Fill to 4 Feet. Bottom of footing at 5 feet.					
TEST PIT #2					Location and Surface Conditions: Planter
2	28.8	78.1	0 - 4	<u>FILL</u>:	Gravelly Silt, black, dark gray-brown, moist, firm, with some clay at 1 foot, 1-inch diameter PVC irrigation line at 3 feet, 4-inch diameter cast iron pipe
5 7	29.7 59.3	74.4 55.7	4 - 7	**ANCIENT LANDSLIDE DEBRIS:**	Siltstone, tan, light brown, moderately hard, weathered at 6 feet, buff, Diatomaceous
End at 7 Feet; No Water; No Caving; Fill to 4 Feet. Bottom of footing at 5 feet.					
TEST PIT #3					Location and Surface Conditions: Planter
2	42.0	73.9	0 - 4	<u>FILL</u>:	Gravelly Silt, dark gray, brown, black, moist, firm with some clay at 6 inches, 1-inch diameter PVC irrigation line
6	53.9	68.3	4 - 6½	**ANCIENT LANDSLIDE DEBRIS:**	Siltstone, light brown, medium hard, weathered at 5½ feet, buff, Diatomaceous
End at 6½ Feet; No Water; No Caving; Fill to 4 Feet. Bottom of footing at 5 feet.					

NOTE: The stratification depths shown on the Log of Test Pits are approximate and are based upon visual classification of samples and cuttings. The actual depths may vary. Variations between test pits may also occur.

BYER GEOTECHNICAL, INC.

1461 E. CHEVY CHASE DR., SUITE 200
GLENDALE, CA 91206
818.549.9959 TEL
818.543.3747 FAX

LOG OF BORING B1

BG No. 21256

CLIENT Harvard-Westlake School **REPORT DATE** **DRILL DATE** 11/24/10

PROJECT LOCATION 3700 Coldwater Canyon Avenue **LOGGED BY** JET

CONTRACTOR Al Roy **DRILLING METHOD** Bucket-Auger **HOLE SIZE** 24-inch

DRIVE WEIGHT 2400#23',1550#24-43',850#44'+ **HAMMER DROP** 18 Inches **ELEV. TOP OF HOLE** 738 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 12 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	0	Surface: Asphalt Parking		ML						
		(ML) **FILL:** 0-3': Gravelly Silt, brown, dark gray-brown, moist, firm, rock, concrete, and brick fragments up to 10-inch diameter, with some clay								
735		**ANCIENT SLIDE DEBRIS:** 3.0': Siltstone, gray-brown to brown, fractured to very fractured, medium hard, Diatomaceous								
	5	Bedding: N70E, 34N								
		7.0': Siltstone and Shale, gray to light brown, slightly fractured, tight, moderately hard, light brown to brown								
730		8.0': Bedding: N70E, 33N								
		9.0': Siltstone and fine Sandstone, tan to light brown								
	10	9.5': very fractured								
		10.5': light gray-brown and light gray, fractured with dark brown staining along fractures			R	4	63.6	58.7	92.7	
		12.0': light gray, less fractured								
725		13.0': Bedding: N45E, 26N								
		13.5': Siltstone and Shale, brown, gray, very fractured, soil-filled fractures up to 1-inch wide, with roots up to 0.12 inch wide								
	15	15.0': Siltstone and fine Sandstone, light gray, gray, gray-brown, slightly fractured, tight			R	2	69.6	55.8	93.8	
		16.0': 3-inch-thick Sandstone layer, reddish-brown Bedding: N30E, 64N								
720		16.5': Siltstone, gray-brown, gray, contorted, soil-filled fractures to 0.25 inch								
	20	19.5': very fractured								
		20.0': very contorted, fractured			R	3	40.1	76.4	91.4	
		22.0': Siltstone and Shale, gray-brown, very fractured								
715		**BEDROCK:** 23.0': BEDROCK: Siltstone and fine Sandstone, gray, tan, moderately hard, tight, well bedded								
	25	24.0': Bedding: N65E, 20N								

■ Ring Sample

BYER GEOTECHNICAL, INC.

1461 E. CHEVY CHASE DR., SUITE 200
GLENDALE, CA 91206
818.549.9959 TEL
818.543.3747 FAX

LOG OF BORING B1

BG No. 21256

CLIENT Harvard-Westlake School **REPORT DATE** ______ **DRILL DATE** 11/24/10

PROJECT LOCATION 3700 Coldwater Canyon Avenue **LOGGED BY** JET

CONTRACTOR Al Roy **DRILLING METHOD** Bucket-Auger **HOLE SIZE** 24-inch

DRIVE WEIGHT 2400#23',1550#24-43',850#44'+ **HAMMER DROP** 18 Inches **ELEV. TOP OF HOLE** 738 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 12 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	25	24.0': Bedding: N65E, 20N *(continued)*			R	6	72	60	94.3	
		26.0': Siltstone and Shale, gray, brown, very hard Bedding: N70E, 18N								
710		28.0': Siltstone, gray, gray-brown, moderately hard to hard								
	30				R	8	39.2	78.9	94.8	
		31.0': Bedding: N72, 16N								
705										
		34.0': Shale, light gray, hard to very hard								
	35	35.0': very hard; Bedding: N70E, 17N			R	30	66.7	59.5	94.3	
		36.0': Siltstone and Shale, light brown, tan, light gray, hard								
700		37.5': Siltstone, light gray to gray, moderately hard to hard								
	40	39.5': Bedding: N68E, 16N 40.0': Siltstone and fine Sandstone, light gray, light brown, hard			R	12	49.6	69.1	94.3	
695										
		44.0': fine Sandstone, gray, light gray, moderately hard to hard								
	45	45.0': Siltstone and fine Sandstone, light gray, tan, light brown, moderately hard to hard 46.0': Bedding: N64E, 17N			R	30	42.3	70.3	94.5	
690										
	50									

Ring Sample

BYER GEOTECHNICAL, INC.
1461 E. CHEVY CHASE DR., SUITE 200
GLENDALE, CA 91206
818.549.9959 TEL
818.543.3747 FAX

LOG OF BORING B1

BG No. 21256

CLIENT Harvard-Westlake School REPORT DATE DRILL DATE 11/24/10

PROJECT LOCATION 3700 Coldwater Canyon Avenue LOGGED BY JET

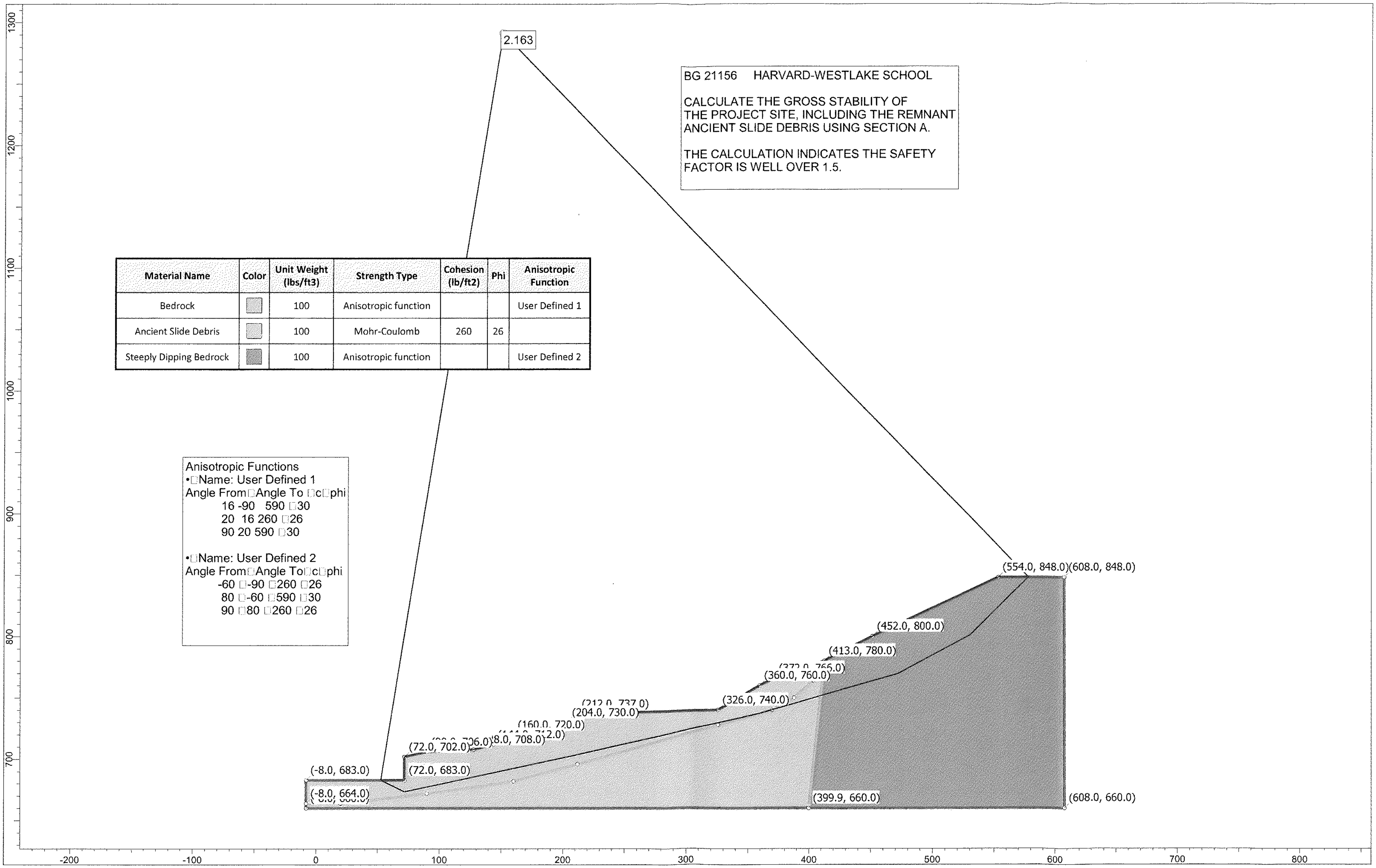
CONTRACTOR Al Roy DRILLING METHOD Bucket-Auger HOLE SIZE 24-inch

DRIVE WEIGHT 2400#23',1550#24-43',850#44'+ HAMMER DROP 18 Inches ELEV. TOP OF HOLE 738 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 12 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	50									
					R	40	32	85.8	94.4	

End at 51 Feet; No Groundwater; Fill to 3.0 Feet.

Ring Sample



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Anisotropic Function
Bedrock		100	Anisotropic function			User Defined 1
Ancient Slide Debris		100	Mohr-Coulomb	260	26	
Steeply Dipping Bedrock		100	Anisotropic function			User Defined 2

Slide Analysis Information

SLIDE - An Interactive Slope Stability Program

Project Summary

- File Name: 21256 Section A ver2.slim
- Slide Modeler Version: 6.006
- Project Title: SLIDE - An Interactive Slope Stability Program
- Date Created: 12/10/2010, 11:08:30 AM

General Settings

- Units of Measurement: Imperial Units
- Time Units: days
- Permeability Units: feet/second
- Failure Direction: Right to Left
- Data Output: Standard
- Maximum Material Properties: 20
- Maximum Support Properties: 20

Analysis Options

Analysis Methods Used

  - Spencer

- Number of slices: 50
- Tolerance: 0.005
- Maximum number of iterations: 50
- Check malpha < 0.2: Yes
- Initial trial value of FS: 1
- Steffensen Iteration: Yes

Groundwater Analysis

- Groundwater Method: Water Surfaces
- Pore Fluid Unit Weight: 62.4 lbs/ft3
- Advanced Groundwater Method: None

Random Numbers

- Pseudo-random Seed: 10116
- Random Number Generation Method: Park and Miller v.3

Surface Options

Material Properties

Property	Bedrock	Ancient Slide Debris	Steeply Dipping Bedrock
Color			
Strength Type	Anisotropic function	Mohr-Coulomb	Anisotropic function
Unit Weight [lbs/ft3]	100	100	100
Cohesion [psf]		260	
Friction Angle [deg]		26	
Water Surface	None	None	None
Ru Value	0	0	0

Anisotropic Functions

- Name: User Defined 1

Angle From	Angle To	c	phi
16	-90	590	30
20	16	260	26
90	20	590	30

- Name: User Defined 2

Angle From	Angle To	c	phi
-60	-90	260	26
80	-60	590	30
90	80	260	26

Global Minimums

Method: spencer

- FS: 2.163100
- Axis Location: 150.656, 1291.168
- Left Slip Surface Endpoint: 52.822, 683.000
- Right Slip Surface Endpoint: 578.490, 848.000
- Resisting Moment=5.29528e+008 lb-ft
- Driving Moment=2.44801e+008 lb-ft
- Resisting Horizontal Force=825711 lb
- Driving Horizontal Force=381726 lb

Global Minimum Coordinates

Method: spencer

X	Y
52.8218	683
72	673.417
210.398	703.376
247.183	711.72
360.847	737.504
472.492	769.517
530.713	800.946
578.49	848

Valid / Invalid Surfaces

Method: spencer

- Number of Valid Surfaces: 1
- Number of Invalid Surfaces: 0

Slice Data

- **Global Minimum Query (spencer) - Safety Factor: 2.1631**

Slice Number	Width [ft]	Weight [lbs]	Base Material	Base Cohesion [psf]	Base Friction Angle [degrees]	Shear Stress [psf]	Shear Strength [psf]	Base Normal Stress [psf]	Pore Pressure [psf]	Effective Normal Stress [psf]
1	9.5891	2297.4	Ancient Slide Debris	260	26	233.385	504.836	501.989	0	501.989
2	9.5891	6892.2	Ancient Slide Debris	260	26	394.882	854.169	1218.23	0	1218.23
3	10.646	30462.5	Ancient Slide Debris	260	26	739.143	1598.84	2745.03	0	2745.03
4	10.646	30435.8	Ancient Slide Debris	260	26	738.602	1597.67	2742.63	0	2742.63
5	10.646	29037.5	Ancient Slide Debris	260	26	710.291	1536.43	2617.07	0	2617.07
6	10.646	27180.6	Ancient Slide Debris	260	26	672.697	1455.11	2450.33	0	2450.33
7	10.646	25323.6	Ancient Slide Debris	260	26	635.098	1373.78	2283.6	0	2283.6
8	10.646	24078.9	Ancient Slide Debris	260	26	609.898	1319.27	2171.82	0	2171.82
9	10.646	24462.7	Ancient Slide Debris	260	26	617.669	1336.08	2206.29	0	2206.29
10	10.646	26851.2	Ancient Slide Debris	260	26	666.026	1440.68	2420.76	0	2420.76
11	10.646	29232	Ancient Slide Debris	260	26	714.23	1544.95	2634.54	0	2634.54
12	10.646	29463.8	Ancient Slide Debris	260	26	718.922	1555.1	2655.35	0	2655.35
13	10.646	29586.2	Ancient Slide Debris	260	26	721.4	1560.46	2666.34	0	2666.34
14	10.646	29708.7	Ancient Slide Debris	260	26	723.882	1565.83	2677.34	0	2677.34
15	10.646	31156.9	Ancient Slide Debris	260	26	753.201	1629.25	2807.38	0	2807.38
16	12.2616	39560.7	Ancient Slide Debris	260	26	813.924	1760.6	3076.69	0	3076.69
17	12.2616	36654.8	Ancient Slide Debris	260	26	763.109	1650.68	2851.32	0	2851.32
18	12.2616	33640	Ancient Slide Debris	260	26	710.388	1536.64	2617.49	0	2617.49

19	10.3331	26008.3	Ancient Slide Debris	260	26	661.809	1431.56	2402.07	0	2402.07
20	10.3331	23867.2	Ancient Slide Debris	260	26	617.382	1335.46	2205.01	0	2205.01
21	10.3331	21726.2	Ancient Slide Debris	260	26	572.953	1239.35	2007.97	0	2007.97
22	10.3331	19585.1	Ancient Slide Debris	260	26	528.523	1143.25	1810.93	0	1810.93
23	10.3331	17444.1	Ancient Slide Debris	260	26	484.094	1047.14	1613.88	0	1613.88
24	10.3331	15303	Ancient Slide Debris	260	26	439.665	951.039	1416.84	0	1416.84
25	10.3331	13161.9	Ancient Slide Debris	260	26	395.236	854.934	1219.8	0	1219.8
26	10.3331	11436.9	Ancient Slide Debris	260	26	359.438	777.501	1061.03	0	1061.03
27	10.3331	14113.9	Ancient Slide Debris	260	26	414.991	897.666	1307.41	0	1307.41
28	10.3331	17972.7	Ancient Slide Debris	260	26	495.064	1070.87	1662.53	0	1662.53
29	10.3331	21828.2	Ancient Slide Debris	260	26	575.071	1243.94	2017.37	0	2017.37
30	9.62926	23059	Ancient Slide Debris	260	26	619.079	1339.13	2212.54	0	2212.54
31	9.85392	25094	Bedrock	260	26	650.705	1407.54	2352.8	0	2352.8
32	9.85392	25643.7	Bedrock	260	26	662.318	1432.66	2404.31	0	2404.31
33	9.85392	26175	Bedrock	260	26	673.543	1456.94	2454.09	0	2454.09
34	9.85392	26706.3	Bedrock	260	26	684.767	1481.22	2503.87	0	2503.87
35	10.4332	29315.2	Steeply Dipping Bedrock	590	30	966.516	2090.67	2599.24	0	2599.24
36	10.4332	31693.3	Steeply Dipping Bedrock	590	30	1022.72	2212.24	2809.8	0	2809.8
37	10.4332	34154.2	Steeply Dipping Bedrock	590	30	1080.87	2338.04	3027.7	0	3027.7
38	10.4332	36615.1	Steeply Dipping Bedrock	590	30	1139.03	2463.84	3245.59	0	3245.59
39	10.4332	38862.3	Steeply Dipping Bedrock	590	30	1192.14	2578.72	3444.56	0	3444.56

40	10.4332	40863.8	Steeply Dipping Bedrock	590	30	1239.44	2681.04	3621.78	0	3621.78
41	11.6443	46254.6	Steeply Dipping Bedrock	590	30	1123.21	2429.61	3186.31	0	3186.31
42	11.6443	45316	Steeply Dipping Bedrock	590	30	1105.66	2391.66	3120.57	0	3120.57
43	11.6443	44377.4	Steeply Dipping Bedrock	590	30	1088.12	2353.71	3054.84	0	3054.84
44	11.6443	43438.8	Steeply Dipping Bedrock	590	30	1070.57	2315.76	2989.1	0	2989.1
45	11.6443	42500.2	Steeply Dipping Bedrock	590	30	1053.03	2277.81	2923.36	0	2923.36
46	9.55543	32143.1	Steeply Dipping Bedrock	590	30	846.151	1830.31	2148.28	0	2148.28
47	9.55543	27447.4	Steeply Dipping Bedrock	590	30	757.468	1638.48	1816.02	0	1816.02
48	9.55543	22070.8	Steeply Dipping Bedrock	590	30	655.927	1418.84	1435.59	0	1435.59
49	9.55543	13488.7	Steeply Dipping Bedrock	590	30	493.845	1068.24	828.329	0	828.329
50	9.55543	4496.24	Steeply Dipping Bedrock	590	30	324.013	700.873	192.038	0	192.038

Interslice Data

- **Global Minimum Query (spencer) - Safety Factor: 2.1631**

Slice Number	X coordinate [ft]	Y coordinate - Bottom [ft]	Interslice Normal Force [lbs]	Interslice Shear Force [lbs]	Interslice Force Angle [degrees]
1	52.8218	683	0	0	0
2	62.4109	678.208	4643.33	1397.91	16.7548
3	72	673.417	14267.3	4295.27	16.7548
4	82.646	675.721	15810.2	4759.76	16.7548
5	93.292	678.026	17352.8	5224.19	16.7548
6	103.938	680.33	18883.4	5685	16.7548
7	114.584	682.635	20398.1	6140.99	16.7548
8	125.23	684.939	21896.7	6592.16	16.7548
9	135.876	687.244	23384.6	7040.11	16.7548
10	146.522	689.548	24875.8	7489.05	16.7548
11	157.168	691.853	26387.6	7944.19	16.7548
12	167.814	694.158	27919.9	8405.5	16.7548
13	178.46	696.462	29454.2	8867.41	16.7548
14	189.106	698.767	30989.6	9329.63	16.7548
15	199.752	701.071	32526	9792.18	16.7548
16	210.398	703.376	34074.8	10258.5	16.7549
17	222.66	706.157	35497.3	10686.7	16.7548
18	234.921	708.939	36923.5	11116.1	16.7548
19	247.183	711.72	38353.7	11546.7	16.7549
20	257.516	714.064	39561.9	11910.4	16.7548
21	267.849	716.408	40772.9	12275	16.7548
22	278.182	718.752	41986.7	12640.4	16.7548
23	288.515	721.096	43203.3	13006.7	16.7548
24	298.849	723.44	44422.6	13373.7	16.7547
25	309.182	725.784	45644.7	13741.7	16.7548
26	319.515	728.128	46869.6	14110.4	16.7548
27	329.848	730.472	48096.7	14479.8	16.7547
28	340.181	732.816	49320.3	14848.2	16.7548
29	350.514	735.16	50538.9	15215.1	16.7548
30	360.847	737.504	51752.6	15580.5	16.7548
31	370.476	740.265	51604.7	15536	16.7548
32	380.33	743.09	51368.7	15464.9	16.7548

33	390.184	745.916	51101.6	15384.5	16.7548
34	400.038	748.741	50804.5	15295.1	16.7548
35	409.892	751.567	50477.2	15196.5	16.7548
36	420.325	754.559	52785.1	15891.3	16.7548
37	430.759	757.55	55049.3	16573	16.7548
38	441.192	760.542	57268.4	17241.1	16.7548
39	451.625	763.534	59442.5	17895.6	16.7548
40	462.058	766.525	61575.3	18537.7	16.7548
41	472.492	769.517	63671.5	19168.8	16.7548
42	484.136	775.803	56722.3	17076.6	16.7547
43	495.78	782.089	49981.9	15047.4	16.7548
44	507.424	788.374	43450.4	13081.1	16.7549
45	519.069	794.66	37127.8	11177.6	16.7548
46	530.713	800.946	31014.1	9337.02	16.7548
47	540.268	810.357	18882.3	5684.66	16.7548
48	549.824	819.767	9029.94	2718.53	16.7548
49	559.379	829.178	1787.53	538.15	16.7548
50	568.935	838.589	-1288.85	-388.017	16.7548
51	578.49	848	0	0	0

List Of Coordinates

External Boundary

X	**Y**
-8	660
399.884	660
608	660
608	848
554	848
452	800
413	780
372	766
360	760
326	740
212	737

204	730
160	720
144	712
128	708
90	706
72	702
72	683
-8	683
-8	664

Material Boundary

X	Y
-8	664
20	664
90	672
160	682
212	696
326	728
370	740
388	750
404	764
413	780

Material Boundary

X	Y
399.884	660
413	780

BYER GEOTECHNICAL INC.

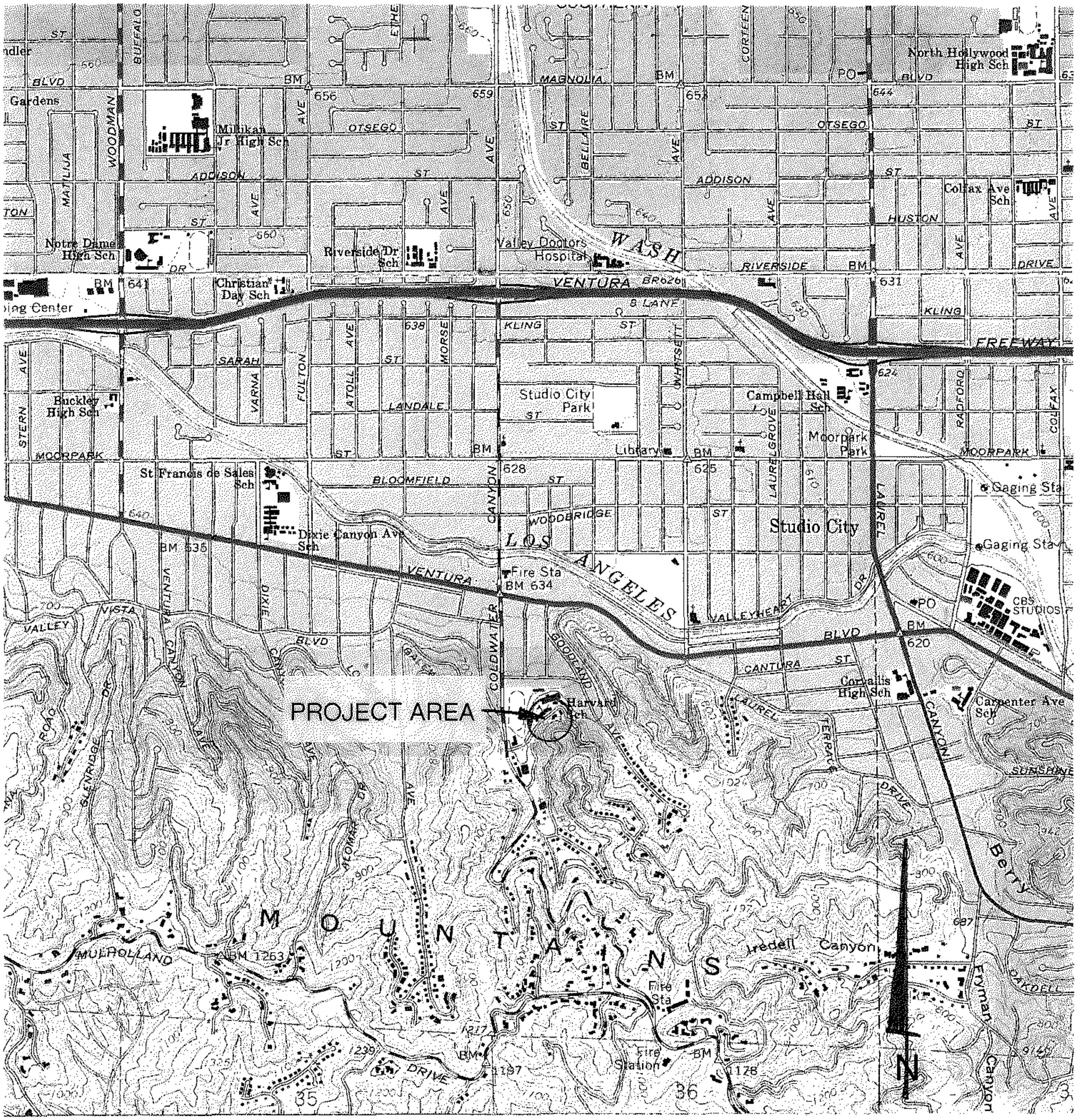
1461 E. CHEVY CHASE DRIVE, # 200, GLENDALE, CA 91206
tel 818.549.9959 fax 818.543.3747

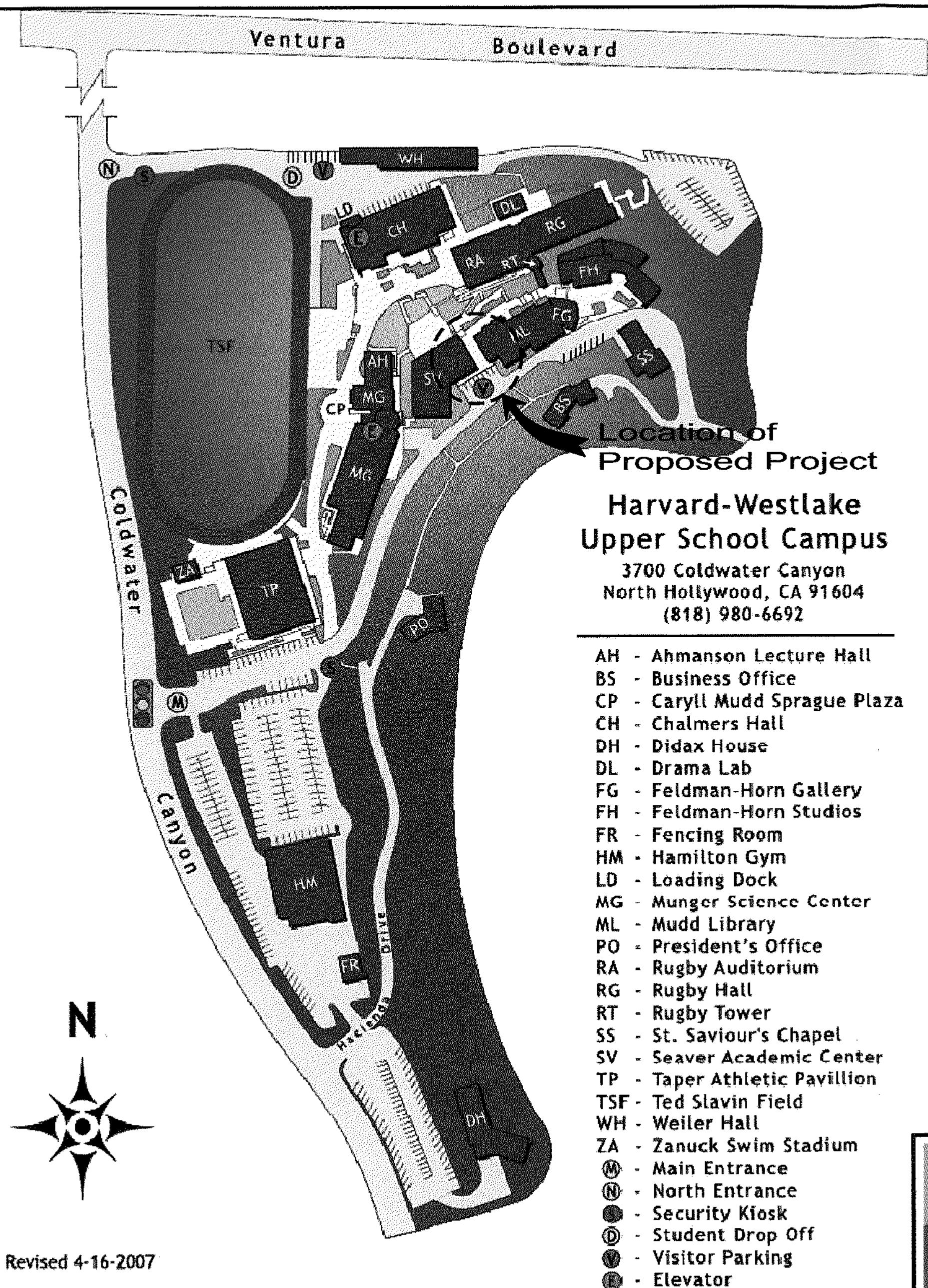
VICINITY MAP

CLIENT: HARVARD WESTLAKE SCHOOL

GEOLOGIST: JET BG: 21256 SCALE: 1" = 2,000'

Reference: U.S.G.S. Topographic Map of the Van Nuys 7½ Minute Quadrangle.





DECEMBER 30, 2010

BYER GEOTECHNICAL INC.
1461 E. CHEVY CHASE DR., SUITE 200
GLENDALE, CA 91206
818.549.9959 TEL
818.543.3747 FAX

LOCATION MAP

BG: 21256 HARVARD-WESTLAKE SCHOOL

CONSULTANT: JET

SCALE: Not to Scale

BYER
GEOTECHNICAL
INC.
1461 E. CHEVY CHASE DR., SUITE 200
GLENDALE, CA 91206
818.549.9959 TEL
818.543.3747 FAX

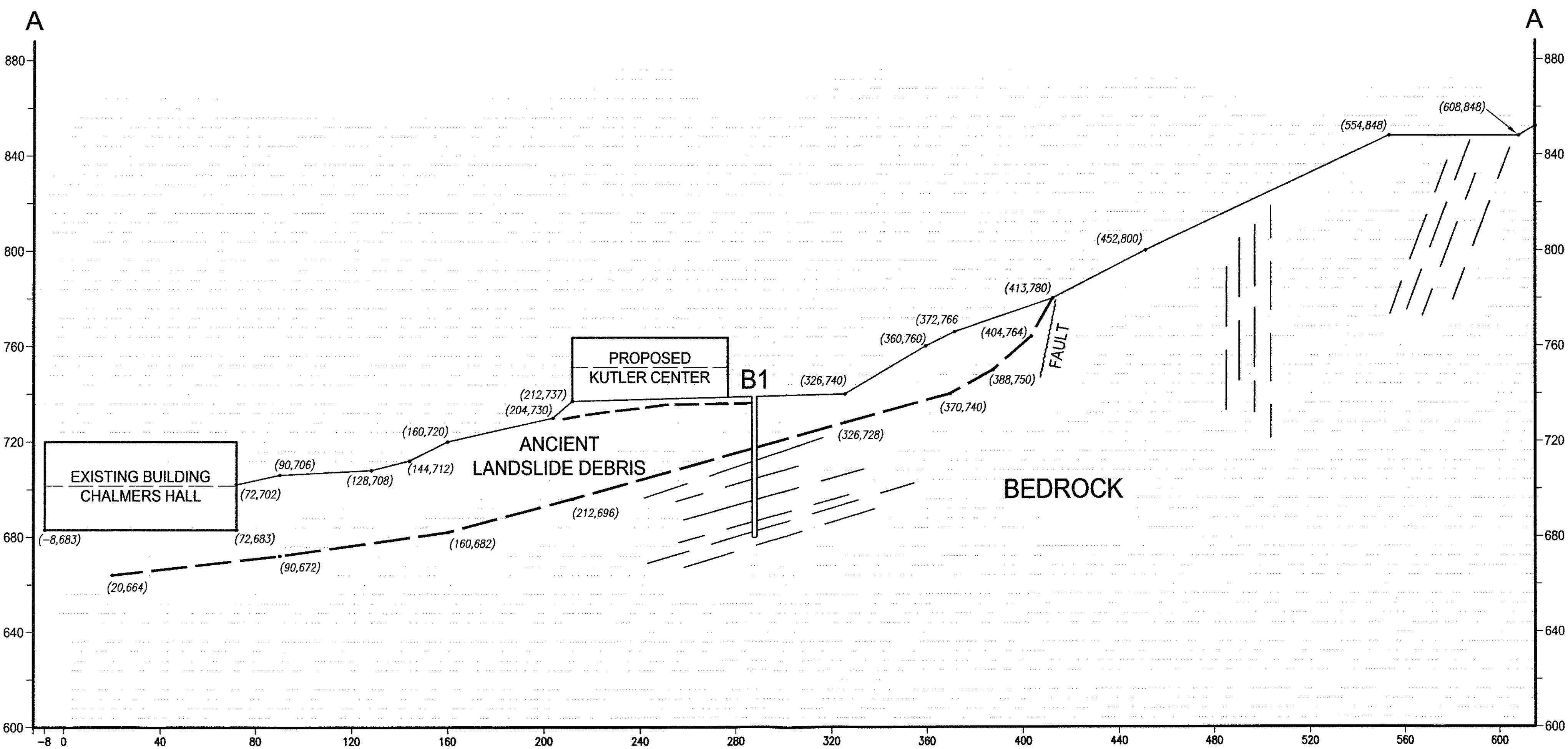
SECTION A

BG: 21256 HARVARD-WESTLAKE SCHOOL

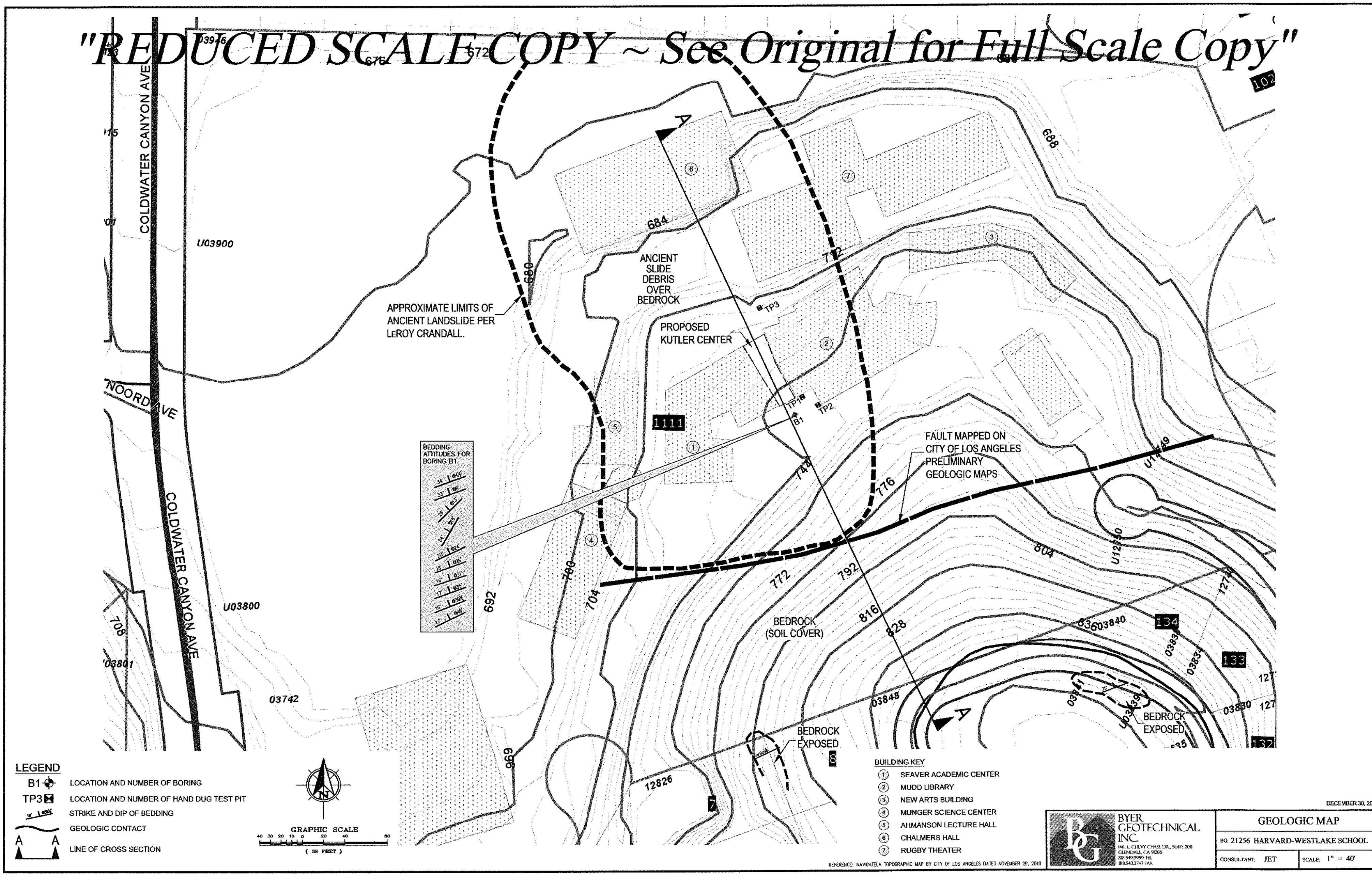
CONSULTANT: JET

SCALE: 1" = 40'

DECEMBER 30, 2010



S:\LandProjects\21256 HARVARD-WESTLAKE SCHOOL\dwg\GEOMAP_12022010.dwg 12/10/2010 11:15:20 AM PST


"REDUCED SCALE COPY ~ See Original for Full Scale Copy"
COLDWATER CANYON AVE
NOORD AVE
U03900
U03800
03742
APPROXIMATE LIMITS OF ANCIENT LANDSLIDE PER LeROY CRANDALL.
ANCIENT SLIDE DEBRIS OVER BEDROCK
PROPOSED KUTLER CENTER
BEDDING ATTITUDES FOR BORING B1
FAULT MAPPED ON CITY OF LOS ANGELES PRELIMINARY GEOLOGIC MAPS
BEDROCK (SOIL COVER)
BEDROCK EXPOSED
BEDROCK EXPOSED
TP1
TP2
TP3
B1
1111
LEGEND
B1 LOCATION AND NUMBER OF BORING
TP3 LOCATION AND NUMBER OF HAND DUG TEST PIT
STRIKE AND DIP OF BEDDING
GEOLOGIC CONTACT
A A LINE OF CROSS SECTION
GRAPHIC SCALE
(IN FEET)
BUILDING KEY
1 SEAVER ACADEMIC CENTER
2 MUDD LIBRARY
3 NEW ARTS BUILDING
4 MUNGER SCIENCE CENTER
5 AHMANSON LECTURE HALL
6 CHALMERS HALL
7 RUGBY THEATER
REFERENCE: NAVIGATELA TOPOGRAPHIC MAP BY CITY OF LOS ANGELES DATED NOVEMBER 29, 2010
BYER GEOTECHNICAL INC.
DECEMBER 30, 2010
GEOLOGIC MAP
BG: 21256 HARVARD-WESTLAKE SCHOOL
CONSULTANT: JET
SCALE: 1" = 40'

BYER GEOTECHNICAL, INC.

September 20, 2011
BG 21401

DWR Construction, Inc.
3051 Bostonian Drive
Los Alamitos, California 90720

Attention: Douglas W. Roberts, President

Subject

Transmittal of Geologic and Soils Engineering Exploration Update
Proposed Pool, Pool House, and Retaining Wall
Harvard-Westlake School
Lot 1111, Tract 1000
3700 Coldwater Canyon Avenue
Studio City, California

Gentlepersons:

Byer Geotechnical has completed our update report dated September 20, 2011, which describes the geologic and soils engineering conditions with respect to construction of the proposed pool, pool house, and retaining wall. The reviewing agency for this document is City of Los Angeles, Department of Building and Safety (LADBS). The reviewing agency requires three unbound copies, one with a wet signature, an application form, and a filing fee. Five copies of the report are enclosed.

It is our understanding that your representative will file the report with the LADBS. Please review the report carefully prior to submittal to the governmental agency. Questions concerning the report should be directed to the project consultant. Byer Geotechnical appreciates the opportunity to offer our consultation and advice on this project.

Very truly yours,
BYER GEOTECHNICAL, INC.

James Tucker

James E. Tucker
Project Consultant

BYER GEOTECHNICAL, INC.

GEOLOGIC AND SOILS ENGINEERING EXPLORATION UPDATE
PROPOSED POOL, POOL HOUSE, AND RETAINING WALL
HARVARD-WESTLAKE SCHOOL
LOT 1111, TRACT 1000
3700 COLDWATER CANYON AVENUE
STUDIO CITY, CALIFORNIA
FOR DWR CONSTRUCTION, INC.
BYER GEOTECHNICAL, INC., PROJECT NUMBER BG 21401
SEPTEMBER 20, 2011

GEOLOGIC AND SOILS ENGINEERING EXPLORATION UPDATE
PROPOSED POOL, POOL HOUSE, AND RETAINING WALL
HARVARD-WESTLAKE SCHOOL
LOT 1111, TRACT 1000
3700 COLDWATER CANYON AVENUE
STUDIO CITY, CALIFORNIA
FOR DWR CONSTRUCTION, INC.
BYER GEOTECHNICAL, INC., PROJECT NUMBER BG 21401
SEPTEMBER 20, 2011

INTRODUCTION

This report has been prepared per our signed Agreement and summarizes findings of Byer Geotechnical, Inc., geologic and soils engineering exploration update performed on a portion of the Harvard-Westlake School campus. The purpose of this study is to evaluate the nature, distribution, engineering properties, relative stability, and geologic structure of the earth materials underlying the site with respect to construction of the proposed pool, pool house, and retaining wall. This report is intended to assist in the design and completion of the proposed project and to reduce geotechnical risks that may affect the project. The professional opinions and advice presented in this report are based upon commonly accepted exploration standards and are subject to the AGREEMENT with TERMS AND CONDITIONS, and the GENERAL CONDITIONS AND NOTICE section of this report. No warranty is expressed or implied by the issuing of this report.

PROPOSED PROJECT

The scope of the proposed project was determined from the preliminary plans prepared by Arch Pac Incorporated. The project consists of the proposed pool, pool house, and retaining wall. A retaining wall up to 12 feet high is planned to support an excavation below the main entrance driveway.

EXPLORATION

The scope of the field exploration was determined from our initial site visit and consultation with DWR Construction, Inc. The preliminary plans prepared by Arch Pac Incorporated, dated June 22, 2011, were a guide to our work on this project. Exploration was conducted using techniques normally applied to this type of project in this setting. This report is limited to the area of the exploration and the proposed project as shown on the Geologic Map and cross sections. The scope of this exploration did not include an assessment of general site environmental conditions for the presence of contaminants in the earth materials and groundwater. Conditions affecting portions of the property outside the area explored are beyond the scope of this report.

Exploration was conducted on August 22, 2011, with the aid of a hollow-stem auger drill rig. It included drilling two borings to a depth of 35½ feet. Samples of the earth materials were obtained and delivered to our soils engineering laboratory for testing and analysis. The cuttings from the borings were visually logged by the project consultant. The borings were backfilled and tamped, but should not be considered compacted.

Office tasks included laboratory testing of selected soil samples, review of published maps and photos for the area, review of our files, review of agency files, preparation of cross sections, preparation of the Geologic Map, engineering analysis, and preparation of this report. Earth materials exposed in the borings are described on the enclosed Log of Borings. Appendix I contains a discussion of the laboratory testing procedures and results.

The proposed project, surface geologic conditions, and the locations of the borings are shown on the Geologic Map. Subsurface distribution of the earth materials, projected geologic structure, and the proposed project are shown on Sections A, B, and C.

RESEARCH - PRIOR WORK

Agency records contain the following geotechnical reports, which were prepared for the property:

> *Geologic Engineering Investigation, Proposed Library and Field House,* **Geologic and Soils Consultants, Inc.**, dated January 29, 1973,;
>
> *Pile Inspection Report, Proposed Field and House and Gymnasium,* **Kovacs-Byer and Associates, Inc.**, dated July 16, 1979, ;
>
> *Addendum Geologic and Soil Engineering Exploration, Proposed Swimming Pool and Pool House,* **Kovacs-Byer and Associates, Inc.**, dated October 29, 1984,; and
>
> *Geologic and Soils Engineering Update, Proposed Field House Addition, Harvard-Westlake School, Portion of Lot 1111, Tract 1000, 3700 Coldwater Canyon Avenue, North Hollywood, California,* **The J. Byer Group, Inc.**, dated October 16, 1998.

The City of Los Angeles, Department of Building and Safety, reviewed the reports and issued the conditional approval letters dated March 9, 1973, December 14, 1984, and January 19, 1999.

The data contained in these reports was reviewed and considered as part of our work on this project.

SITE DESCRIPTION

The subject property consists of a school campus on a graded hillside parcel on the north flank of the Santa Monica Mountains, in the Studio City section of the city of Los Angeles, California (34.1396° N Latitude, 118.4132° W Longitude). It is located on the east side of Coldwater Canyon Avenue, approximately one-half of a mile south of Ventura Boulevard. The area of the proposed development is located in the east-central portion of the school campus, adjacent to Coldwater Canyon Avenue and immediately north of the main entrance driveway. A swimming pool and pool house previously occupied the site. Both have been demolished and the debris removed. Currently, the site consists of a level dirt area. In the area previously occupied by the pool and pool house, cut slopes associated with the demolition ascend to the south and west to the driveway and Coldwater

Canyon Avenue, respectively. The slopes are up to 15 feet high and range in gradient from 2:1 to 3:1. A dirt access ramp descends from a level area, north of the driveway, along the west portion of the site to the lower level area. A two-story gym-locker building is located to the east and the athletic field to the north of the site.

The site is mostly barren, with scattered trees and shrubs along the west side adjacent to Coldwater Canyon Avenue. Surface drainage is by sheetflow runoff down the contours of the land to the north, where it is collected in drains that discharge to the south of the athletic field.

GROUNDWATER

In *Seismic Hazard Zone Report 08, Seismic Hazard Zone Report for the Van Nuys 7.5-Minute Quadrangle, Los Angeles County, California,* 1997, the California Geological Survey (CGS) has estimated the historically-highest groundwater level at the site was between 20 and 40 feet below ground surface. Water was found in Boring 1 at 29 feet below grade. Water was not encountered in Boring 2 to a depth of 35 feet.

Seasonal fluctuations in groundwater levels occur due to variations in climate, irrigation, development, and other factors not evident at the time of the exploration. Groundwater levels may also differ across the site. Groundwater can saturate earth materials causing subsidence or instability of slopes.

EARTH MATERIALS

Fill

Fill, associated with previous site grading and demolition of the old pool and pool house, underlies the site to a maximum observed depth of three feet in Boring 1. In addition, fill associated with construction and installation of the storm drain underlies the west portion of the site. This fill is anticipated to be on the order of 15 feet deep (see Section C). Greater depths of fill may occur. The fill consists of clayey silt, which is black, dark gray-brown, moist, firm with some rock and concrete fragments up to four inches.

Alluvium

Natural alluvium underlies the site. The alluvium observed in the borings is 23 and 24 feet thick, and is anticipated to thicken toward the northeast. The alluvium consists of clayey to sandy silt and gravelly clay, which is dark brown, brown, tan, dark gray, moist to very moist, and firm to very firm.

Bedrock

Bedrock underlying the site and encountered in the borings consists of diatomaceous siltstone and shale mapped as part of the Modelo Formation (Hoots, 1931). The bedrock is gray, gray-brown, and soft to moderately hard.

GEOLOGIC STRUCTURE

The bedrock described above is common to this area of the eastern Santa Monica Mountains and the geologic structure is consistent with regional trends. The regional structure consists of beds, which generally strike east-west and dip shallowly to moderately to the north.

The geologic structure of the bedrock is favorably oriented for stability of the site and proposed project.

GENERAL SEISMIC CONSIDERATIONS

The subject property is located in an active seismic region. Moderate to strong earthquakes can occur on numerous local faults. The United States Geological Survey, California Geological Survey

(CGS), private consultants, and universities have been studying earthquakes in southern California for several decades. Early studies were directed toward earthquake prediction and estimation of the effects of strong ground shaking. Studies indicate that earthquake prediction is not practical and not sufficiently accurate to benefit the general public. Governmental agencies now require earthquake-resistant structures. The purpose of the code seismic-design parameters is to prevent collapse during strong ground shaking. Cosmetic damage should be expected.

Southern California faults are classified as "active" or "potentially active." Faults from past geologic periods of mountain building that do not display evidence of recent offset are considered "potentially active." Faults that have historically produced earthquakes or show evidence of movement within the past 11,000 years are known as "active faults." No known active faults cross the subject property.

The following table lists the applicable 2011 City of Los Angeles Building Code seismic coefficients for the project:

SEISMIC COEFFICIENTS (2011 City of Los Angeles Building Code)		
Latitude = 34.1396° N Longitude = 118.4132° W	Short Period (0.2s)	One-Second Period
Earth Materials and Site Class from Table 1613.5.2 and Section 1613.5.2	Alluvium - D	
Mapped Spectral Accelerations from Figures 1613.5(3) and 1613.5(4) and USGS	S_s = 1.500 (g)	S_1 = 0.600 (g)
Site Coefficients from Tables 1613.5.3 (1) and 1613.5.3 (2) and USGS	F_A = 1.0	F_V = 1.5
Maximum Considered Spectral Response Accelerations from Equations 16-36 and 16-37	S_{MS} = 1.500 (g)	S_{M1} = 0.900 (g)
Design Spectral Response Accelerations from Equations 16-38 and 16-39	S_{DS} = 1.000 (g)	S_{D1} = 0.600 (g)

Reference: U.S. Geological Survey, **Earthquake Hazards Program, Seismic Design Values for Buildings,** http://earthquake.usgs.gov/hazards/design/buildings.php.

The mapped spectral response acceleration parameter for the site for a 1-second period (S_1) is less than 0.75g. The design spectral response acceleration parameters for the site for a 1-second period (S_{D1}) is greater than or equal to 0.20g, and the short period (S_{DS}) is greater than or equal to 0.50g. Therefore, the project is considered to be in Seismic Design Category D.

The principal seismic hazard to the proposed project is strong ground shaking from earthquakes produced by local faults. Modern, well-constructed buildings are designed to resist ground shaking through the use of shear panels, moment frames, and reinforcement. Additional precautions may be taken, including strapping water heaters and securing furniture to walls and floors. It is likely that the subject property will be shaken by future earthquakes produced in southern California.

Ground Motion

In *Seismic Hazard Zone Report 08,* the California Geological Survey (CGS) has assigned ground motion values for this area of Los Angeles. The Design Basis Ground Motion (10 percent exceedance in 50 years) is a peak ground acceleration (PGA) of 0.52 and an earthquake with moment magnitude (M_w) of 6.4. These ground motions could occur at the site during the life of the project.

Liquefaction

The CGS has mapped the subject site within an area where historic occurrence of liquefaction or geological, geotechnical, and groundwater conditions indicate a potential for permanent ground displacement such that mitigation as defined in Public Resources Code Section 2693 (c) would be required.

Liquefaction is a process that occurs when saturated sediments are subjected to repeated strain reversals during an earthquake. The strain reversals cause increased pore water pressure such that the internal pore pressure approaches the overburden stress and the shear strength approaches zero. Liquefied soils may be subject to flow or excessive strain, which may induce settlement.

Liquefaction occurs in soils below the groundwater table. Soils commonly subject to liquefaction include loose to medium-dense sand and silty sand. Predominantly fine-grained soils, such as silts and clay, are less susceptible to liquefaction. Generally, soils with fines content (percent passing the 200 sieve) greater than 35 percent are not considered susceptible to liquefaction. Soils data collected in the borings were utilized to quantify the liquefaction potential of the site. A ground acceleration of 0.52g and a design magnitude earthquake of 6.4 (M_w) were used for the analyses. For a conservative analysis, it was assumed that groundwater rose to the historic-high groundwater level, approximately 20 feet below the ground surface.

A liquefaction potential analysis based upon SPT data from Borings 1 and 2 is presented in Appendix II on the enclosed plate entitled "Liquefaction Susceptibility Analysis: SPT Method." The column labeled "Factor of Safety" lists the calculated safety factor of each 2½-foot-thick layer of soil encountered in the borings and below the historic-high groundwater level. The stresses and safety factor for liquefaction were calculated using the methodology of Idriss and Boulanger (2008) and the Southern California Earthquake Center (1999). A factor of safety of 1.2 against liquefaction was used to differentiate between potentially-liquefiable and non-liquefiable soil layers.

The liquefaction analysis based upon SPT data from Borings 1 and 2 indicates that the alluvial soils underlying the subject site are not considered susceptible to liquefaction.

CONCLUSIONS AND RECOMMENDATIONS

General Findings

The conclusions and recommendations of this exploration are based upon review of the preliminary plans, review of published maps, two borings, field geologic mapping, research of available records, laboratory testing, engineering analysis, and years of experience performing similar studies on similar sites. It is the finding of Byer Geotechnical, Inc., that development of the proposed project is feasible from a geologic and soils engineering standpoint provided the advice and recommendations contained in this report are included in the plans and are implemented during construction.

The recommended bearing material is the future compacted fill and alluvium. Soils to be exposed at finished grade will be in the medium expansion range. Geotechnical issues affecting the project include the existing storm-drain pipe and associated backfill located in the southwest portion of the site.

It is recommended that the soils in the area of the proposed pool and pool house be removed to a depth of three feet below the future pool bottom and pool-house footings, and replaced with a certified compacted fill to provide a uniform bearing material. The proposed surge tank and retaining wall may be supported by the alluvium. The proposed retaining wall adjacent to the driveway will require excavations up to 12 feet high, which will remove support from the driveway. It is recommended that these excavations be shored utilizing soldier piles. The piles may be utilized in the permanent design of the retaining wall.

SITE PREPARATION - REMOVALS

Surficial materials consisting of uncertified fill and disturbed alluvium are present on the site. Remedial grading is recommended to improve site conditions. The fill and disturbed alluvium should be removed to a depth of three feet below the future pool bottom and pool-house footings, and replaced as certified compacted fill. The following general grading specifications may be used in preparation of the grading plan and job specifications. Byer Geotechnical would appreciate the opportunity of reviewing the plans to ensure that these recommendations are included. The grading contractor should be provided with a copy of this report.

A. The area to receive compacted fill should be prepared by removing all vegetation, debris, existing fill, and disturbed alluvium. The exposed excavated area should be observed by the geologist prior to placing compacted fill. The exposed grade should be scarified to a depth of six inches, moistened to optimum moisture content, and recompacted to 90 percent of the maximum density.

B. The proposed pool and pool house shall be excavated to a minimum depth of three feet below the bottom of all footings. The excavation shall extend beyond the edge of the exterior footing a minimum of three feet or to the depth of fill below the footing. The excavated areas shall be observed by the soils engineer/geologist prior to placing compacted fill.

C. Fill, consisting of soil approved by the soils engineer, shall be placed in horizontal lifts, moistened as required, and compacted in six-inch layers with suitable compaction equipment. The excavated onsite materials are considered satisfactory for reuse in the controlled fills. Any imported fill shall be observed by the soils engineer prior to use in fill areas. Rocks larger than six inches in diameter shall not be used in the fill.

D. The moisture content of the fill should be near optimum moisture content. When the moisture content of the fill is too wet or dry, the fill shall be moisture conditioned and mixed until the proper moisture is attained.

E. The fill shall be compacted to at least 90 percent of the maximum laboratory density for the material used. The maximum density shall be determined by ASTM D 1557-09 or equivalent.

F. Field observation and testing shall be performed by the soils engineer during grading to assist the contractor in obtaining the required degree of compaction and the proper moisture content. Where compaction is less than required, additional compactive effort shall be made with adjustment of the moisture content, as necessary, until 90 percent compaction is obtained. A minimum of one compaction test is required for each 500 cubic yards or two vertical feet of fill placed.

FOUNDATION DESIGN

Spread Footings

Continuous and/or pad footings may be used to support the proposed pool house, provided they are founded in future compacted fill. Continuous footings should be a minimum of 12 inches in width. Pad footings should be a minimum of 24-inches square. The following chart contains the recommended design parameters.

Bearing Material	Minimum Embedment Depth of Footing (Inches)	Vertical Bearing (psf)	Coefficient of Friction	Passive Earth Pressure (pcf)	Maximum Earth Pressure (psf)
Future Compacted Fill	12	2,000	0.30	300	4,000

Increases in the bearing value are allowable at a rate of 400 pounds-per-square-foot for each additional foot of footing width or depth to a maximum of 4,000 pounds-per-square-foot. For bearing calculations, the weight of the concrete in the footing may be neglected.

The bearing value shown above is for the total of dead and frequently applied live loads and may be increased by one-third for short duration loading, which includes the effects of wind or seismic forces. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

Footings adjacent to retaining walls or pool walls should be deepened below a 1:1 plane from the bottom of the lower retaining wall or pool. All continuous footings should be reinforced with a minimum of four #4 steel bars: two placed near the top, and two near the bottom of the footings. Footings should be cleaned of all loose soil, moistened, free of shrinkage cracks, and approved by the geologist prior to placing forms, steel, or concrete.

Shoring Piles

Drilled, cast-in-place concrete shoring piles may be utilized to support excavations for the proposed retaining wall adjacent to the driveway. The piles should be a minimum of 24 inches in diameter and a minimum of eight feet into alluvium below the grade of future excavations. Piles may be assumed fixed at three feet into alluvium below the grade of future excavations. The piles may be designed for a skin friction of 600 pounds-per-square-foot for that portion of pile in contact with the alluvium below the grade of future excavations. Piles should be spaced a maximum of eight feet on center. Based upon the enclosed calculations, the piles may be designed for 30 pounds-per-cubic-foot. The equivalent fluid pressure should be multiplied by the pile spacing. The piles may be included in the permanent retaining wall.

Tieback Anchors

Tieback anchors may be used to resist lateral loads. Conventional, drilled friction anchors or pressure-grouted anchors may be used. The active wedge adjacent to the shoring is defined by a plane drawn at 35 degrees with the vertical through the bottom of the excavation. The friction anchors should extend at least 15 feet beyond the active wedge or to a greater length if necessary to develop the desired resistance. For design purposes, it is estimated that drilled friction anchors a minimum of 10 feet beyond the active wedge will develop an average friction value of 600 pounds-per-square-foot. Only the frictional resistance developed beyond the active wedge will be effective in resisting lateral loads. If anchors are spaced no closer than six feet, on center, no reduction in the capacity of the anchors is necessary. The anchors may be installed at angles of 20 to 40 degrees below the horizontal. Tieback anchors should be tested during installation in accordance with the specifications of the shoring engineer.

Lagging

Continuous lagging is anticipated between the soldier piles. The soldier piles and anchors should be designed for the full anticipated lateral pressure. However, the pressure on the lagging will be less due to arching in the soils. Lagging should be designed for the recommended earth pressure, but may be limited to a maximum value of 400 pounds-per-square-foot.

Deflection

Some deflection of the shored embankment should be anticipated. Where shoring is planned adjacent to existing structures, it is recommended that lateral deflection not exceed one-half of an inch. For shoring not surcharged by a structure, the allowable deflection is deferred to the structural engineer. If greater deflection occurs during construction, additional bracing or anchors may be necessary to minimize deflection. If desired to reduce the deflection of the shoring, a greater active pressure could be used in the shoring design.

Lateral Design

The friction value is for the total of dead and frequently applied live loads and may be increased by one-third for short duration loading, which includes the effects of wind or seismic forces. Resistance to lateral loading may be provided by passive earth pressure within the alluvium below the grade of the future excavation.

Passive earth pressure may be computed as an equivalent fluid having a density of 300 pounds-per-cubic-foot. The maximum allowable earth pressure is 4,000 pounds-per-square-foot. For design of isolated piles, the allowable passive and maximum earth pressures may be increased by 100 percent. Piles spaced more than 2½-pile diameters on center may be considered isolated.

Foundation Settlement

Settlement of the foundation system is expected to occur on initial application of loading. A total settlement of one-fourth to one-half of an inch may be anticipated. Differential settlement should not exceed one-fourth of an inch.

SWIMMING POOL

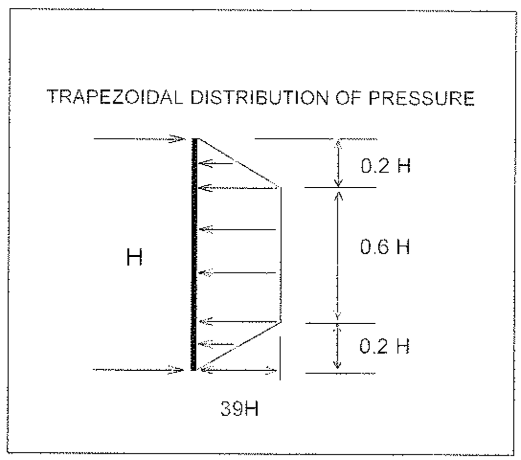
The proposed swimming pool shall be constructed using a freestanding design. Pool walls should be designed for an inward pressure of 43 pounds-per-cubic-foot. The pool should derive support entirely from the future compacted fill. A hydrostatic relief valve is recommended.

RETAINING WALLS

General Design

Retaining walls up to 12 feet high with a level backslope may be designed for an equivalent fluid pressure of 43 pounds-per-cubic-foot. Retaining walls should be provided with a subdrain or weepholes covered with a minimum of 12 inches of ¾-inch crushed gravel.

If the proposed walls for the pool house are to be restrained, they be designed for a lateral earth pressure of 39H, where H is the height of the wall. The diagram illustrates the trapezoidal distribution of earth pressure. The design earth pressures assume that the walls are free draining. Basement walls should be provided with a subdrain or weepholes covered with a minimum of 12 inches of ¾-inch crushed gravel.



Backfill

Retaining wall backfill should be compacted to a minimum of 90 percent of the maximum density as determined by ASTM D 1557-09, or equivalent. Where access between the retaining wall and the temporary excavation prevents the use of compaction equipment, retaining walls should be backfilled with ¾-inch crushed gravel to within two feet of the ground surface. Where the area between the wall and the excavation exceeds 18 inches, the gravel must be vibrated or wheel-rolled, and tested for compaction. The upper two feet of backfill above the gravel should consist of a compacted-fill blanket to the surface. Restrained walls should not be backfilled until the restraining system is in place.

Foundation Design

Retaining wall footings may be sized per the "Spread Footings" section of this report.

Retaining Wall Deflection

It should be noted that non-restrained retaining walls can deflect up to one percent of their height in response to loading. This deflection is normal and results in lateral movement and settlement of the backfill toward the wall. The zone of influence is within a 1½:1 plane from the bottom of the wall. Hard surfaces or footings placed on the retaining wall backfill should be designed to avoid the effects of differential settlement from this movement. Decking that caps a retaining wall should be provided with a flexible joint to allow for the normal deflection of the retaining wall. Decking that does not cap a retaining wall should not be tied to the wall. The space between the wall and the deck will require periodic caulking to prevent moisture intrusion into the retaining wall backfill.

Temporary Excavations

Temporary excavations will be required during grading to construct the proposed retaining wall. The excavations will be up to 12 feet in height and will expose fill over alluvium. The fill should be trimmed to 1:1 for wall excavations. The alluvium is capable of maintaining vertical excavations up to five feet. Where vertical excavations in the alluvium exceed five feet in height, the upper portion should be trimmed to 1:1 (45 degrees).

Vertical excavations removing support from driveway will require the use of shoring consisting of soldier piles. Temporary shoring should be designed for an equivalent fluid pressure of 30 pounds-per-cubic-foot. Values can be found in the "Shoring Piles" design section of this report.

The geologist should be present during grading to see temporary slopes. All excavations should be stabilized within 30 days of initial excavation. Water should not be allowed to pond on top of the excavations nor to flow toward them. No vehicular surcharge should be allowed within three feet of the top of the cut.

FLOOR SLABS

Floor slabs should be cast over approved compacted fill and reinforced with a minimum of #4 bars on 16-inch centers, each way. For deepened foundations, the slabs should be designed to bridge between the piles and grade beams.

Slabs that will be provided with a floor covering should be protected by a polyethylene plastic vapor barrier. The barrier should be sandwiched between the layers of sand, about two inches each, to prevent punctures and aid in the concrete cure. A low-slump concrete may be used to minimize possible curling of the slab. The concrete should be allowed to cure properly before placing vinyl or other moisture sensitive floor covering.

Prior to the placement of concrete slabs on expansive soils, the subgrade shall be pre-moistened until the moisture content reaches at least 120 percent of the optimum moisture content to a depth of twelve inches. The pre-moistened soils should be tested, and verified to be 120 percent of optimum moisture content, prior to pouring.

It should be noted that cracking of concrete slabs is common. The cracking occurs because concrete shrinks as it cures. Control joints, which are commonly used in exterior decking to control such cracking, are normally not used in interior slabs. The reinforcement recommended above is intended to reduce cracking and its proper placement is critical to the performance of the slab. The minor shrinkage cracks, which often form in interior slabs, generally do not present a problem when carpeting, linoleum, or wood floor coverings are used. The slab cracks can, however, lead to surface cracks in brittle floor coverings such as ceramic tile.

EXTERIOR CONCRETE DECKS AND BLEACHERS

Decking and bleachers should be cast over approved compacted fill placed in accordance with the "Site Preparation" section of this report. Decking should be reinforced with a minimum of #3 bars placed 18 inches on center, each way. Decking that caps a retaining wall should be provided with a flexible joint to allow for the normal one to two percent deflection of the retaining wall. Decking that does not cap a retaining wall should not be tied to the wall. The space between the wall and the deck will require periodic caulking to prevent moisture intrusion into the retaining wall backfill. The subgrade should be moistened prior to placing concrete.

DRAINAGE

Control of site drainage is important for the performance of the proposed project. Pad and roof drainage should be collected and transferred to the street or approved location in non-erosive drainage devices. Drainage should not be allowed to pond on the pad or against any foundation or retaining wall. Drainage should not be allowed to flow uncontrolled over any descending slope. Planters located within retaining wall backfill should be sealed to prevent moisture intrusion into the backfill. Planters located next to raised-floor-type construction also should be sealed to the depth of the footings. Drainage control devices require periodic cleaning, testing, and maintenance to remain effective.

Irrigation

Control of irrigation water is a necessary part of site maintenance. Soggy ground and perched water may result if irrigation water is excessively applied. Irrigation systems should be adjusted to provide the minimum water needed. Adjustments should be made for changes in climate and rainfall.

WATERPROOFING

Interior and exterior retaining walls are subject to moisture intrusion, seepage, and leakage, and should be waterproofed. Waterproofing paints, compounds, or sheeting can be effective if properly installed. Equally important is the use of a subdrain that daylights to the atmosphere. The subdrain should be covered with ¾-inch crushed gravel to help the collection of water. Landscape areas above the wall should be sealed or properly drained to prevent moisture contact with the wall or saturation of wall backfill.

PLAN REVIEW

Formal plans ready for submittal to the building department should be reviewed by Byer Geotechnical. Any change in scope of the project may require additional work.

SITE OBSERVATIONS DURING CONSTRUCTION

The building department requires that the geotechnical engineer provide site observations during grading and construction. Foundation excavations should be observed and approved by the geotechnical engineer or geologist prior to placing steel, forms, or concrete. The engineer/geologist should observe bottoms for fill, compaction of fill, pool excavations, temporary slopes, and subdrains. All fill that is placed should be approved by the geotechnical engineer and the building department prior to use for support of structural footings and floor slabs.

Please advise Byer Geotechnical, Inc., at least 24 hours prior to any required site visit. The building department stamped plans, the permits, and the geotechnical reports should be at the job site and available to our representative. The project consultant will perform the observation and post a notice at the job site with the findings. This notice should be given to the agency inspector.

FINAL REPORTS

The geotechnical engineer will prepare interim and final compaction reports upon request. The geologist will prepare reports summarizing pile excavations.

CONSTRUCTION SITE MAINTENANCE

It is the responsibility of the contractor to maintain a safe construction site. The area should be fenced and warning signs posted. All excavations must be covered and secured. Soil generated by foundation excavations should be either removed from the site or placed as compacted fill. Soil should not be spilled over any descending slope. Workers should not be allowed to enter any unshored trench excavations over five feet deep. Water shall not be allowed to saturate open footing trenches.

GENERAL CONDITIONS AND NOTICE

This report and the exploration are subject to the following conditions. Please read this section carefully; it limits our liability.

In the event of any changes in the design or location of any structure, as outlined in this report, the conclusions and recommendations contained herein may not be considered valid unless the changes are reviewed by Byer Geotechnical, Inc., and the conclusions and recommendations are modified or reaffirmed after such review.

The subsurface conditions, excavation characteristics, and geologic structure described herein have been projected from test excavations on the site and may not reflect any variations that occur between these test excavations or that may result from changes in subsurface conditions.

Fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, irrigation, and other factors not evident at the time of the measurements reported herein. Fluctuations also may occur across the site. High groundwater levels can be extremely hazardous. Saturation of earth materials can cause subsidence or slippage of the site.

If conditions encountered during construction appear to differ from those disclosed herein, notify us immediately so we may consider the need for modifications. Compliance with the design concepts, specifications, and recommendations requires the review of the engineering geologist and geotechnical engineer during the course of construction.

THE EXPLORATION WAS PERFORMED ONLY ON A PORTION OF THE SITE, AND CANNOT BE CONSIDERED AS INDICATIVE OF THE PORTIONS OF THE SITE NOT EXPLORED.

This report, issued and made for the sole use and benefit of the client, is not transferable. Any liability in connection herewith shall not exceed the Phase I fee for the exploration and report or a negotiated fee per the Agreement. No warranty is expressed, implied, or intended in connection with the exploration performed or by the furnishing of this report.

THIS REPORT WAS PREPARED ON THE BASIS OF THE PRELIMINARY DEVELOPMENT PLAN FURNISHED. FINAL PLANS SHOULD BE REVIEWED BY THIS OFFICE AS ADDITIONAL GEOTECHNICAL WORK MAY BE REQUIRED.

Byer Geotechnical appreciates the opportunity to provide our service on this project. Any questions concerning the data or interpretation of this report should be directed to the undersigned.

Respectfully submitted,
BYER GEOTECHNICAL, INC.

James E. Tucker
P. G. 6628

Robert I. Zweigler
E. G. 1210/G. E. 2120

REGISTERED PROFESSIONAL ENGINEER
ROBERT I. ZWEIGLER
No. 2120
Exp. 06-30-12
GEOTECHNICAL
STATE OF CALIFORNIA

GC:RIZ:mh
S:\FINAL\BG\21401_DWR_Construction\21401_DWR_Construction_Geo_and_Soils_Update.wpd

Enc: Appendix I - Laboratory Testing (2 Pages)
Shear Test Diagrams (3 Pages)
Consolidation Curves (5 Pages)
Log of Borings 1 and 2 (4 Pages)
Calculation Sheet
Appendix II - Liquefaction Susceptibility Analysis (2 Pages)
Regional Geologic Map
Vicinity Map
Section C
Sections A and B (1 Sheet)
Geologic Map

xc: (5) Addressee (E-mail and Mail)

APPENDIX I

LABORATORY TESTING

Undisturbed and bulk samples of the fill, alluvium, and bedrock were obtained from the borings and transported to the laboratory for testing and analysis. The samples were obtained by driving a ring-lined, barrel sampler conforming to ASTM D 3550-01 with successive drops of the sampler. Experience has shown that sampling causes some disturbance of the sample. However, the test results remain within a reasonable range. The samples were retained in brass rings of 2.50 inches outside diameter and 1.00 inches in height. The samples were stored in close fitting, waterproof containers for transportation to the laboratory.

Moisture-Density

The dry density of the samples was determined using the procedures outlined in ASTM D 2937-10. The moisture content of the samples was determined using the procedures outlined in ASTM D 2216-10. The results are shown on the Log of Borings.

Maximum Density

The maximum dry density and optimum moisture content of the future compacted fill were determined using the procedures outlined in ASTM D 1557-09, a five-layer standard. Remolded samples were prepared at 90 percent of the maximum density. The remolded samples were tested for shear strength.

Boring	Depth (Feet)	Earth Material	Color and Soil Type	Maximum Density (pcf)	Optimum Moisture %	Expansion Index
1	2	Fill	Dark Gray-Brown Clayey Silt	110.0	19.0	70 - Moderate

Expansion Test

To find the expansiveness of the soil, a swell test was performed using the procedures outlined in ASTM D 4829-08A. Based upon the testing, the future fill will be moderately expansive.

APPENDIX I (Continued)

Shear Tests

Shear tests were performed on samples of future compacted fill, alluvium, and bedrock using the procedures outlined in ASTM D 3080-04 and a strain controlled, direct-shear machine manufactured by Soil Test, Inc. The rate of deformation was 0.025 inches per minute. The samples were tested in an artificially saturated condition. Following the shear test, the moisture content of the samples was determined to verify saturation. The results are plotted on the Shear Test Diagrams.

Consolidation

Consolidation tests were performed on *in situ* samples of the alluvium using the procedures outlined in ASTM D 2435-11. Results are graphed on the Consolidation Curves.

BYER
GEOTECHNICAL,
INC.
1461 E. CHEVY CHASE DR., SUITE 200
GLENDALE, CA 91206
818.549.9959 TEL
818.543.3747 FAX

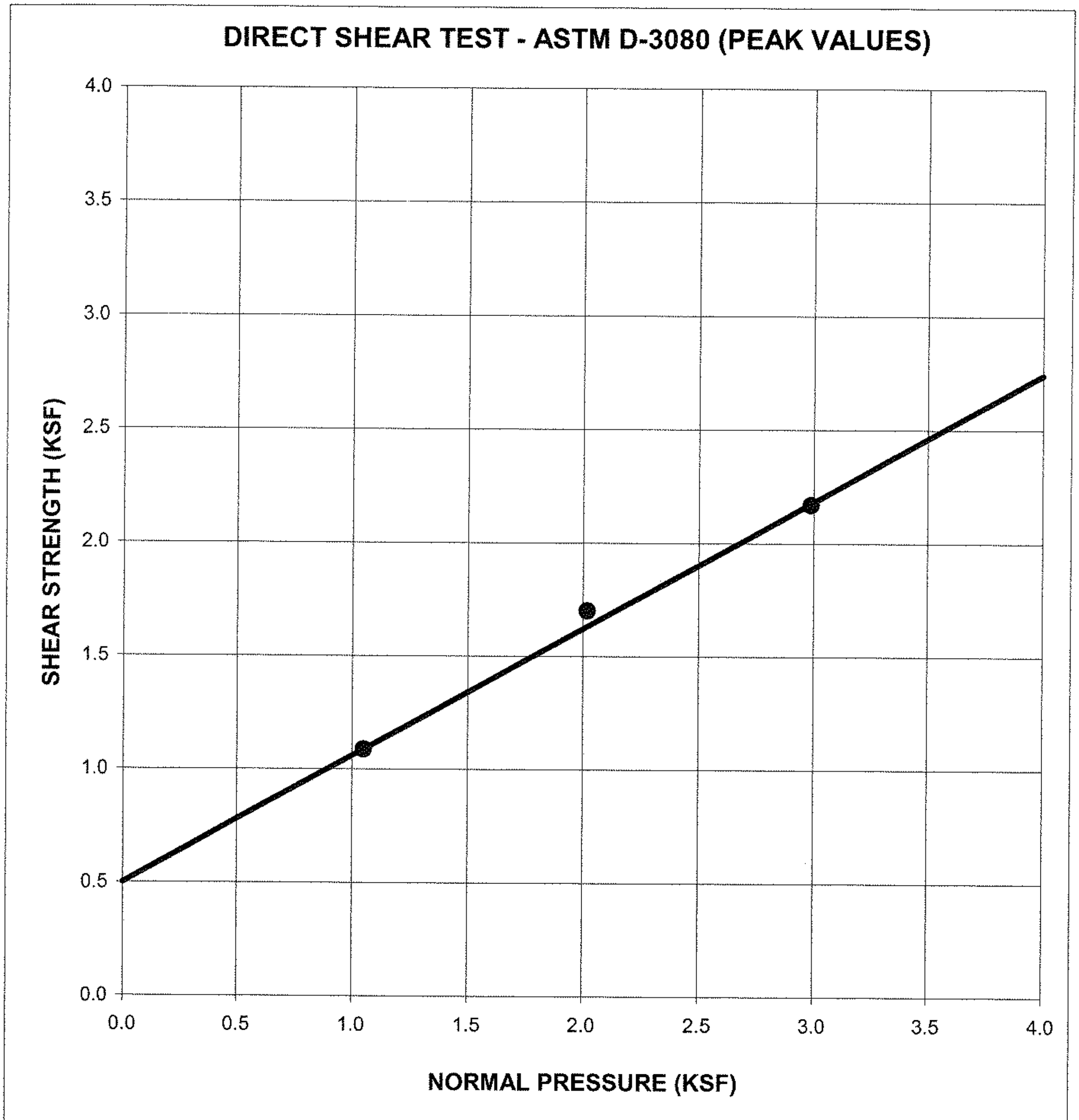
SHEAR DIAGRAM #1

BG: **21401** CONSULTANT: **JET**
CLIENT: **DWR Construction**

EARTH MATERIAL: **FUTURE FILL**

SAMPLE REMOLDED TO 90% OF THE MAXIMUM DENSITY

			B1-2'
Phi Angle =	**29 degrees**	**Moisture Content**	**23.7%**
Cohesion =	**500 psf**	**Dry Density (pcf)**	**99.9**
		Percent Saturation	**95.8%**



BYER GEOTECHNICAL, INC.

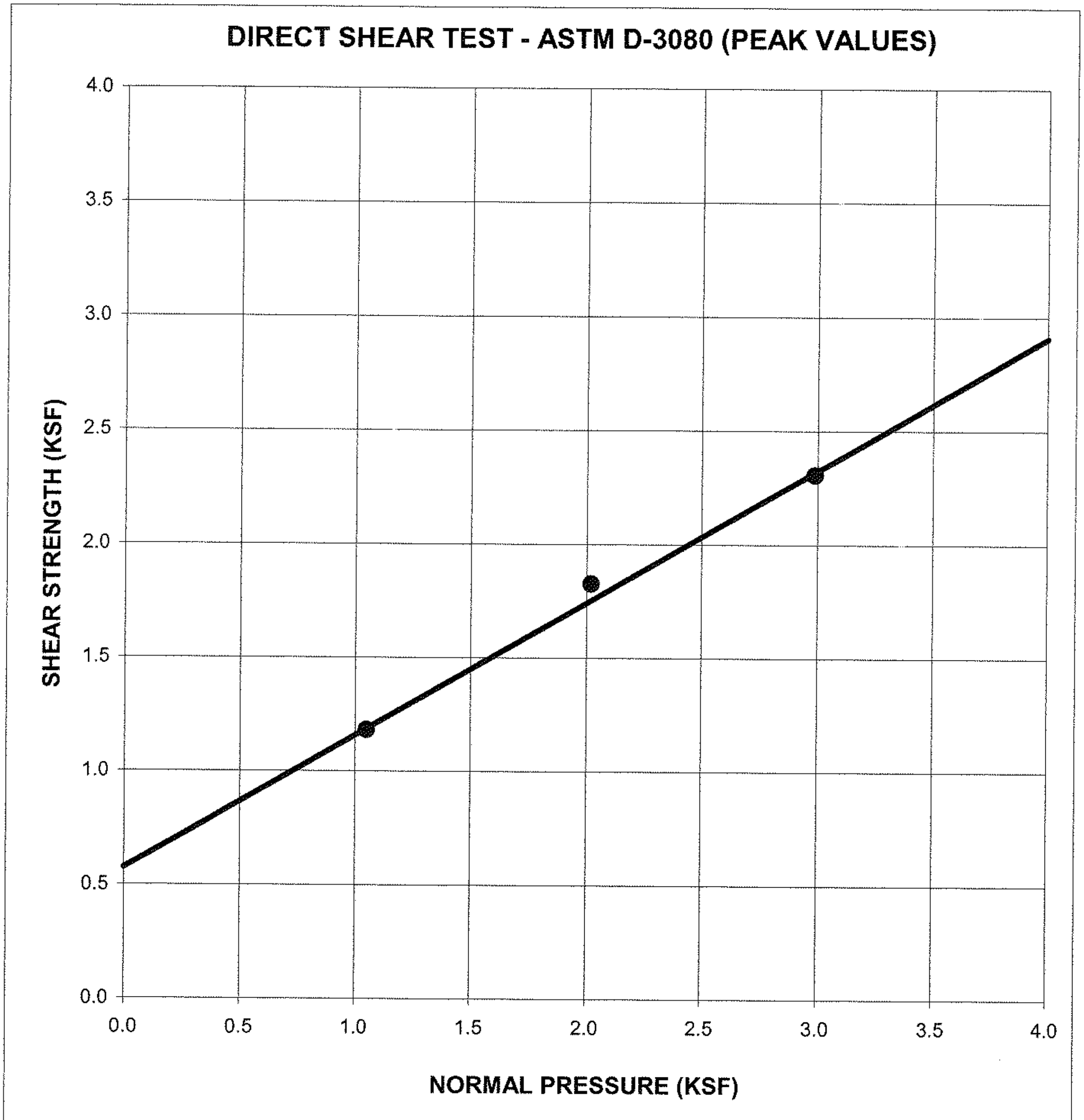
1461 E. CHEVY CHASE DR., SUITE 200
GLENDALE, CA 91206
818.549.9959 TEL
818.543.3747 FAX

SHEAR DIAGRAM #2

BG: 21401 CONSULTANT: JET
CLIENT: DWR Construction

EARTH MATERIAL: ALLUVIUM

			B1-7.5'
Phi Angle =	30 degrees	Moisture Content	20.4%
Cohesion =	574 psf	Dry Density (pcf)	105.6
		Percent Saturation	95.5%



BYER GEOTECHNICAL, INC.

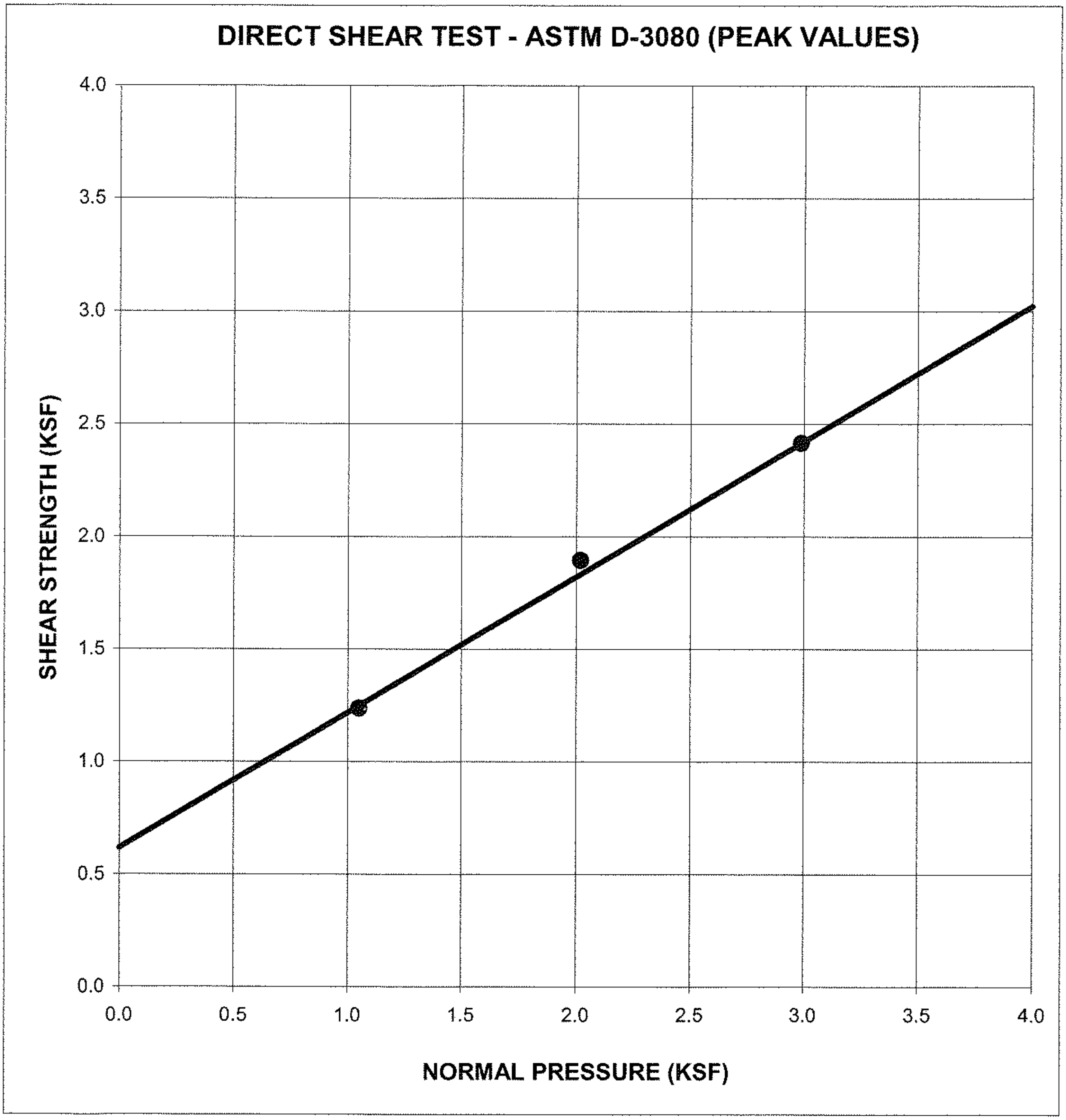
1461 E. CHEVY CHASE DR., SUITE 200
GLENDALE, CA 91206
818.549.9959 TEL
818.543.3747 FAX

SHEAR DIAGRAM #3

BG: 21401 CONSULTANT: JET
CLIENT: DWR Construction

EARTH MATERIAL: BEDROCK

			B1-32.5'
Phi Angle =	31 degrees	Moisture Content	64.5%
Cohesion =	617 psf	Dry Density (pcf)	60.5
		Percent Saturation	98.6%



BYER GEOTECHNICAL, INC.
1461 E. CHEVY CHASE DR., SUITE 200
GLENDALE, CA 91206
818.549.9959 TEL
818.543.3747 FAX

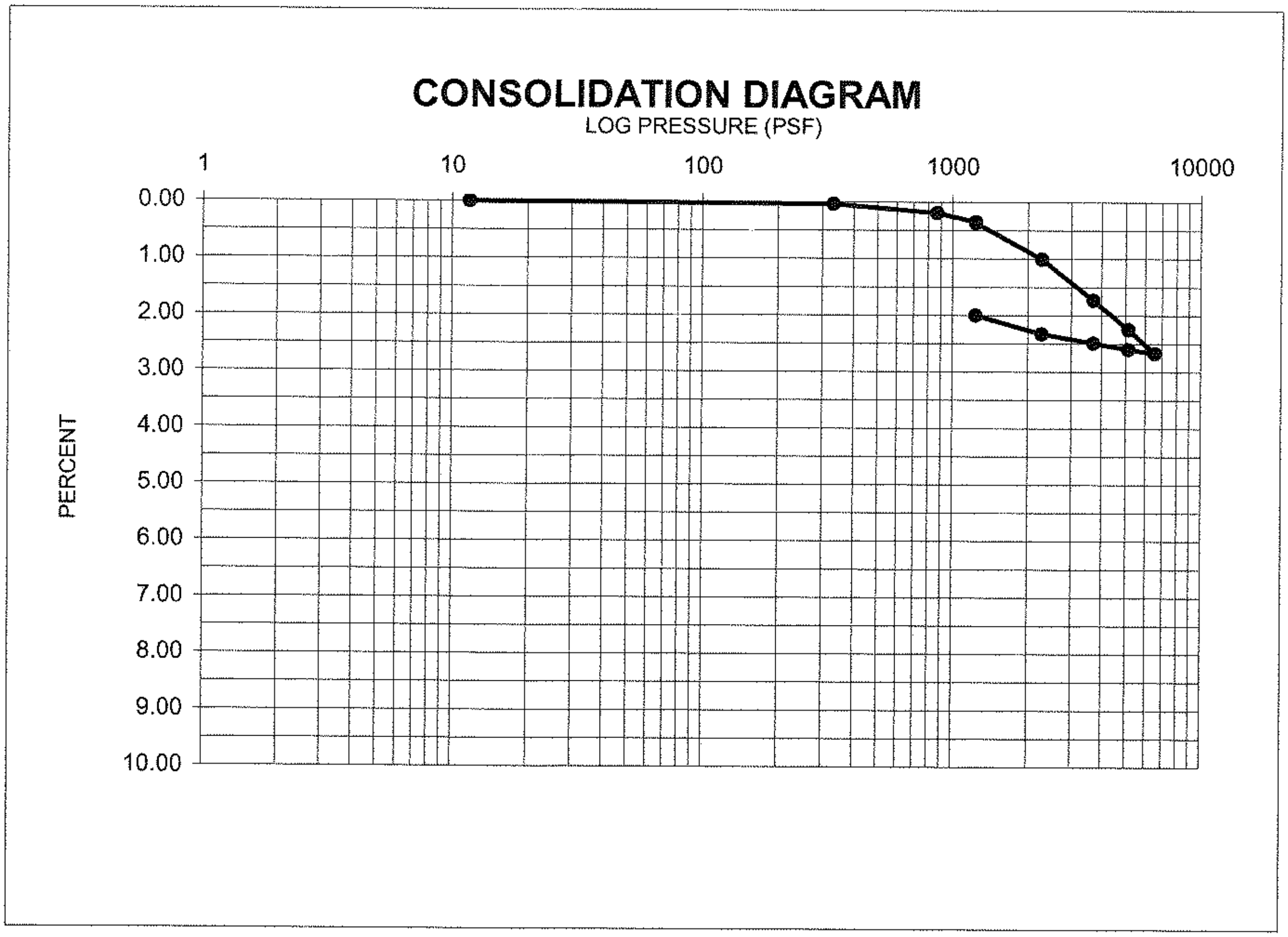
CONSOLIDATION DIAGRAM #1

BG: **21401** CONSULTANT: **JET**

CLIENT: **DWR Construction**

Earth Material:	ALLUVIUM
Sample Location:	B2-5'
Dry Weight (pcf):	100.6
Initial Moisture:	15.6%
Initial Saturation:	64.2%
Water Added at (psf)	1237

Specific Gravity:	2.65
Initial Void Ratio:	0.64
Compression Index (Cc):	0.065
Recompression Index (Cr):	0.020



BYER GEOTECHNICAL, INC.
1461 E. CHEVY CHASE DR., SUITE 200
GLENDALE, CA 91206
818.549.9959 TEL
818.543.3747 FAX

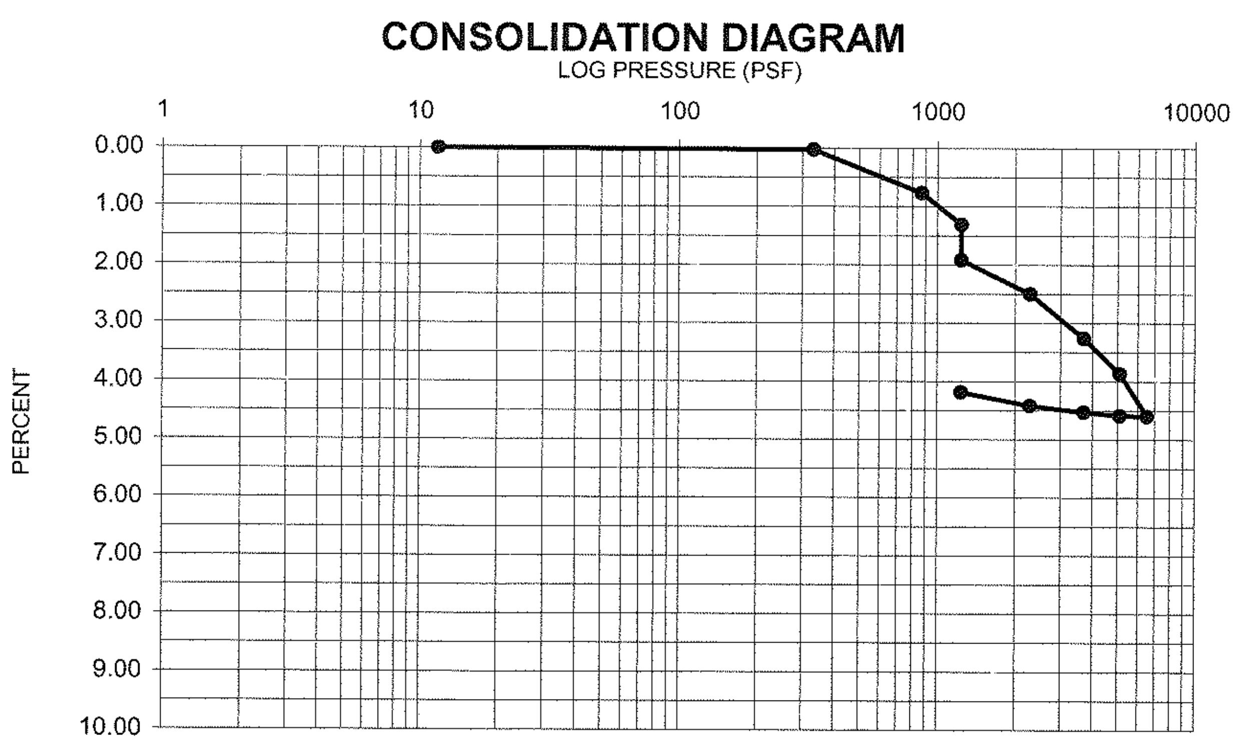
CONSOLIDATION DIAGRAM #2

BG: **21401** CONSULTANT: **JET**

CLIENT: **DWR Construction**

Earth Material:	ALLUVIUM
Sample Location:	B1-7.5'
Dry Weight (pcf):	105.5
Initial Moisture:	19.8%
Initial Saturation:	92.5%
Water Added at (psf)	1237

Specific Gravity:	2.65
Initial Void Ratio:	0.57
Compression Index (Cc):	0.107
Recompression Index (Cr):	0.014



BYER GEOTECHNICAL, INC.
1461 E. CHEVY CHASE DR., SUITE 200
GLENDALE, CA 91206
818.549.9959 TEL
818.543.3747 FAX

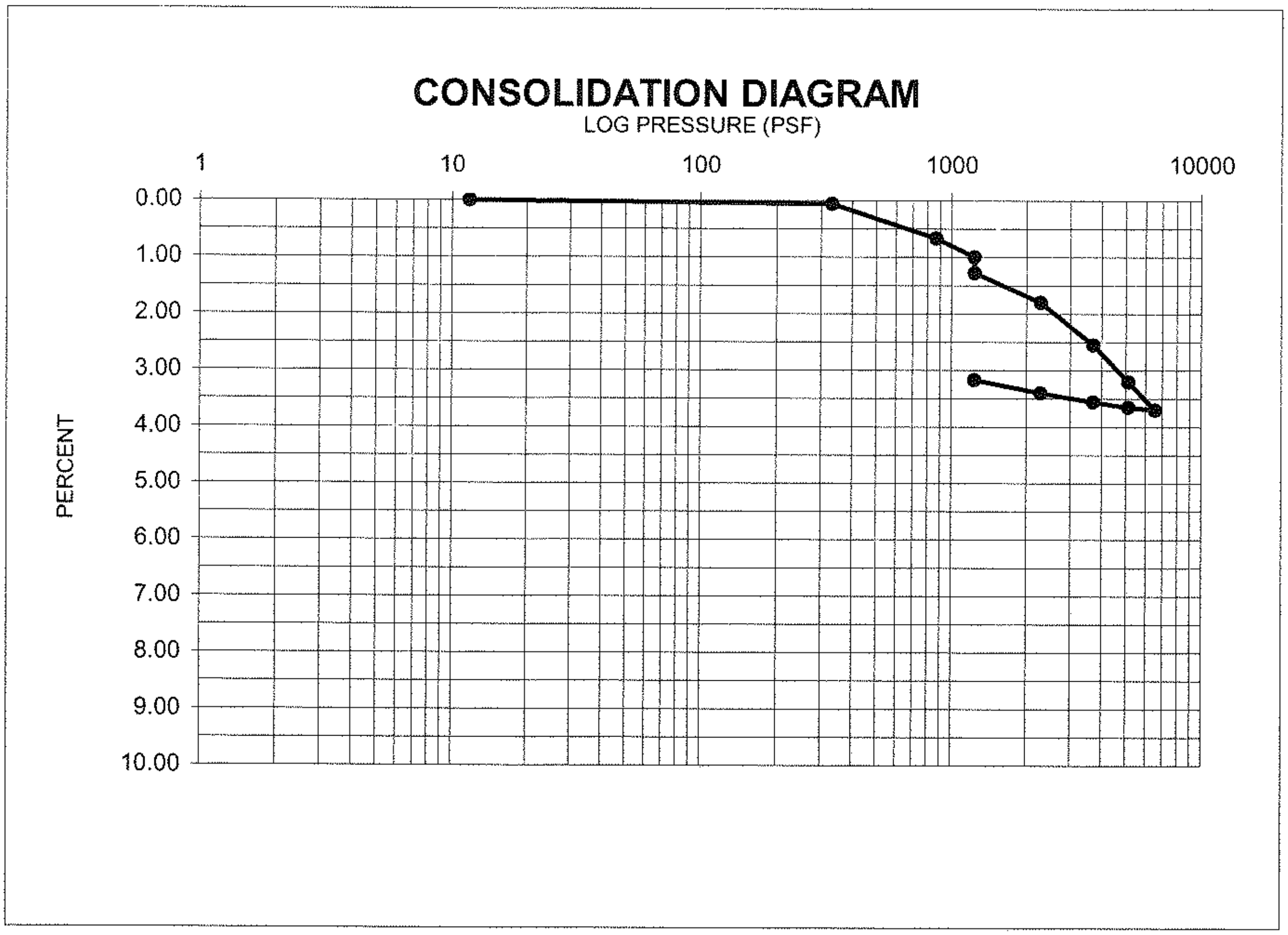
CONSOLIDATION DIAGRAM #3

BG: 21401 CONSULTANT: JET

CLIENT: DWR Construction

Earth Material:	ALLUVIUM
Sample Location:	B1-12.5'
Dry Weight (pcf):	104.3
Initial Moisture:	21.5%
Initial Saturation:	97.3%
Water Added at (psf)	1237

Specific Gravity:	2.65
Initial Void Ratio:	0.59
Compression Index (Cc):	0.074
Recompression Index (Cr):	0.014



BYER GEOTECHNICAL, INC.
1461 E. CHEVY CHASE DR., SUITE 200
GLENDALE, CA 91206
818.549.9959 TEL
818.543.3747 FAX

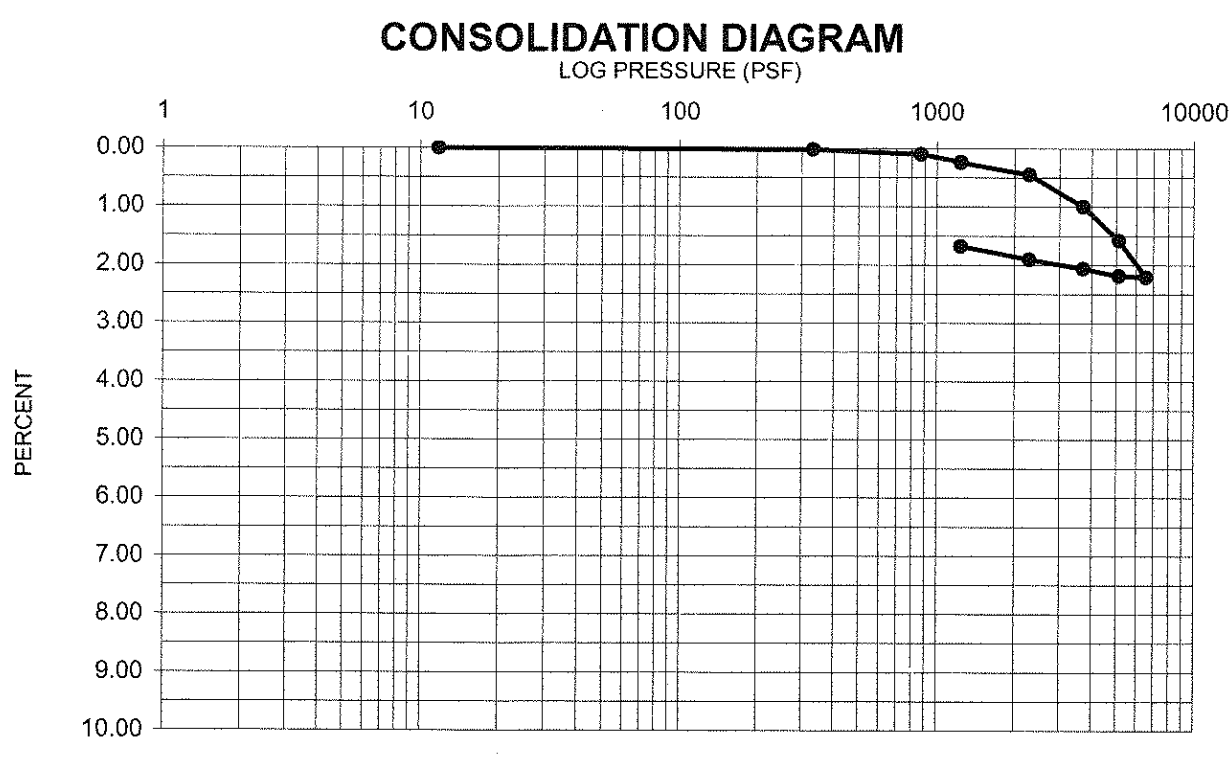
CONSOLIDATION DIAGRAM #4

BG: **21401** CONSULTANT: **JET**

CLIENT: **DWR Construction**

Earth Material:	ALLUVIUM
Sample Location:	B1-20'
Dry Weight (pcf):	96.7
Initial Moisture:	25.4%
Initial Saturation:	94.7%
Water Added at (psf)	1237

Specific Gravity:	2.65
Initial Void Ratio:	0.71
Compression Index (Cc):	0.101
Recompression Index (Cr):	0.016



BYER GEOTECHNICAL, INC.
1461 E. CHEVY CHASE DR., SUITE 200
GLENDALE, CA 91206
818.549.9959 TEL
818.543.3747 FAX

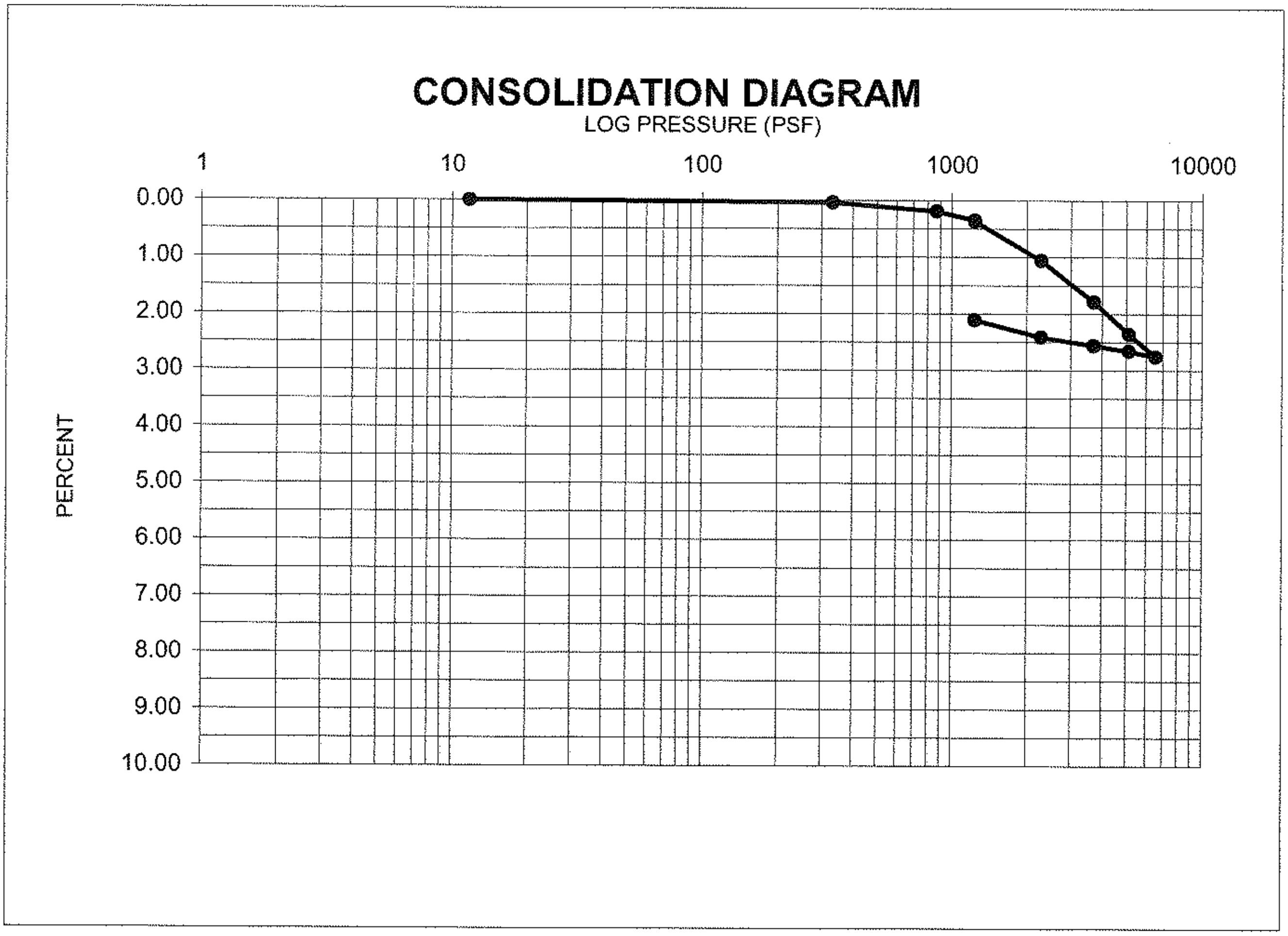
CONSOLIDATION DIAGRAM #5

BG: **21401** CONSULTANT: **JET**

CLIENT: **DWR Construction**

Earth Material:	FUTURE FILL
Sample Location:	90% REMOLD
Dry Weight (pcf):	100.6
Initial Moisture:	15.6%
Initial Saturation:	64.2%
Water Added at (psf)	1237

Specific Gravity:	2.65
Initial Void Ratio:	0.64
Compression Index (Cc):	0.066
Recompression Index (Cr):	0.019



BYER GEOTECHNICAL, INC.

1461 E. CHEVY CHASE DR., SUITE 200
GLENDALE, CA 91206
818.549.9959 TEL
818.543.3747 FAX

LOG OF BORING B1

BG No. 21401

CLIENT DWR Construction, Inc. REPORT DATE 9/20/11 DRILL DATE 8/22/11

PROJECT LOCATION 3700 Coldwater Canyon Avenue, Studio City LOGGED BY JET

CONTRACTOR Choice Drilling DRILLING METHOD Hollow-Stem Auger HOLE SIZE 8-inch diameter

DRIVE WEIGHT 140-Pound Automatic Hammer HAMMER DROP 30 Inches ELEV. TOP OF HOLE 688 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	0	Surface: Level Dirt (ML) **FILL:** 0-3': Clayey Silt, black, dark gray-brown, moist, firm, some rock and concrete fragments up to 4 inches		ML						
685		(ML) **ALLUVIUM:** 3': Clayey Silt, dark brown, moist, firm		ML	Ring Sample	8 10 10	14.9	99.4		
	5				Standard Penetration Test	6 7 8				
680					Ring Sample	12 14 20	19.8	105.6		Direct Shear, Consolidation
	10	(SW-SM) 10.5': Sandy Silt, brown, moist, firm, some clay and gravel		SW-SM	Standard Penetration Test	7 8 9				
675					Ring Sample	13 18 20	97.2	104.3		Consolidation
		(CL) 14': Gravelly Clay, brown, tan, dark gray, moist, firm		CL						
	15				Standard Penetration Test	8 11 14				
		(CL) 17': Gravelly Clay, brown, tan, gray, moist, very dense		CL						
670					Standard Penetration Test	11 13 15				
	20									

Ring Sample | Standard Penetration Test

BYER GEOTECHNICAL, INC.

1461 E. CHEVY CHASE DR., SUITE 200
GLENDALE, CA 91206
818.549.9959 TEL
818.543.3747 FAX

LOG OF BORING B1

BG No. 21401

CLIENT DWR Construction, Inc. **REPORT DATE** 9/20/11 **DRILL DATE** 8/22/11

PROJECT LOCATION 3700 Coldwater Canyon Avenue, Studio City **LOGGED BY** JET

CONTRACTOR Choice Drilling **DRILLING METHOD** Hollow-Stem Auger **HOLE SIZE** 8-inch diameter

DRIVE WEIGHT 140-Pound Automatic Hammer **HAMMER DROP** 30 Inches **ELEV. TOP OF HOLE** 688 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	20	(CL) 20': Sandy Clay, light brown, very moist, dense, some gravel		CL	Standard Penetration Test	18 24 24				Consolidation
665					Standard Penetration Test	9 11 14				
	25	(CL) 24.5': Sandy Clay, more gravel, wet		CL	Standard Penetration Test	10 12 16				
660		**BEDROCK:** 27': Diatomaceous Siltstone and Shale, gray, light gray, soft to moderately hard, contorted, no structure			Standard Penetration Test	12 16 18				
	30				Standard Penetration Test	12 16 18				
655					Ring Sample	50	62.3	60.5		Direct Shear
		33': gray, moderately hard								
	35				Ring Sample	50				

End at 35.5 Feet; No Caving ; Groundwater at 29 Feet;
Fill to 3 Feet

Ring Sample

Standard Penetration Test

BYER GEOTECHNICAL, INC.

1461 E. CHEVY CHASE DR., SUITE 200
GLENDALE, CA 91206
818.549.9959 TEL
818.543.3747 FAX

LOG OF BORING B2

BG No. 21401
PAGE 1 OF 2

CLIENT DWR Construction, Inc. REPORT DATE 9/20/11 DRILL DATE 8/22/11
PROJECT LOCATION 3700 Coldwater Canyon Avenue, Studio City LOGGED BY JET
CONTRACTOR Choice Drilling DRILLING METHOD Hollow-Stem Auger HOLE SIZE 8-inch diameter
DRIVE WEIGHT 140-Pound Automatic Hammer HAMMER DROP 30 Inches ELEV. TOP OF HOLE 682 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	0	Surface: Level Dirt								
		(ML) **FILL:** 0-2': Clayey Silt, black, dark gray-brown, moist, firm, some gravel		ML						
680		(ML) **ALLUVIUM:** 2': Clayey Silt, dark brown, moist, firm		ML						
					Ring Sample	10 11 12	23.1	98		Consolidation
		(CL) 4': Gravelly Clay, tan, light brown, moist, dense		CL						
	5									
					Ring Sample	12 14 18	15.6	100.6		
675										
					Standard Penetration Test	10 12 14				
	10									
					Standard Penetration Test	10 12 16				
		(CL-ML) 11': Silty Clay, dark gray, moist, firm		CL-ML						
670										
					Standard Penetration Test	10 13 15				
		(CL-ML) 14': Silty Clay, brown, light gray		CL-ML						
	15									
					Standard Penetration Test	9 12 15				
665										
					Standard Penetration Test	9 14 15				
	20									

Ring Sample   Standard Penetration Test

BYER GEOTECHNICAL, INC.
1461 E. CHEVY CHASE DR., SUITE 200
GLENDALE, CA 91206
818.549.9959 TEL
818.543.3747 FAX

LOG OF BORING B2

BG No. 21401

CLIENT DWR Construction, Inc. **REPORT DATE** 9/20/11 **DRILL DATE** 8/22/11

PROJECT LOCATION 3700 Coldwater Canyon Avenue, Studio City **LOGGED BY** JET

CONTRACTOR Choice Drilling **DRILLING METHOD** Hollow-Stem Auger **HOLE SIZE** 8-inch diameter

DRIVE WEIGHT 140-Pound Automatic Hammer **HAMMER DROP** 30 Inches **ELEV. TOP OF HOLE** 682 ft

ELEVATION (ft)	DEPTH (ft)	EARTH MATERIAL DESCRIPTION	GRAPHIC SYMBOL	USCS UNIT	SAMPLE TYPE & NUMBER	BLOW COUNT (Per 6 Inches)	MOISTURE CONTENT (%)	DRY UNIT WT. (pcf)	SATURATION (%)	TYPE OF TEST
	20	(SW-SM) 20': Sandy Silt, tan, light gray, moist, dense, some clay		SW-SM	Standard Penetration Test	10 14 20				
660										
		(ML) 22.5': Clayey Silt, brown, gray, moist, firm		ML	Standard Penetration Test	8 12 15				
	25	**BEDROCK:** 25': Diatomaceous Siltstone and Shale, gray, light gray, soft to moderately hard			Standard Penetration Test	11 18 18				
655					Standard Penetration Test	12 18 15				
	30									
		30.5': gray, dark, gray, moderately hard			Ring Sample	25 25 35	39.8	78.7		
650										
	35				Ring Sample	50	35.7	78.8		

End at 35 Feet; No Water; No Caving; Fill to 2 Feet

Ring Sample   Standard Penetration Test

BYER GEOTECHNICAL INC.
1461 East Chevy Chase Drive, Suite 200, Glendale, CA 91206
tel 818.549.9959 fax 818.543.3747

SHORING PILE

BG: **21401** CONSULTANT: **JET**
CLIENT: **DWR CONSTRUCTION**

CALCULATION SHEET # **1**

CALCULATE THE DESIGN MINIMUM EQUIVALENT FLUID PRESSURE (EFP) FOR CANTILEVERED RETAINING WALL. ASSUME BACKFILL IS SATURATED AND THERE IS NO HYDROSTATIC PRESURE THE RETAINED HEIGHT AND BACKSLOPE AND SURCHARGE CONDITIONS ARE LISTED BELOW. USE THE MONONOBE-OKABE METHOD FOR SEISMIC FORCES.

CALCULATION PARAMETERS

EARTH MATERIAL:	ALLUVIUM	RETAINED LENGTH	12 feet
SHEAR DIAGRAM:	2	BACKSLOPE ANGLE:	0
COHESION:	574 psf	SURCHARGE:	300 pounds
PHI ANGLE:	30 degrees	SURCHARGE TYPE:	U Uniform
DENSITY	127 pcf	INITIAL FAILURE ANGLE:	30 degrees
SAFETY FACTOR:	1.5	FINAL FAILURE ANGLE:	70 degrees
PILE FRICTION	0 degrees	INITIAL TENSION CRACK:	3 feet
CD (C/FS):	382.7 psf	FINAL TENSION CRACK:	25 feet

PHID = ATAN(TAN(PHI)/FS) = 21.1 degrees

HORIZONTAL PSEUDO STATIC SEISMIC COEFFICIENT (k_h) 0 %g

VERTICAL PSEUDO STATIC SEISMIC COEFFICIENT (k_v) 0 %g

CALCULATED RESULTS

CRITICAL FAILURE ANGLE	53 degrees
AREA OF TRIAL FAILURE WEDGE	37.4 square feet
TOTAL EXTERNAL SURCHARGE	300.0 pounds
WEIGHT OF TRIAL FAILURE WEDGE	5047.7 pounds
NUMBER OF TRIAL WEDGES ANALYZED	943 trials
LENGTH OF FAILURE PLANE	6.6 feet
DEPTH OF TENSION CRACK	6.7 feet
HORIZONTAL DISTANCE TO UPSLOPE TENSION CRACK	4.0 feet
CALCULATED THRUST ON PILE	**350.4 pounds**
CALCULATED EQUIVALENT FLUID PRESSURE	**4.9 pcf**
DESIGN EQUIVALENT FLUID PRESSURE	**30.0 pcf**

CONCLUSION:

THE CALCULATION INDICATES THAT THE PROPOSED SHORING PILES MAY MAY BE DESIGNED FOR AN EQUIVALENT FLUID PRESSURE OF 30 POUNDS PER CUBIC FOOT. THE FLUID PRESSURE SHOULD BE MULTIPLIED BY THE PILE SPACING.

APPENDIX II

Liquefaction Susceptibility Analysis: SPT Method
(Input Data)

Project No.: 21401 Client: DWR Construction, Inc.

Project Description.: Proposed Pool and Pool House

Engineer: RSB Date: September 13, 2011

Boring No.	Total Depth (ft)	Existing GW Depth (ft)	Historic GW Depth (ft)	Design Fill Height (ft)
B1	35	29	20	5
B2	35	No GW	20	5

Peak Ground Acceleration:	0.52
Earthquake Magnitude:	6.4
Borehole Diamter (inches):	8
Delivered Energy Ratio, ERm (%):	75
Energy Ratio Correction Factor, $C_{E:}$	1.25
Borehole Diameter Correction Factor, $C_{B:}$	1.15
Rod Length Correction Factor, $C_{R:}$	1
Sampler Correction with or without Liners, $C_{S:}$	1
Minimum Factor of Safety, FS_{liq}:	1.2

References: - Idriss, I. M., and Boulanger, R. W. (2008), Soil Liquefaction During Earthquakes, Earthquake Engineering Research Institute (EERI), Monograph No. MNO-12.

- Tokimatsu and Seed (1987), Evaluation of Settlements in Sands due to Earthquake Shaking, American Society for Civil Engineers, Journal of Geotechnical Engineering, Vol. 113, No. 8, August, 1987.

- California Geological Survey (2008), Special Publication 117A, Guidelines for Evaluating and Mitigating Seismic Hazards in California.

- County of Los Angeles, Department of Public Works (2009), Liquefaction/Lateral Spreading, Administrative Manual, Publication No. GS 045-0, May 28, 2009.

APPENDIX II

Liquefaction Susceptibility Analysis: SPT Method (Detailed Calculations)

Factor, C_E = 1.25

Factor, C_B = 1.15

Liners, C_S = 1 (No Liners)

Boring No.	SPT Depth (ft)	Approximate Layer Depth (ft)	Approx. Layer Thick. (ft)	Soil Type (USCS)	Flag "Clay" "Unsaturat "Unreliabl	$CRR_{M=7.5, \sigma'_{vc}=1}$	CRR	Factor of Safety FS_{liq} (Liquefiable/ Non Liquefiable) (Min FS = 1.2)		Post-Liquefaction Reconsolidation Settlement Vol. Strain ε_v	Seismic Settle. (in)	Cum. Settle. (in)
B1	2.5	0 to 3.80	3.80	ML	Compacted	0.245	N/A	N/A	**Non Liq**	0.0000	0.000	0.00
B1	5	3.8 to 6.30	2.50	ML	Compacted	0.376	N/A	N/A	**Non Liq**	0.0000	0.000	0.00
B1	7.5	6.3 to 8.80	2.50	ML		0.622	0.913	2.76	**Non Liq**	0.0000	0.000	0.00
B1	10	8.8 to 11.30	2.50	ML		0.315	0.460	1.41	**Non Liq**	0.0000	0.000	0.00
B1	12.5	11.3 to 13.80	2.50	CL		0.690	0.994	3.09	**Non Liq**	0.0000	0.000	0.00
B1	15	13.8 to 16.30	2.50	CL		1.370	1.200	3.79	**Non Liq**	0.0000	0.000	0.00
B1	17.5	16.3 to 18.80	2.50	GC		1.200	1.200	3.85	**Non Liq**	0.0000	0.000	0.00
B1	20	18.8 to 21.30	2.50	GC		1.200	1.200	3.91	**Non Liq**	0.0000	0.000	0.00
B1	22.5	21.3 to 23.80	2.50	SC		0.646	0.826	2.58	**Non Liq**	0.0000	0.000	0.00
B1	25	23.8 to 26.30	2.50	Bedrock		1.200	1.200	3.63	**Non Liq**	0.0000	0.000	0.00
B1	27.5	26.3 to 28.80	2.50	Bedrock		1.200	1.200	3.54	**Non Liq**	0.0000	0.000	0.00
B1	30	28.8 to 31.30	2.50	Bedrock		1.200	1.200	3.48	**Non Liq**	0.0000	0.000	0.00
B1	32.5	31.3 to 33.80	2.50	Bedrock		1.200	1.200	3.44	**Non Liq**	0.0000	0.000	0.00
B1	35	33.8 to 36.50	2.70	Bedrock		1.200	1.200	3.41	**Non Liq**	0.0000	0.000	0.00
B2	2.5	0 to 3.80	3.80	ML	Compacted	0.320	N/A	N/A	**Non Liq**	0.0000	0.000	0.00
B2	5	3.8 to 6.30	2.50	SP	Compacted	1.035	N/A	N/A	**Non Liq**	0.0000	0.000	0.00
B2	7.5	6.3 to 8.80	2.50	SP		1.200	1.200	3.63	**Non Liq**	0.0000	0.000	0.00
B2	10	8.8 to 11.30	2.50	SP		1.200	1.200	3.68	**Non Liq**	0.0000	0.000	0.00
B2	12.5	11.3 to 13.80	2.50	CL		1.200	1.200	3.73	**Non Liq**	0.0000	0.000	0.00
B2	15	13.8 to 16.30	2.50	CL		1.200	1.200	3.79	**Non Liq**	0.0000	0.000	0.00
B2	17.5	16.3 to 18.80	2.50	CL		1.200	1.200	3.85	**Non Liq**	0.0000	0.000	0.00
B2	20	18.8 to 21.30	2.50	SM		1.200	1.200	3.91	**Non Liq**	0.0000	0.000	0.00
B2	22.5	21.3 to 23.80	2.50	ML		1.040	1.200	3.75	**Non Liq**	0.0000	0.000	0.00
B2	25	23.8 to 26.30	2.50	Bedrock		1.200	1.200	3.63	**Non Liq**	0.0000	0.000	0.00
B2	27.5	26.3 to 28.80	2.50	Bedrock		1.200	1.200	3.54	**Non Liq**	0.0000	0.000	0.00
B2	30	28.8 to 32.50	3.70	Bedrock		1.200	1.200	3.48	**Non Liq**	0.0000	0.000	0.00
B2	35	32.5 to 36.50	4.00	Bedrock		1.200	1.200	3.41	**Non Liq**	0.0000	0.000	0.00

BYER
GEOTECHNICAL
INC.

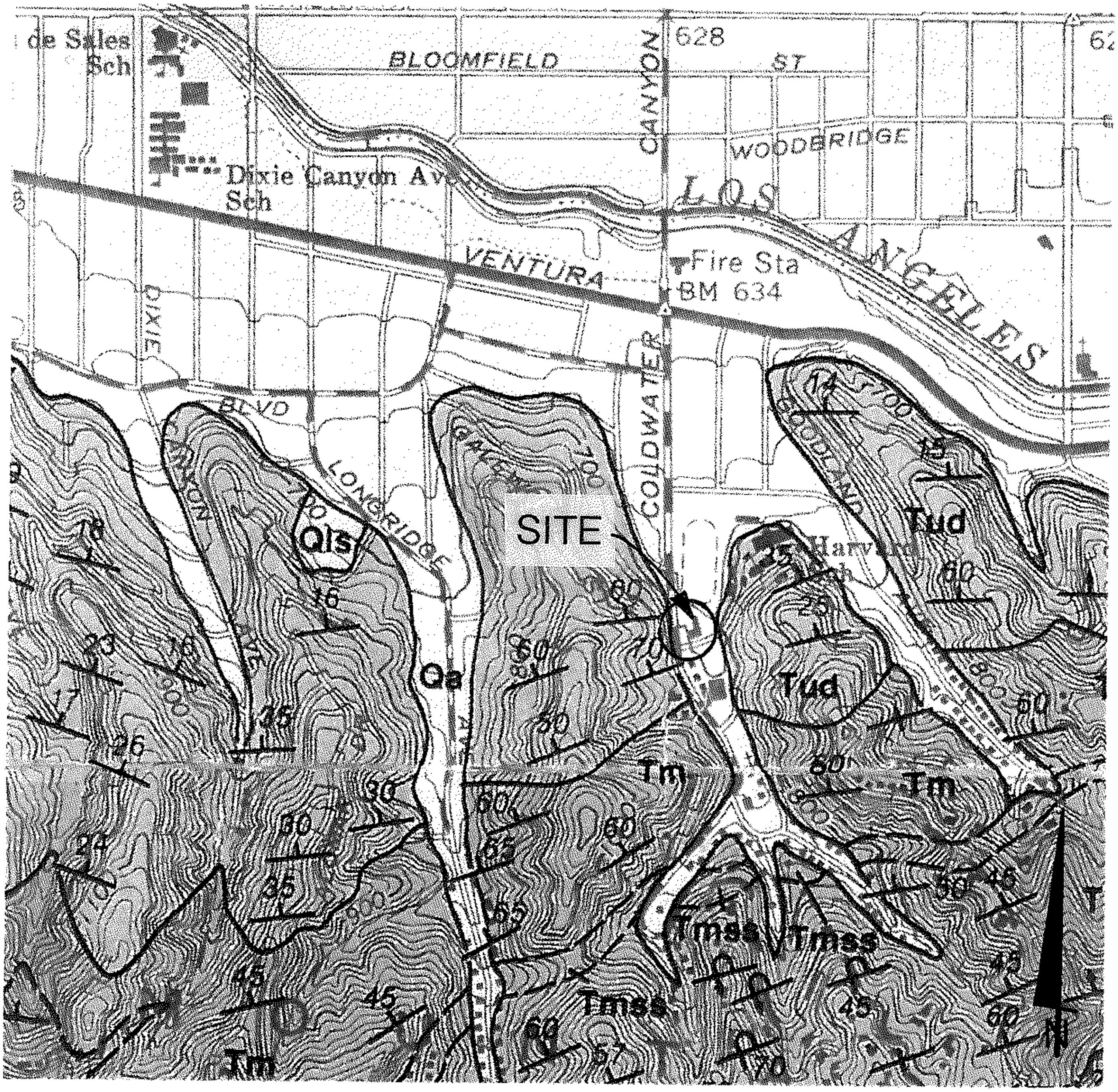
1461 E. CHEVY CHASE DRIVE, # 200, GLENDALE, CA 91206
tel 818.549.9959 fax 818.543.3747

REGIONAL GEOLOGIC MAP

CLIENT: DWR Construction

GEOLOGIST: JET BG: 21401 SCALE: 1" = 1000'

Reference: *Geologic Map of the Beverly Hills and Van Nuys (South ½) Quadrangles*, Dibblee, 1991.



BYER
GEOTECHNICAL
INC.

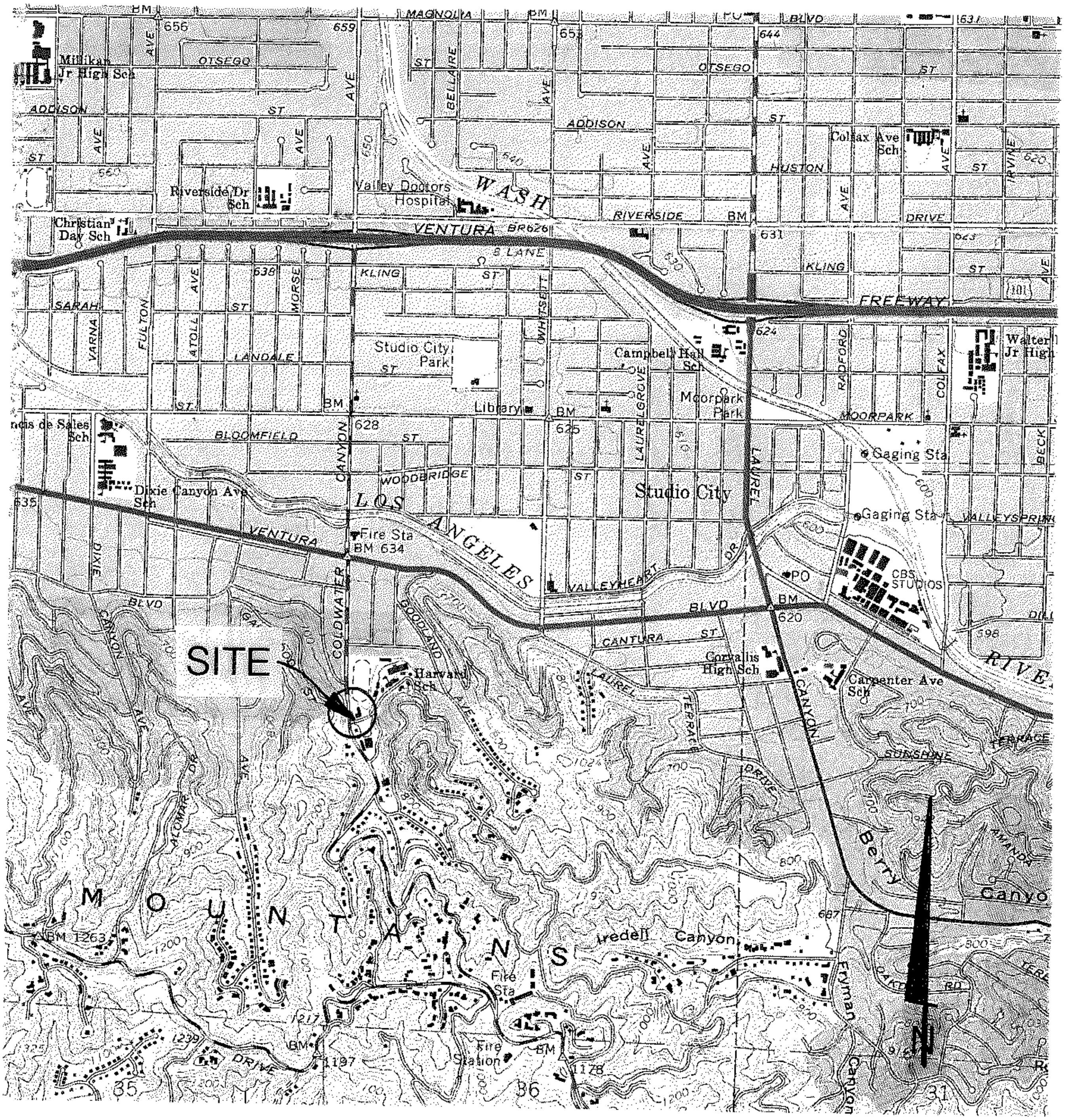
1461 E. CHEVY CHASE DRIVE, # 200, GLENDALE, CA 91206
tel 818.549.9959 fax 818.543.3747

VICINITY MAP

CLIENT: DWR Construction

GEOLOGIST: JET BG: 21401 SCALE: 1" = 2000'

Reference: *U.S.G.S. 7.5 Minute Topographic Map Series, Van Nuys Quadrangle.*



BYER GEOTECHNICAL INC.
1461 E. CHEVY CHASE DR., SUITE 200
GLENDALE, CA 91206
818.549.9959 TEL
818.543.3747 FAX

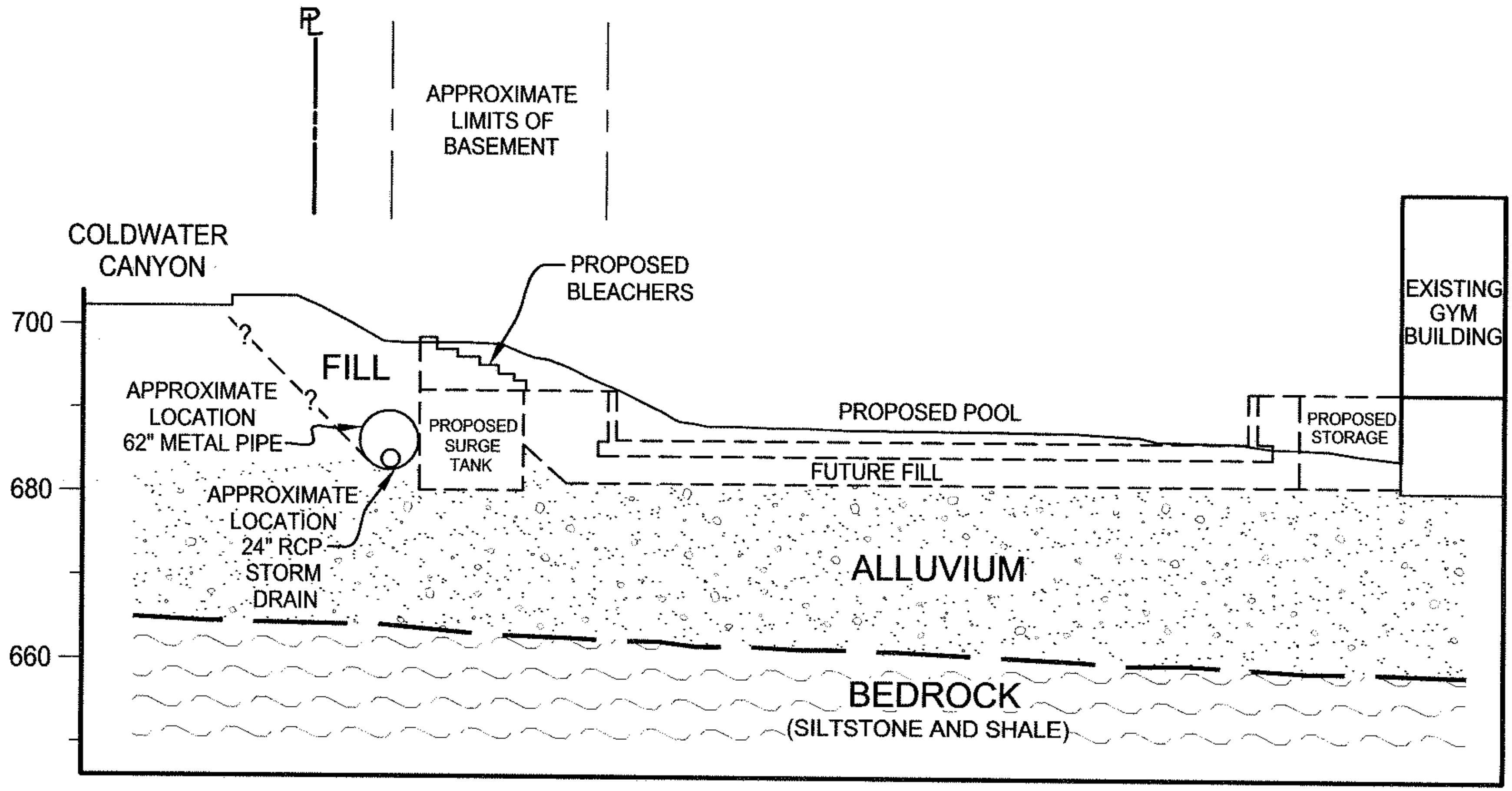
SECTION C

BG: 21401 DWR CONSTRUCTION

CONSULTANT: JET

SCALE: 1" = 20'

SEPTEMBER 20, 2011



SECTION C

S:\LandProjects\21401 DWR CONSTRUCTION\dwg\SITEPLAN_09022011.dwg 9/2/2011 10:08:47 AM PDT

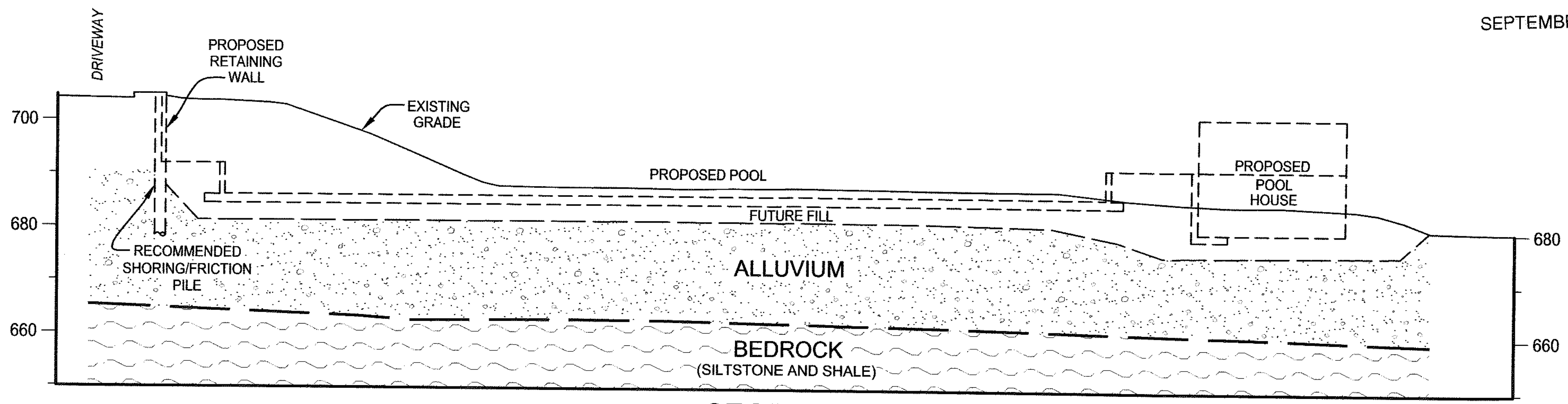
BYER GEOTECHNICAL INC.
1461 E. CHEVY CHASE DR., SUITE 200
GLENDALE, CA 91206
818.549.9959 TEL
818.543.3747 FAX

SECTION A AND B

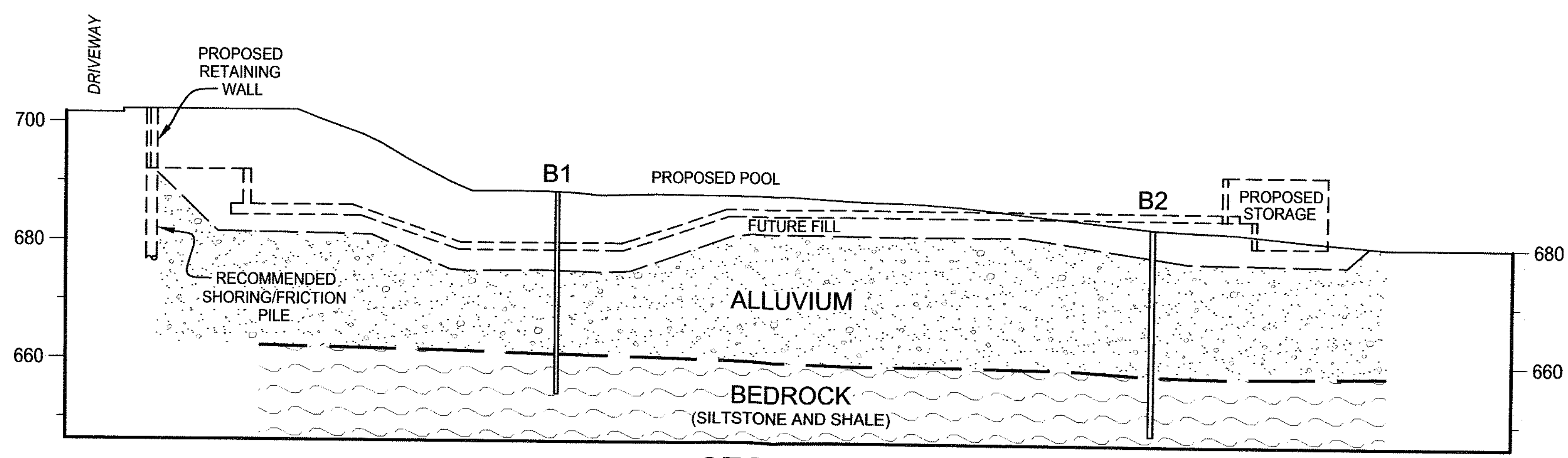
BG: 21401 DWR CONSTRUCTION

CONSULTANT: JET | SCALE: 1" = 20'

SEPTEMBER 20, 2011

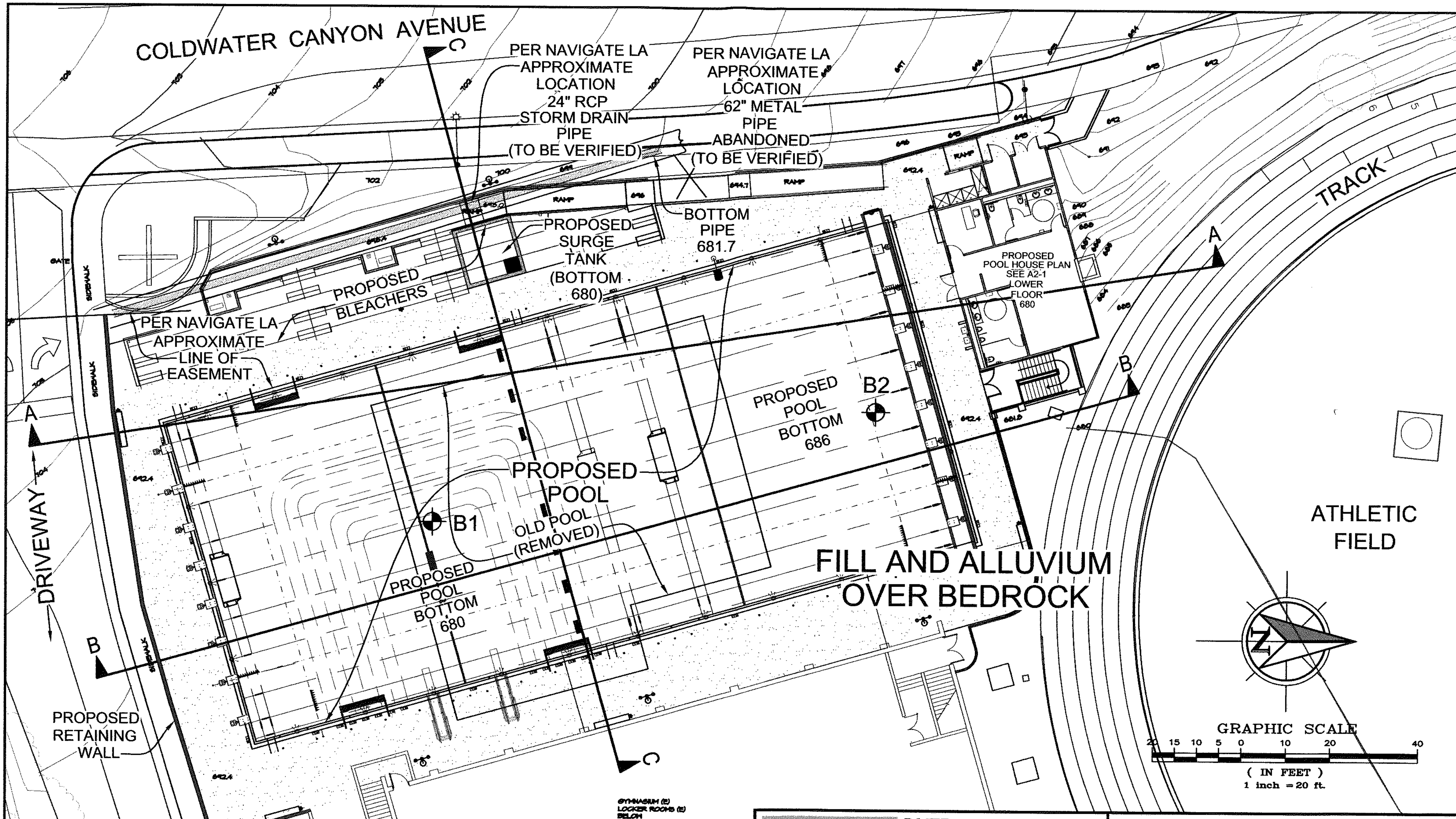


SECTION A



SECTION B

S:\LandProjects\21401 DWR CONSTRUCTION\dwg\SITEPLAN_09022011.dwg 9/2/2011 10:08:47 AM PDT



LEGEND

B2 — LOCATION AND NUMBER OF BORING

SEPTEMBER 20, 2011

REFERENCE: SITE PLAN, SHEET A1-1, PREPARED BY ARCH - PAC AQUATICS, DATED 09/02/2011.

BYER GEOTECHNICAL INC.
1461 E. CHEVY CHASE DR., SUITE 200
GLENDALE, CA 91206
818.549.9959 TEL
818.543.3747 FAX

GEOLOGIC MAP	
BG: 21401 DWR CONSTRUCTION	
CONSULTANT: JET	SCALE: 1" = 20'

S:\LandProjects\21401 DWR CONSTRUCTION\dwg\SITEPLAN_09022011.dwg 9/2/2011 10:08:47 AM PDT

CITY OF LOS ANGELES

CALIFORNIA

ANTONIO R. VILLARAIGOSA
MAYOR

DEPARTMENT OF
BUILDING AND SAFETY
201 NORTH FIGUEROA STREET
LOS ANGELES, CA 90012

ROBERT R. "BUD" OVROM
GENERAL MANAGER

RAYMOND S. CHAN, C.E., S.E.
EXECUTIVE OFFICER

GEOLOGY AND SOILS REPORT APPROVAL LETTER

October 28, 2011

LOG # 75188
SOILS/GEOLOGY FILE - 2
LIQ

Harvard-Westlake School
3700 N. Coldwater Canyon Avenue
Los Angeles, CA 91604

TRACT:	1000
LOT(S):	1111 (arb. 1)
LOCATION:	3700 N. Coldwater Canyon Avenue

CURRENT REFERENCE REPORT/LETTER(S)	REPORT NO.	DATE(S) OF DOCUMENT	PREPARED BY
Geology/Soil Report	BG 21401	09/20/2011	Byer Geotechnical

The referenced report concerning the proposed construction of a new pool and pool house on the subject school property, has been reviewed by the Grading Division of the Department of Building and Safety.

Based on the report, the previous pool and pool house were demolished and removed. The proposed pool and pool house are shown on the geologic map and sections A & B. A retaining wall is planned along the south side of the pool, north of the main driveway. Bleachers and a 15 feet by 15 feet surge tank structure (with retaining walls), are proposed as shown on the geologic map and Section C. This section also shows the easement along the west side of the proposed pool and along the area east of Coldwater Canyon Avenue.

The excavation into the slope area south of the pool will be shored to maintain support to the existing main driveway to the south.

New certified compacted fill and the naturally-occurring alluvium are the recommended bearing material for the structures.

The site is located in a designated liquefaction hazard zone as shown on the "Seismic Hazard Zones" map issued by the State of California. The Liquefaction study included as a part of the report demonstrates that the site does not possess a liquefaction potential. This satisfies the requirement of the 2011 Los Angeles City Building Code Section 1802.2.7.

The referenced report is acceptable, provided the following conditions are complied with during site development:

(Note: Numbers in parenthesis ( ) refer to applicable sections of the 2011 City of LA Building Code. P/BC numbers refer the applicable Information Bulletin. Information Bulletins can be accessed on the internet at LADBS.ORG.)

1. The geologist and soils engineer shall review and approve the detailed plans prior to issuance of any permits. This approval shall be by signature on the plans which clearly indicates that the geologist and soils engineer have reviewed the plans prepared by the design engineer and that the plans include the recommendations contained in their reports. (7006.1)

2. All recommendations of the report(s) which are in addition to or more restrictive than the conditions contained herein shall be incorporated into the plans.

3. A copy of the subject and appropriate referenced reports and this approval letter shall be attached to the District Office and field set of plans. Submit one copy of the above reports to the Building Department Plan Checker prior to issuance of the permit. (7006.1)

4. Approval shall be obtained from the utility company with regard to proposed construction within or adjacent to the utility easement.

5. A grading permit shall be obtained for all structural fill and retaining wall backfill. (106.1.2)

6. All man-made fill shall be compacted to a minimum 90 percent of the maximum dry density of the fill material per the latest version of ASTM D 1557. Where cohesionless soil having less than 15 percent finer than 0.005 millimeters is used for fill, it shall be compacted to a minimum of 95 percent relative compaction based on maximum dry density (D1556). Placement of gravel in lieu of compacted fill is allowed only if complying with Section 91.7011.3 of the Code. (7011.3)

7. If import soils are used, no footings shall be poured until the soils engineer has submitted a compaction report containing in-place shear test data and settlement data to the Grading Division of the Department, and obtained approval. (7008.2)

8. Compacted fill shall extend beyond the footings a minimum distance equal to the depth of the fill below the bottom of footings or a minimum of three feet whichever is greater.

9. Existing uncertified fill shall not be used for support of footings, concrete slabs or new fill. (1809.2)

10. The applicant is advised that the approval of this report does not waive the requirements for excavations contained in the State Construction Safety Orders enforced by the State Division of Industrial Safety. (3301.1)

11. The soils engineer shall review and approve the shoring plans prior to issuance of the permit. (3307.3.2)

12. Prior to the issuance of the permits, the soils engineer and/or the structural designer shall evaluate the surcharge loads used in the report calculations for the design of the retaining walls and shoring. If the surcharge loads used in the calculations do not conform to the actual surcharge loads, the soil engineer shall submit a supplementary report with revised recommendations to the Department for approval.

13. Unsurcharged temporary excavations over 5 feet exposing alluvium and any portion of the excavation exposing fill shall be trimmed back at a gradient not exceeding 1:1, as recommended.

14. Shoring shall be designed for a minimum EFP of 30 PCF; all surcharge loads shall be included into the design, as recommended.

15. Shoring shall be designed for a maximum lateral deflection of ½ inch where a structure is within a 1:1 plane projected up from the base of the excavation, as recommended.

16. A shoring monitoring program shall be implemented to the satisfaction of the soils engineer.

17. Pool and pool house foundations shall derive entire support from a blanket of properly placed fill a minimum of 3 feet thick, and the surge tank and retaining wall shall derive entire support from native undisturbed soils, as recommended and approved by the geologist and soils engineer by inspection.

18. Footings supported on approved compacted fill or expansive soil shall be reinforced with a minimum of four (4) ½-inch diameter (#4) deformed reinforcing bars. Two (2) bars shall be placed near the bottom and two (2) bars placed near the top.

19. The seismic design shall be based on a Site Class D as recommended. All other seismic design parameters shall be reviewed by LADBS building plan check.

20. Retaining walls shall be designed for the minimum lateral earth pressures specified in the section titled "Retaining Walls" starting on page 14 of the report. All surcharge loads shall be included into the design.

21. All retaining walls shall be provided with a standard surface backdrain system and all drainage shall be conducted to the street in an acceptable manner and in a non-erosive device. (7013.11)

22. With the exception of retaining walls designed for hydrostatic pressure, all retaining walls shall be provided with a subdrain system to prevent possible hydrostatic pressure behind the wall. Prior to issuance of any permit, the retaining wall subdrain system recommended in the soil report shall be incorporated into the foundation plan which shall be reviewed and approved by the soils engineer of record. (1805.4)

23. Installation of the subdrain system shall be inspected and approved by the soils engineer of record and the City grading/building inspector. (108.9)

24. Prefabricated drainage composites (Miradrain) (Geotextiles) may be only used in addition to traditionally accepted methods of draining retained earth.

25. The proposed swimming pool walls shall be designed for a freestanding condition, as recommended. (1808.7.3)

26. The proposed swimming pool walls shall be designed for an inward pressure of 43 pcf, as recommended.

27. All pool deck, roof and pad drainage shall be conducted to the street in an acceptable manner. (7013.10)

28. The geologist and soils engineer shall inspect all excavations to determine that conditions anticipated in the report have been encountered and to provide recommendations for the correction of hazards found during grading. (7008 & 1704.7)

29. Prior to the pouring of concrete, a representative of the consulting soils engineer shall inspect and approve the footing excavations. He shall post a notice on the job site for the LADBS Building Inspector and the Contractor stating that the work so inspected meets the conditions of the report, but that no concrete shall be poured until the City Building Inspector has also inspected and approved the footing excavations. A written certification to this effect shall be filed with the Grading Division of the Department upon completion of the work. (108.9 & 7008.2)

30. Prior to excavation, an initial inspection shall be called with LADBS Inspector at which time sequence of shoring, protection fences and dust and traffic control will be scheduled. (108.9.1)

31. Installation of shoring shall be performed under the continuous inspection and approval of the soils engineer and deputy grading inspector. (1704.7)

32. The installation and testing of tie-back anchors shall comply with the recommendations included in the report or the standard sheets titled "Requirement for Tie-back Earth Anchors", whatever is more restrictive. (Research Report #23835)

33. Prior to the placing of compacted fill, a representative of the soils engineer shall inspect and approve the bottom excavations. He shall post a notice on the job site for the City Grading Inspector and the Contractor stating that the soil inspected meets the conditions of the report, but that no fill shall be placed until the LADBS Grading Inspector has also inspected and approved the bottom excavations. A written certification to this effect shall be included in the final compaction report filed with the Grading Division of the Department. All fill shall be placed under the inspection and approval of the soils engineer. A compaction report together with the approved soil report and Department approval letter shall be submitted to the Grading Division of the Department upon completion of the compaction. In addition, an Engineer's Certificate of Compliance with the legal description as indicated in the grading permit and the permit number shall be included. (7011.3)

34. No footing/slab shall be poured until the compaction report is submitted and approved by the Grading Division of the Department.

STEPHEN DAWSON
Engineering Geologist II

J. ADOLFO ACOSTA
Geotechnical Engineer I

SD/JAA:sd/jaa
Log No. 75188
213-482-0480

cc: Jim Dematte, Applicant
Byer Geotechnical, Inc., Project Consultant
VN District Office

BYER GEOTECHNICAL, INC.

October 13, 2011
BG 21256

Harvard-Westlake School
700 North Faring Road
Los Angeles, California 90077

Attention: Mr. Jim DeMatte

Subject

Transmittal of Addendum Geologic and Soils Engineering Exploration
Response to City of Los Angeles Correction Letter
Proposed Brendon Kutler Center and Mudd Library Renovation
Arb. 1, Portion of Lot 1111, Tract 1000
3700 North Coldwater Canyon Avenue
North Hollywood, California

Gentlepersons:

Byer Geotechnical has completed our addendum report, which provides the information requested in the City of Los Angeles, Department of Building and Safety (LADBS), Geology and Soils Report Correction Letter, dated August 10, 2011. The reviewing agency for this document is the LADBS. The reviewing agency requires three unbound copies, one with a wet signature, an application form, and a filing fee. Copies have been distributed as follows:

(1) Addressee (E-mail and Mail)
(1) Tobias Architecture (E-mail)
(1) John A. Martin & Associates, Attention: Kurt Clandening (E-mail)
(3) City of Los Angeles, Department of Building and Safety

Byer Geotechnical will file the report with the LADBS. Any questions concerning the report should be directed to the undersigned. Byer Geotechnical appreciates the opportunity to continue to offer our consultation and advice on this project.

Very truly yours,
BYER GEOTECHNICAL, INC.

James Tucker

James E. Tucker
Project Consultant

BYER GEOTECHNICAL, INC.

ADDENDUM GEOLOGIC AND SOILS ENGINEERING EXPLORATION

RESPONSE TO CITY OF LOS ANGELES CORRECTION LETTER

PROPOSED BRENDON KUTLER CENTER AND MUDD LIBRARY RENOVATION

ARB. 1, PORTION OF LOT 1111, TRACT 1000

3700 NORTH COLDWATER CANYON AVENUE

NORTH HOLLYWOOD, CALIFORNIA

FOR HARVARD-WESTLAKE SCHOOL

BYER GEOTECHNICAL, INC., PROJECT NUMBER BG 21256

OCTOBER 31, 2011

1461 East Chevy Chase Drive, Suite 200 • Glendale, California 91206 • tel 818.549.9959 • fax 818.543.3747 • www.byergeo.com

ADDENDUM GEOLOGIC AND SOILS ENGINEERING EXPLORATION
RESPONSE TO CITY OF LOS ANGELES CORRECTION LETTER
PROPOSED BRENDON KUTLER CENTER AND MUDD LIBRARY RENOVATION
ARB. 1, PORTION OF LOT 1111, TRACT 1000
3700 NORTH COLDWATER CANYON AVENUE
NORTH HOLLYWOOD, CALIFORNIA
FOR HARVARD-WESTLAKE SCHOOL
BYER GEOTECHNICAL, INC., PROJECT NUMBER BG 21256
OCTOBER 31, 2011

References: **Report by Byer Geotechnical, Inc.:**

Geologic and Soils Engineering Exploration, Proposed Brendon Kutler Center and Mudd Library Renovation, Arb. 1, Portion of Lot 1111, Tract 1000, 3700 North Coldwater Canyon Avenue, North Hollywood, California, dated December 30, 2010.

Response by the City of Los Angeles, Department of Building and Safety (LADBS):

Geology and Soils Report Correction Letter, Log # 74548, dated August 10, 2011.

Gentlepersons:

This addendum geologic and soils engineering exploration has been prepared to provide the additional information requested by the LADBS in the Geology and Soils Report Correction Letter, dated August 10, 2011. A copy of the Correction Letter is enclosed with this report. The items requested in the City letter are listed below, followed by Byer Geotechnical's item-by-item response.

Item 1. *Provide for review purposes, one complete copy (including all diagrams, maps and sections) of the 08/26/1966 report by Leroy Crandall & Associates, and the associated review letter/s.*

Response: A complete copy of the report by Leroy Crandall & Associates (LCA), dated August 26, 1966, is enclosed. The LADBS review letter for this report was not found during research of the records.

Item 2. *Provide for review purposes, one complete copy (including all diagrams, maps and sections) of the 01/29/1973 report by Geology & Soils Consultants referenced on pg. 10, and the associated review letter.*

Response: A copy of the report by Geology and Soils Consultants, Inc., dated January 29, 1973, was provided by the client and is enclosed. The Geologic Map and the associated LADBS review letter for this report were not located.

Item 3. *Provide the geologic cross-section or sections which formed the basis for the calculations that previously demonstrated that the ancient landslide debris on the school property is stable, and all approvals from the department (from the grading division or from the board of commissioners). Note: Please verify that the items requested in items 1, 2 & 3 of this letter, are referenced.*

Response: The geologic cross section that formed the basis for the previous calculations was not located. The reports described in the items above are included on the enclosed Prior Work reference list.

Item 4. *It appears that the Board of Building & Safety Commissioners in their previous action (1967), granted approval to construct an addition to the school within an ancient landslide mass on the basis of certain key provisions. Summarize these key provisions in detail, confirm that they will be adhered to as part of the currently proposed construction, and/or provide suitable other recommendations if they will not.*

Response: A copy of the Board of Building and Safety Commissioners letter, Board File #670179, dated January 30, 1967, is enclosed. Following are the provisions of the letter:

1. *Granted approval to construct the addition on an ancient landslide area, provided:*

   a. *That the petitioner clearly understands that inasmuch as the building is on a landslide area which may at some future time move, damage may occur after construction.*

   b. *Piezometers are installed to indicate any rise in the ground water level. The piezometers shall be read at six month intervals and a report containing the readings shall be submitted to the Department for approval.*

   c. *All recommendations contained in the Foundation Engineers and Geologist Reports are complied with.*

2. *Granted approval to use bearing values in access* [excess] *of Code allowables in the design of spread footings as contained in the Foundation Investigation Report dated August 26, 1966, by LeRoy Crandall and Associates.*

3. *Granted approval to use lateral bearing values in access* [excess] *of Code allowables for use in the Code 'flagpole formula' as contained in the Foundation Investigation Report.*

4. *Granted approval to use friction values in access* [excess] *of code allowables in the design of cast-in-place friction piles as contained in the Foundation Investigation Report.*

5. *Denied the request to disregard the pile efficiency equation in the design of the pile groups.*

It is not known if the piezometers requested in item 1.b. were installed. The piezometers were intended to verify that site drainage control recommended by LCA was effective in preventing groundwater from developing under the site. However, no groundwater was encountered in the recent boring to a depth of 50 feet. The various bearing values referred to will not be exceeded by the current design recommendations.

Item 5. *Plot all locations explored previously by Leroy Crandall & Associates on the school site, incorporate the existing geologic information into the current evaluation, verify and revise the current map and/or sections, accordingly.*

Response: The locations of the test pits and borings performed by LCA and included in the report dated August 26, 1966, are approximately shown on the enclosed Revised Geologic Map. Section A and the slope stability analysis have been revised based on the geologic information obtained from the LCA exploration.

Item 6. *Discuss and provide all necessary geotechnical recommendations for renovation of the existing library and for all improvements to the academic center.*

Response: No improvements are planned for the academic building. It is proposed to construct additional foundations as part of the renovation and remodel of the library building, which includes an addition connecting the library to the academic building. The addition foundations will be independent of the academic building foundations. The foundations for the library were exposed in the test pits. The foundations were observed to be in the ancient landslide debris. It is recommended new foundations be founded in the ancient landslide debris per recommendations contained in the "Foundation Design" section of the referenced Byer Geotechnical report.

Item 7. *If building plans are available now, utilize them to present the proposed construction in the addendum, and also provide a geologic cross-section drawn in the approximate west to east direction to show the proposed construction relative to the existing sub-surface conditions and existing foundations for the academic center and the library.*

Response: Building plans are available and were used to prepare the enclosed Geologic Map 2 showing the proposed Brendon Kutler Center, existing buildings, borings, and test pits. Also, a new section, Section B, shows the proposed construction, existing buildings, and distribution of the earth materials. The proposed Brendon Kutler Center is to be founded on a deepened foundation system consisting of friction piles.

Item 8. *Clarify if the proposed 2-story Brendon Kutler Center is an addition to the existing library and the academic center, and if it will or will not exceed 50% of the replacement value of either building.*

Response: The proposed Brendon Kutler Center is an addition to the library and academic center. The proposed addition will not exceed 50 percent of the replacement value of either the academic center or the library.

Item 9. *The report did not include any recommendations for retaining walls. Verify, and show the entire length of any retaining walls proposed as part of the proposed work.*

Response: No retaining walls are planned as part of the proposed work.

Item 10. *Supplement the slope stability analyses with circular slope stability analysis considering potential failure planes through the ancient landslide material and, through the landslide material and the bedrock.*

Response: Circular slope stability trails have also been performed using Section A, both through the ancient landslide material and the bedrock, as requested. The results indicate a factor of safety still well above 1.5 and are enclosed.

BYER GEOTECHNICAL, INC.
1461 East Chevy Chase Drive, Suite 200 • Glendale, California 91206 • tel 818.549.9959 • fax 818.543.3747 • www.byergeo.com

Byer Geotechnical appreciates the opportunity to continue to provide our service on this project. Any questions concerning the data or interpretation of this report should be directed to the undersigned.

Respectfully submitted,
BYER GEOTECHNICAL, INC.

REGISTERED PROFESSIONAL ENGINEER
ROBERT I. ZWEIGLER
No. 2120
Exp. 06-30-12
GEOTECHNICAL
STATE OF CALIFORNIA

James E. Tucker
P. E. 6628

Robert I. Zweigler
E. G. 1210/G. E. 2120

JET:RIZ:mh
S:\FINAL\BG\21256_Harvard-Westlake\21256_Harvard-Westlake_Addendum_Geo_and_Soils.wpd

Enc: List of Prior Work (2 Pages)
LADBS, Geology and Soils Report Correction Letter, dated August 10, 2011 (2 Pages)
Board of Building and Safety Commissioners, letter dated January 30, 1967 (Corrected)
Stability Calculations (12 Pages)
Appendix I - Leroy Crandall and Associates, report dated August 26, 1966
Appendix II - Geology and Soils Consultants, Inc., report dated January 29, 1973
Appendix III - Figures
Section B
Revised Section A
Geologic Map 2

In Pocket: Revised Geologic Map

xc: (1) Addressee (E-mail and Mail)
(1) Tobias Architecture (E-mail)
(1) John A. Martin & Associates, Attention: Kurt Clandening (E-mail)
(3) City of Los Angeles, Department of Building and Safety (BG to Submit)

PRIOR WORK

Geology and Soils Consultants, Inc.:

Geologic Engineering Investigation, Proposed Library and Field House, 3700 Coldwater Canyon Avenue, Los Angeles, California, dated January 29, 1973.

Epsilon Engineering & Inspection, Inc.:

Report of Preliminary Soil Investigation, Harvard School, Gallery Basement, 3700 Coldwater Canyon, North Hollywood, California, dated March 14, 1997; and

Report of Preliminary Soil Investigation for Harvard School, Upper Level, 3700 Coldwater Canyon, North Hollywood, California, dated January 10, 1991.

Converse Consultants West:

Geotechnical Investigation, Proposed Science Building, Harvard Westlake School, 3700 Coldwater Canyon Avenue, Studio City, California, dated April 22, 1994.

LeRoy Crandall and Associates:

Report of Foundation Investigation, Proposed School Additions, 3700 Coldwater Canyon Drive, Los Angeles, California, for the Harvard School, dated August 26, 1966;

Inspection of Caisson Excavations, Proposed Subsurface Drainage System, Inspection of Foundation Excavations and Inspection and Testing of Compacted Fill, Proposed Lower School Building - Harvard School, 3700 Coldwater Canyon Drive, Los Angeles, California, dated November 17, 1967;

Placement of Rock Backfill, Academic Center, Harvard School, 3700 Coldwater Canyon Avenue, Los Angeles, California, dated December 29, 1969; and

Review of Foundation Recommendations, Proposed Academic Center, 3700 Coldwater Canyon Drive, Los Angeles, California, dated August 12, 1968.

The J. Byer Group, Inc. (JB 17866-B):

Geologic and Soils Engineering Exploration, Proposed Parking Lot Extension and Gymnasium Addition, Portion of Lot 1111, Tract 1000, 3700 Coldwater Canyon Avenue, North Hollywood, California, dated October 16, 1998; and

PRIOR WORK (Continued)

Geotechnical Engineering Exploration, Proposed Sports-Field Lighting, Lot 1111, Tract 1000, 3700 Coldwater Canyon Avenue, Studio City, California, dated October 9, 2006.

Responses by the City of Los Angeles, Department of Building and Safety (LADBS):

Conditional approval letter, Log # 27150, dated March 18, 1999, and Geology/Soil Report Approval Letter, Log # 56969, dated January 23, 2007.

Report by Byer Geotechnical, Inc.:

Geologic and Soils Engineering Exploration, Proposed Brendon Kutler Center and Mudd Library Renovation, Arb. 1, Portion of Lot 1111, Tract 1000, 3700 North Coldwater Canyon Avenue, North Hollywood, California, dated December 30, 2010.

Response by the LADBS:

Geology and Soils Report Correction Letter, Log # 74548, dated August 10, 2011.

CITY OF LOS ANGELES
CALIFORNIA

ANTONIO R. VILLARAIGOSA
MAYOR

DEPARTMENT OF
BUILDING AND SAFETY
201 NORTH FIGUEROA STREET
LOS ANGELES, CA 90012

ROBERT R. "BUD" OVROM
GENERAL MANAGER

RAYMOND S. CHAN, C.E., S.E.
EXECUTIVE OFFICER

GEOLOGY AND SOILS REPORT CORRECTION LETTER

August 10, 2011

Log # 74548
SOILS/GEOLOGY FILE - 2

Harvard-Westlake School
3700 N. Coldwater Canyon Avenue
Los Angeles, CA 91604

TRACT: 1000
LOT: 1111 (arb. 1)
LOCATION: 3700 N. Coldwater Canyon Avenue

CURRENT REFERENCE REPORT/LETTER(S)	REPORT NO.	DATE(S) OF DOCUMENT	PREPARED BY
Geology/Soil Report	BG 21256	12/30/2010	Byer Geotechnical
Oversized Document	"	"	"

The referenced report concerning renovation of the Mudd Library and construction of a new two-story building (the Brendon Kutler Center) to be located between the Mudd Library to the east and the Seaver Academic Center to the west, has been reviewed by the Grading Division of the Department of Building and Safety. The review of the subject report cannot be completed at this time and will be continued upon submittal of an addendum to the report which shall include, but need not be limited to, the following:

1. Provide for review purposes, one complete copy (including all diagrams, maps and sections) of the 08/26/1966 report by Leroy Crandall & Associates, and the associated review letter/s.

2. Provide for review purposes, one complete copy (including all diagrams, maps and sections) of the 01/29/1973 report by Geology & Soils Consultants referenced on pg. 10, and the associated review letter.

3. Provide the geologic cross-section or sections which formed the basis for the calculations that previously demonstrated that the ancient landslide debris on the school property is stable, and all approvals from the department (from the grading division or from the board of commissioners). Note: Please verify that the items requested in items 1, 2 & 3 of this letter, are referenced.

4. It appears that the Board of Building & Safety Commissioners in their previous action

(1967), granted approval to construct an addition to the school within an ancient landslide mass on the basis of certain key provisions. Summarize these key provisions in detail, confirm that they will be adhered to as part of the currently proposed construction, and/or provide suitable other recommendations if they will not.

5. Plot all locations explored previously by Leroy Crandall & Associates on the school site, incorporate the existing geologic information into the current evaluation, verify and revise the current map and/or sections, accordingly.

6. Discuss and provide all necessary geotechnical recommendations for renovation of the existing library and for all improvements to the academic center.

7. If building plans are available now, utilize them to present the proposed construction in the addendum, and also provide a geologic cross-section drawn in the approximate west to east direction to show the proposed construction relative to the existing sub-surface conditions and existing foundations for the academic center and the library.

8. Clarify if the proposed 2-story Brendon Kutler Center is an addition to the existing library and the academic center, and if it will or will not exceed 50% of the replacement value of either building.

9. The report did not include any recommendations for retaining walls. Verify, and show the entire length of any retaining walls proposed as part of the proposed work.

10. Supplement the slope stability analyses with circular slope stability analysis considering potential failure planes through the ancient landslide material and, through the landslide material and the bedrock.

The licensed geologist and soil engineer shall prepare an addendum response report containing the corrections indicated in this letter. The report shall be in the form of an itemized response. It is recommended that the undersigned reviewers be contacted at the phone number listed below, to schedule an appointment to verify that the report contains the required information. Please do not schedule an appointment until all correction items have been addressed. Bring three copies of the addendum response report, including one unbound wet-signed original for microfilming, in the event that the report is found to be acceptable.

STEPHEN DAWSON
Engineering Geologist II

J. ADOLFO ACOSTA
Geotechnical Engineer I

Log # 74548
(213) 482-0480

cc: Lester Tobias (Applicant)
Byer Geotechnical
VN District Office

January 30, 1967

Board File #670179

Harvard School
3700 Coldwater Canyon Road
North Hollywood, California 91604

CORRECTED LETTER

3700 COLDWATER CANYON DRIVE
TRACT 1000, LOT 1111

The Board of Building and Safety Commissioners at its meeting of January 30, 1967, gave consideration to the request concerning the construction of an addition to a school at the above referenced property and took the following action:

1. Granted approval to construct the addition on an ancient landslide area, provided:

    a. That the petitioner clearly understands that inasmuch as the building is on a landslide area which may at some future time move, damage may occur after construction.

    b. Piezometers are installed to indicate any rise in the ground water level. *The piezometers shall be read at six month intervals and a report containing the readings shall be submitted to the Department for approval.

    c. All recommendations contained in the Foundation Engineers and Geologist Reports are complied with.

2. Granted approval to use bearing values in excess of Code allowables in the design of spread footings as contained in the Foundation Investigation Report dated August 26, 1966, by LeRoy Crandall and Associates.

3. Granted approval to use lateral bearing values in excess of Code allowables for use in the Code "flagpole formula" as contained in the Foundation Investigation Report.

4. Granted approval to use friction values in excess of Code allowables in the design of cast-in-place friction piles as contained in the Foundation Investigation Report.

0510004000131

5. Denied the request to disregard the pile efficiency equation in the design of the pile groups.

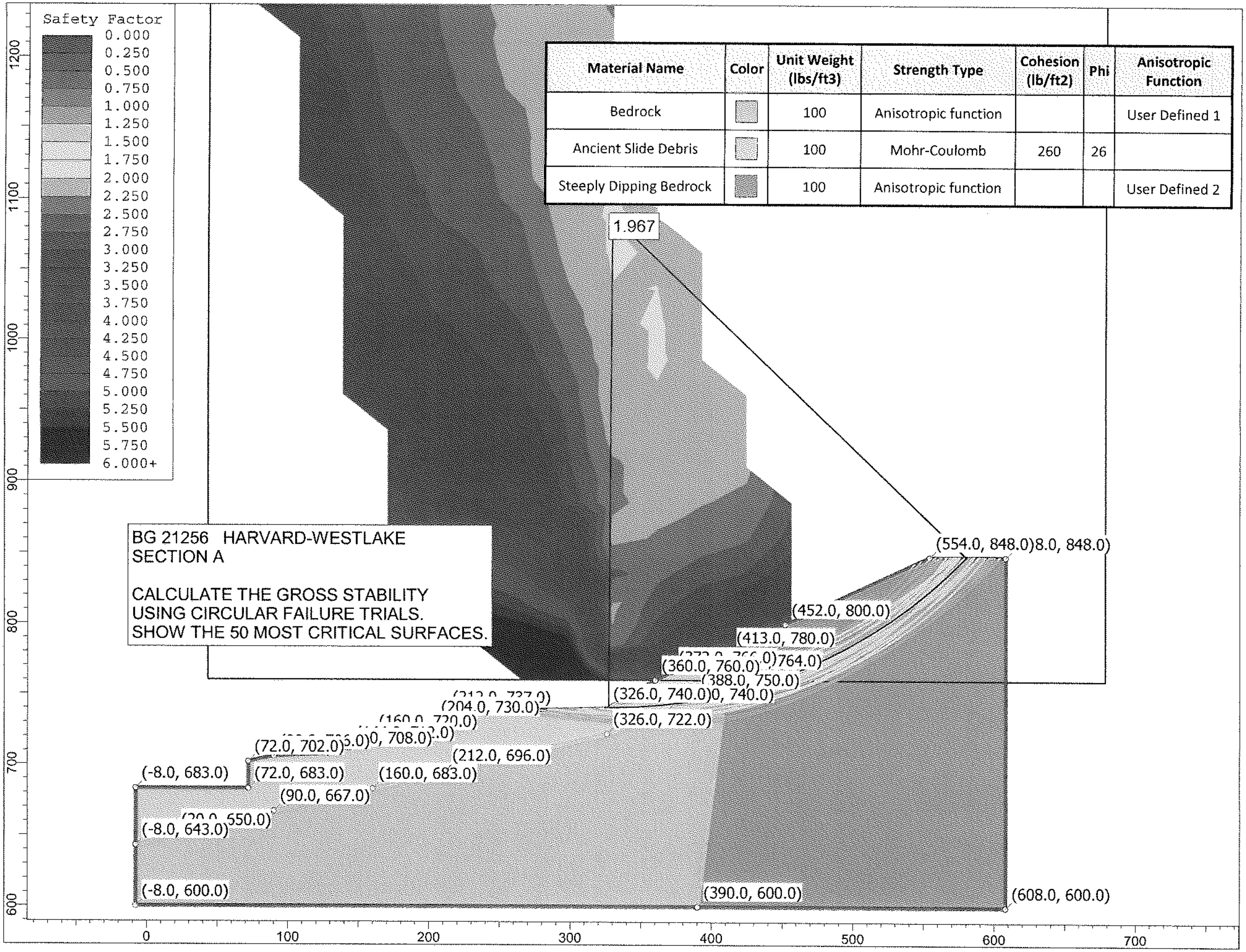
The above action is in accordance with the recommendation of the Grading Consultants at their meeting of January 24, 1967.

S. A. MAIMARK, Secretary
BOARD OF BUILDING AND
SAFETY COMMISSIONERS

lch:w
Ext. 1226

cc: LeRoy Crandall & Assoc.
Hood & Schmidt
Grading

*This letter supersedes letter dated January 30, 1967, to add a sentence to provision 1(b) which was inadvertently omitted.



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (lb/ft2)	Phi	Anisotropic Function
Bedrock		100	Anisotropic function			User Defined 1
Ancient Slide Debris		100	Mohr-Coulomb	260	26	
Steeply Dipping Bedrock		100	Anisotropic function			User Defined 2

Slide Analysis Information

SLIDE - An Interactive Slope Stability Program

Project Summary

- File Name: 21256 Section A ver4.slim
- Slide Modeler Version: 6.012
- Project Title: SLIDE - An Interactive Slope Stability Program
- Date Created: 12/10/2010, 11:08:30 AM

General Settings

- Units of Measurement: Imperial Units
- Time Units: days
- Permeability Units: feet/second
- Failure Direction: Right to Left
- Data Output: Standard
- Maximum Material Properties: 20
- Maximum Support Properties: 20

Analysis Options

Analysis Methods Used

  - Spencer

- Number of slices: 50
- Tolerance: 0.005
- Maximum number of iterations: 50
- Check malpha < 0.2: Yes
- Initial trial value of FS: 1
- Steffensen Iteration: Yes

Groundwater Analysis

- Groundwater Method: Water Surfaces
- Pore Fluid Unit Weight: 62.4 lbs/ft3
- Advanced Groundwater Method: None

Random Numbers

- Pseudo-random Seed: 10116
- Random Number Generation Method: Park and Miller v.3

Surface Options

- Surface Type: Circular
- Search Method: Grid Search
- Radius Increment: 10
- Composite Surfaces: Disabled
- Reverse Curvature: Create Tension Crack
- Minimum Elevation: Not Defined
- Minimum Depth: Not Defined

Material Properties

Property	Bedrock	Ancient Slide Debris	Steeply Dipping Bedrock
Color			
Strength Type	Anisotropic function	Mohr-Coulomb	Anisotropic function
Unit Weight [lbs/ft3]	100	100	100
Cohesion [psf]		260	
Friction Angle [deg]		26	
Water Surface	None	None	None
Ru Value	0	0	0

Anisotropic Functions

- Name: User Defined 1

Angle From	Angle To	c	phi
16	-90	590	30
20	16	260	26
90	20	590	30

- Name: User Defined 2

Angle From	Angle To	c	phi
-60	-90	260	26
80	-60	590	30
90	80	260	26

Global Minimums

Method: spencer

- FS: 1.966680
- Center: 329.104, 1087.711
- Radius: 347.156
- Left Slip Surface Endpoint: 326.955, 740.562
- Right Slip Surface Endpoint: 580.214, 848.000
- Resisting Moment=1.78169e+008 lb-ft
- Driving Moment=9.05938e+007 lb-ft
- Resisting Horizontal Force=462510 lb
- Driving Horizontal Force=235174 lb

Valid / Invalid Surfaces

Method: spencer

- Number of Valid Surfaces: 1798
- Number of Invalid Surfaces: 3053

Error Codes:

- Error Code -101 reported for 32 surfaces
- Error Code -103 reported for 219 surfaces
- Error Code -108 reported for 7 surfaces
- Error Code -111 reported for 15 surfaces
- Error Code -112 reported for 38 surfaces
- Error Code -113 reported for 3 surfaces
- Error Code -1000 reported for 2739 surfaces

Error Codes

The following errors were encountered during the computation:

- -101 = Only one (or zero) surface / slope intersections.
- -103 = Two surface / slope intersections, but one or more surface / nonslope external polygon intersections lie between them. This usually occurs when the slip surface extends past the bottom of the soil region, but may also occur on a benched slope model with two sets of Slope Limits.
- -108 = Total driving moment or total driving force < 0.1. This is to limit the calculation of extremely high safety factors if the driving force is very small (0.1 is an arbitrary number).
- -111 = safety factor equation did not converge
- -112 = The coefficient M-Alpha = cos(alpha)(1+tan(alpha)tan(phi)/F) < 0.2 for the final iteration of the safety factor calculation. This screens out some slip surfaces which may not be valid in the context of the analysis, in particular, deep seated slip surfaces with many high negative base angle slices in the passive zone.
- -113 = Surface intersects outside slope limits.
- -1000 = No valid slip surfaces are generated at a grid center. Unable to draw a surface.

Slice Data

- **Global Minimum Query (spencer) - Safety Factor: 1.96668**

Slice Number	Width [ft]	Weight [lbs]	Base Material	Base Cohesion [psf]	Base Friction Angle [degrees]	Shear Stress [psf]	Shear Strength [psf]	Base Normal Stress [psf]	Pore Pressure [psf]	Effective Normal Stress [psf]
1	5.00115	734.367	Ancient Slide Debris	260	26	187.374	368.504	222.467	0	222.467
2	5.00115	2185.08	Ancient Slide Debris	260	26	265.399	521.955	537.087	0	537.087
3	5.00115	3599.75	Ancient Slide Debris	260	26	339.891	668.456	837.457	0	837.457
4	5.00115	4978.33	Ancient Slide Debris	260	26	410.945	808.197	1123.97	0	1123.97
5	5.00115	6320.77	Ancient Slide Debris	260	26	478.654	941.359	1396.99	0	1396.99
6	5.00115	7626.98	Ancient Slide Debris	260	26	543.103	1068.11	1656.87	0	1656.87
7	5.00115	8879.88	Ancient Slide Debris	260	26	603.492	1186.88	1900.38	0	1900.38
8	5.00115	9933.35	Ancient Slide Debris	260	26	652.429	1283.12	2097.71	0	2097.71
9	5.00115	10909.5	Ancient Slide Debris	260	26	696.448	1369.69	2275.2	0	2275.2
10	5.00115	11653.4	Ancient Slide Debris	260	26	727.876	1431.5	2401.93	0	2401.93
11	5.36485	13061.7	Bedrock	590	30	1050.01	2065.03	2554.83	0	2554.83
12	5.36485	13597.9	Bedrock	590	30	1069.66	2103.67	2621.76	0	2621.76
13	5.36485	14087.8	Bedrock	590	30	1086.26	2136.33	2678.32	0	2678.32
14	5.36485	14531	Bedrock	590	30	1099.89	2163.14	2724.75	0	2724.75
15	5.36485	14927.2	Bedrock	590	30	1110.61	2184.21	2761.27	0	2761.27
16	5.36485	15275.9	Bedrock	590	30	1118.47	2199.68	2788.05	0	2788.05
17	5.03113	14612.2	Steeply Dipping Bedrock	590	30	1124.14	2210.82	2807.34	0	2807.34

18	5.03113	15143.7	Steeply Dipping Bedrock	590	30	1143.33	2248.56	2872.7	0	2872.7
19	5.03113	15761.3	Steeply Dipping Bedrock	590	30	1166.67	2294.47	2952.22	0	2952.22
20	5.03113	16337.9	Steeply Dipping Bedrock	590	30	1187.19	2334.82	3022.12	0	3022.12
21	5.03113	16873	Steeply Dipping Bedrock	590	30	1204.92	2369.7	3082.53	0	3082.53
22	5.03113	17366.1	Steeply Dipping Bedrock	590	30	1219.92	2399.19	3133.6	0	3133.6
23	5.03113	17816.4	Steeply Dipping Bedrock	590	30	1232.2	2423.35	3175.46	0	3175.46
24	5.03113	18223.3	Steeply Dipping Bedrock	590	30	1241.82	2442.26	3208.21	0	3208.21
25	5.03113	18573.6	Steeply Dipping Bedrock	590	30	1248.16	2454.73	3229.81	0	3229.81
26	5.03113	18799	Steeply Dipping Bedrock	590	30	1247.87	2454.16	3228.82	0	3228.82
27	5.03113	18964.4	Steeply Dipping Bedrock	590	30	1244.39	2447.32	3216.96	0	3216.96
28	5.03113	19083.3	Steeply Dipping Bedrock	590	30	1238.47	2435.68	3196.81	0	3196.81
29	5.03113	19154.8	Steeply Dipping Bedrock	590	30	1230.14	2419.29	3168.43	0	3168.43
30	5.03113	19177.7	Steeply Dipping Bedrock	590	30	1219.42	2398.2	3131.89	0	3131.89
31	5.03113	19151.1	Steeply Dipping Bedrock	590	30	1206.31	2372.43	3087.25	0	3087.25

32	5.03113	19073.7	Steeply Dipping Bedrock	590	30	1190.85	2342.02	3034.59	0	3034.59
33	5.03113	18944.4	Steeply Dipping Bedrock	590	30	1173.05	2307.01	2973.96	0	2973.96
34	5.03113	18761.7	Steeply Dipping Bedrock	590	30	1152.92	2267.42	2905.38	0	2905.38
35	5.03113	18524.2	Steeply Dipping Bedrock	590	30	1130.47	2223.27	2828.91	0	2828.91
36	5.03113	18230.4	Steeply Dipping Bedrock	590	30	1105.72	2174.59	2744.59	0	2744.59
37	5.03113	17878.5	Steeply Dipping Bedrock	590	30	1078.67	2121.39	2652.44	0	2652.44
38	5.03113	17466.7	Steeply Dipping Bedrock	590	30	1049.32	2063.68	2552.48	0	2552.48
39	5.03113	16993.1	Steeply Dipping Bedrock	590	30	1017.69	2001.47	2444.74	0	2444.74
40	5.03113	16455.4	Steeply Dipping Bedrock	590	30	983.78	1934.78	2329.22	0	2329.22
41	5.03113	15851.4	Steeply Dipping Bedrock	590	30	947.587	1863.6	2205.94	0	2205.94
42	5.03113	15178.4	Steeply Dipping Bedrock	590	30	909.111	1787.93	2074.87	0	2074.87
43	5.03113	14433.7	Steeply Dipping Bedrock	590	30	868.352	1707.77	1936.05	0	1936.05
44	5.03113	13614.2	Steeply Dipping Bedrock	590	30	825.31	1623.12	1789.42	0	1789.42
45	5.03113	12690.2	Steeply Dipping Bedrock	590	30	778.925	1531.9	1631.42	0	1631.42

46	5.03113	10890.8	Steeply Dipping Bedrock	590	30	699.095	1374.9	1359.48	0	1359.48
47	5.03113	8633.99	Steeply Dipping Bedrock	590	30	603.558	1187.01	1034.05	0	1034.05
48	5.03113	6286.53	Steeply Dipping Bedrock	590	30	507.145	997.391	705.621	0	705.621
49	5.03113	3843.3	Steeply Dipping Bedrock	590	30	409.876	806.094	374.286	0	374.286
50	5.03113	1298.53	Steeply Dipping Bedrock	590	30	312.465	614.518	42.4664	0	42.4664

Interslice Data

- **Global Minimum Query (spencer) - Safety Factor: 1.96668**

Slice Number	X coordinate [ft]	Y coordinate - Bottom [ft]	Interslice Normal Force [lbs]	Interslice Shear Force [lbs]	Interslice Force Angle [degrees]
1	326.955	740.562	0	0	0
2	331.956	740.567	934.781	381.613	22.2071
3	336.957	740.644	2218.99	905.877	22.2072
4	341.958	740.793	3791.73	1547.93	22.2072
5	346.959	741.014	5595.48	2284.29	22.2072
6	351.96	741.308	7575.91	3092.77	22.2071
7	356.962	741.674	9681.76	3952.46	22.2071
8	361.963	742.113	11861.8	4842.43	22.2071
9	366.964	742.626	14046.4	5734.29	22.2072
10	371.965	743.211	16193.2	6610.66	22.2071
11	376.966	743.87	18245.6	7448.53	22.2071
12	382.331	744.66	21854.5	8921.83	22.2071
13	387.696	745.535	25290.6	10324.6	22.2072
14	393.061	746.497	28534.3	11648.8	22.2072
15	398.426	747.546	31568.3	12887.4	22.2072
16	403.79	748.684	34378.2	14034.5	22.2072
17	409.155	749.91	36951.5	15085	22.2071
18	414.186	751.142	39141.5	15979	22.2071
19	419.217	752.454	41117.5	16785.7	22.2071
20	424.249	753.847	42867.5	17500.1	22.2071
21	429.28	755.322	44375.5	18115.7	22.2071
22	434.311	756.88	45627.4	18626.8	22.2071
23	439.342	758.523	46611	19028.4	22.2072
24	444.373	760.25	47316	19316.2	22.2072
25	449.404	762.065	47734.1	19486.9	22.2072
26	454.435	763.968	47859.4	19538	22.2071
27	459.467	765.961	47694.9	19470.9	22.2072
28	464.498	768.046	47241.9	19285.9	22.2071
29	469.529	770.224	46502.5	18984.1	22.2072
30	474.56	772.497	45481	18567.1	22.2072
31	479.591	774.868	44183.6	18037.4	22.2071
32	484.622	777.338	42618.6	17398.5	22.2071

33	489.653	779.91	40796.3	16654.6	22.2071
34	494.684	782.587	38729.4	15810.8	22.2071
35	499.716	785.372	36432.8	14873.2	22.2071
36	504.747	788.266	33924	13849.1	22.2072
37	509.778	791.275	31223.1	12746.4	22.2071
38	514.809	794.4	28352.8	11574.7	22.2072
39	519.84	797.647	25338.8	10344.2	22.207
40	524.871	801.018	22210	9066.95	22.2071
41	529.902	804.519	18998.6	7755.96	22.2072
42	534.934	808.155	15740.7	6425.94	22.2071
43	539.965	811.93	12476.1	5093.2	22.2071
44	544.996	815.85	9249.06	3775.82	22.2072
45	550.027	819.923	6108.85	2493.86	22.2071
46	555.058	824.154	3119.76	1273.6	22.2071
47	560.089	828.552	653.543	266.801	22.2071
48	565.12	833.126	-1042.76	-425.695	22.2072
49	570.152	837.884	-1852.05	-756.075	22.2071
50	575.183	842.838	-1646.75	-672.264	22.2071
51	580.214	848	0	0	0

List Of Coordinates

External Boundary

X	Y
-8	600
390	600
608	600
608	848
554	848
452	800
413	780
372	766
360	760
326	740
212	737

204	730
160	720
144	712
128	708
90	706
72	702
72	683
-8	683
-8	643

Material Boundary

X	Y
-8	643
20	650
90	667
160	683
212	696
326	722
370	740
388	750
404	764
413	780

Material Boundary

X	Y
390	600
413	780

Appendix I

Leroy Crandall and Associates, report dated August 26, 1966

CONSULTING FOUNDATION ENGINEERS

LeROY CRANDALL
AND ASSOCIATES

August 26, 1966

Johnson & Silvestri & Associates
13135 Ventura Boulevard
North Hollywood, California 91604

(Our Job No. A-66131)

Gentlemen:

Our "Report of Foundation Investigation, Proposed School Additions, 3700 Coldwater Canyon Drive, Los Angeles, California, for the Harvard School" is herewith submitted.

The scope of the investigation was planned in collaboration with your firm and with King·Benioff·Steinmann·King, Consulting Engineers. We were advised of the structural features of the proposed buildings, including planned grades and foundation loads, by your office and by King·Benioff·Steinmann·King. The results of our investigation and foundation recommendations were discussed with the interested parties as the data became available.

The proposed building additions, as well as several of the existing school buildings, are located on or adjacent to an ancient landslide. Based on our investigation and on a supplementary geological investigation, the ancient landslide mass is presently stable. The proposed grading will result in increased stability and if proper subsurface drainage of the mass is provided, the site will be suitable for construction as planned. The proposed buildings may be supported on conventional spread footings or drilled cast-in-place piles. Recommendations for foundation design, for grading, and for floor support are presented in the report. The results of a geologic investigation performed by Hood & Schmidt, Inc., are also presented.

Respectfully submitted,

LeROY CRANDALL AND ASSOCIATES

by LeRoy Crandall
LeRoy Crandall

LC-PM:lb
(6 copies submitted)

cc: (2) King·Benioff·Steinmann·King

0420110 1153

REPORT OF FOUNDATION INVESTIGATION

PROPOSED SCHOOL ADDITIONS

3700 COLDWATER CANYON DRIVE

LOS ANGELES, CALIFORNIA

FOR THE

HARVARD SCHOOL

TABLE OF CONTENTS

Text

Plates

SCOPE

This report presents the results of a foundation investigation of the site of the subject additions to the Harvard School. The locations of the existing buildings, the proposed additions, and our exploration borings and pits are shown on Plate 1, Plot Plan.

The investigation was authorized to determine the characteristics of the soils at the site and to provide recommendations for foundation design. To provide supplementary data, a geologic investigation of the site was performed by Hood & Schmidt, Inc., Consulting Engineering Geologists.

The results of the field explorations and laboratory tests, which together with the geologic investigation form the basis of our recommendations, are presented in the attached Appendix A. The results of our stability analyses are presented in Appendix B. The results of the geologic investigation are presented in the attached Appendix C.

STRUCTURAL CONSIDERATIONS

The proposed building additions, which are included in our investigation, are identified and shown in plan on Plate 1. The Lower School will be three stories high and of Type I reinforced concrete construction with concrete or masonry walls. Column loads will range from 100 to 200 kips; wall loads will range from 2 to 6 kips per lineal foot. The proposed Infirmary will be of Type V wood frame construction and part one and part two stories high. This building will be supported on continuous wall footings imposing loads up to 4 kips per lineal foot.

The Faculty Residence buildings, which are planned for future construction, will be two stories high and of Type V wood frame construction.

Column loads will range from 90 to 100 kips; wall loads will be 4 kips per lineal foot. The structural features of the Academic Center, which is also planned for future construction, have not been established at the present time; however, it is anticipated that this building will be similar to the Lower School.

The planned floor grades for the various buildings are indicated on Plate 1. As indicated by the planned floor grades and existing topography, excavation will be required beneath the majority of the Lower School, the Academic Center, and the Infirmary. Very likely, excavation will also be required beneath the Faculty Residence buildings. Where floor levels are above the present grade, it may be desirable to use structural floors in lieu of placing compacted fill.

SITE CONDITIONS

The proposed school additions will be located on the existing Harvard School property. The site is currently occupied by numerous buildings and appurtenant paved and planted areas. Construction of the future Academic Center will require the removal of some existing buildings. Many of the existing buildings were constructed some 30 or 40 years ago, and were formerly part of a country club. Some of the existing buildings, however, including Rugby Hall, were constructed within the last few years. The property slopes generally downward west to northwest; contours describing the existing topography are shown on Plate 1.

Existing fill soils, two to six feet in thickness, were encountered in several of the borings. The fill consists primarily of clay and is not uniformly well compacted. The fill is immediately underlain by natural overburden soils consisting of silty clay and clayey silt. The soils are moderately firm to firm at present moisture content, and [illegible] weaker and more compressible when wet. The soils are somewhat expansive and would swell and shrink with changes in the moisture content. The overburden soils in turn are underlain by shale of the Modelo formation. The shale contains lenses of sandstone and some layers of highly cemented limestone. (Some of these underlying materials could be classified as siltstone; the materials are, however, referred to as shale throughout this report.) Water seepage was encountered in Boring 11 at a depth of 26 feet, and in Boring 12 at depths of 35 and 39 feet. Water was not observed in the borings at completion of the drilling.

As described in more detail in the geological report, a major synclinal fold strikes northeast across the property. Bedding north of the fold axis generally dips northwest at 10 to 22 degrees from the horizontal, while bedding south of this axis is generally found to dip northwest at 60 to 81 degrees from the horizontal. However, south of the synclinal axis, in the area of the future proposed Facilty Residences, both northwest and southeast dipping strata were observed.

Borings 1 through 4 and 7 through 12 were drilled within an ancient landslide mass. The general limits of this ancient slide mass are indicated on Plate 1. As mentioned previously, the overburden soils, including those within the slide mass, are moderately firm. The shale deposits within the slide mass, although somewhat disturbed and weathered at some locations, were also firm. The shale deposits below the slide plane are firm to very firm. A subsurface section through the Lower School and future Academic Center is shown on Plate 2; the location of the section is indicated on Plate 1.

RECOMMENDATIONS

GENERAL

As discussed in previous sections, the Lower School and the Academic Center will be located on an ancient landslide. Existing school buildings on the property are presently located on this ancient landslide; based on our observations, no movement of the mass has occurred since construction of the buildings. Although some structural damage was observed adjacent to one building, exploration pits revealed that the observed damage is due to settlement of poorly compacted backfill.

To confirm the stability of the existing landslide mass, graphical analyses were made. These analyses, which are described in Appendix B, indicate that the mass is presently stable against movement with a factor of safety of 1.6. Since the planned grading will consist essentially of excavation, the proposed construction will not decrease the stability of the mass.

Although presently stable, we recommend that provisions be taken to provide positive drainage of the slide mass in order to minimize the possibility of saturation of the mass and possible development of hydrostatic pressures. Proper surface drainage should be provided throughout the slide area and the surrounding areas. Various schemes of providing drainage of the interior of the mass are possible. Such drainage could be provided by conventional horizontal drains drilled from the lower northwest portion of the property parallel to the axis of the landslide. Alternately, a series of seepage pits could be installed and connected by horizontal drains drilled perpendicular to the axis of the slide. We would be pleased to discuss the alternate means of drainage as the plans for the development are established.

If grading of the proposed building area consists entirely of excavation and drainage of the slide mass is provided, the site will be suitable for construction as planned. The proposed buildings, including those located on the slide mass, may be supported on spread-type foundations or on drilled cast-in-place concrete piles.

Recommendations for foundation design, for excavating and walls below grade, and for floor slab support are presented in the following sections. Because of the geological factors of the site, any grading should be carefully considered, and grading plans reviewed and approved by both the foundation engineer and the engineering geologist. Also, the plans for drainage of the landslide mass should be similarly reviewed and approved.

FOUNDATIONS

SPREAD FOUNDATIONS

Excavation for the Lower School, for the Academic Center, and for the Faculty Residences will extend to shale at most all locations, permitting the use of conventional spread footings. Where the shale is not close to grade, drilled-and-belled caissons may be used. Conventional spread footings and drilled-and-belled caissons carried at least one foot into the shale and at least three feet below the lowest adjacent grade or floor level may be designed to impose a dead plus live load pressure of 4,000 pounds per square foot. A one-third increase in the bearing value may be used when considering wind or seismic loads.

Excavation for the Infirmary will not expose the shale, except possibly along the easterly wall. This building, however, will be relatively light and may be satisfactorily supported on shallow spread footings established in the upper natural soils. Conventional spread footings for this building, carried at least one foot into undisturbed natural soils and at least two feet below the adjacent final grade or floor level, may be designed to impose a dead plus live load pressure of 1,500 pounds per square foot. A one-third increase in the bearing value may be used for wind or seismic loads.

The maximum ultimate settlement of the Lower School, supported on spread foundations as recommended, will be less than one-half inch. Settlement of the Academic Center will depend on the loads imposed but should be within acceptable limits. The maximum ultimate settlement of the Infirmary and Residences will be on the order of one-fourth inch.

All footing excavations and caisson excavations must be carefully inspected and approved by our firm to assure satisfactory support. To allow personnel to enter caisson excavations, the use of at least 24-inch-diameter caisson shafts is recommended. The installation of the caissons will require the use of temporary casing for the safety of personnel. Necessary footing backfill and all utility trench backfill should be mechanically compacted to 90% and not flooded. The natural overburden soils are somewhat expansive and the soils should not be allowed to dry out and crack before pouring foundations for the Infirmary. Footings should be deepened as necessary to extend below any dry cracked soils.

The passive resistance of the soils and soil friction may be used for resisting lateral loads. A coefficient of friction of 0.4 may be used between footings or the floor slabs and the supporting materials. The passive resistance of the overburden soils against spread footings may be assumed to be equal to the pressure developed by a fluid with a density of 300 pounds per cubic foot. The lateral resistance of caisson shafts may be determined using the pole formula in the City of Los Angeles Building Code. When using the pole formula, an allowable lateral bearing value of 600 pounds per square foot per foot of depth, up to a maximum of 8,000 pounds per square foot, may be used.

DRILLED PILING

As an alternate to spread foundations, drilled cast-in-place concrete piling may be used. The downward and upward capacities of 16-, 20-, and 24-inch-diameter drilled piles are presented on Plate 3, Drilled Pile Capacities. The capacities are presented as a function of penetration into the underlying shale; the capacity of the overlying overburden soils has been neglected. Dead plus live load capacities are shown; a one-third increase may be used when considering wind or seismic loads. The pile capacities shown are based on the strength of the supporting shale; the allowable compressible strength of the concrete may require the use of lower values. Piles in groups should be spaced at least 2½ diameters on centers; if so spaced, there will be no reduction in the downward capacities of the piles due to group action. The pile excavations should be inspected by personnel of our firm prior to pouring. The maximum ultimate settlement of the Lower School, supported on drilled piling, will be less than one-fourth inch.

The exploration borings were drilled to depths up to 48 feet below the existing ground surface with conventional bucket-type drilling equipment. Water seepage was encountered in two of the borings; however, difficulties due to water are not anticipated if the piles are poured immediately after drilling and inspection. Closely-spaced piles should be drilled and filled alternately with the concrete permitted to set at least eight hours before drilling an adjacent hole. The concrete should be placed with special equipment so that the concrete is not allowed to fall freely more than ten feet and to prevent concrete from striking the walls of the excavations.

A chopping bucket was used in two of the borings to penetrate highly cemented layers. Similar hard layers should be anticipated in the drilled pile excavations, with resulting difficult drilling.

The passive resistance of the soils and soil friction may be used for resisting lateral loads. The lateral resistance of drilled piles may be computed by using the pole formula in the City of Los Angeles Building Code. When using the pole formula, an allowable lateral bearing value of 600 pounds per square foot per foot of depth, up to a maximum of 8,000 pounds per square foot may be used. The effective depth of the drilled piles may be taken as the depth of reinforcing, but not more than 20 feet.

EXCAVATION AND WALLS BELOW GRADE

The overburden soils may be excavated with conventional earth-moving equipment. Although the underlying shale is firm, we believe this material may be excavated with heavy earth-moving equipment including heavy rippers. Temporary cut slopes may be made at 1:1.

For design of walls below grade, it may be assumed that the soils will exert a lateral pressure equal to that developed by a fluid with a density of 30 pounds per cubic foot. This value is for a condition of level backfill; if the backfill is sloped upward at 2:1 (horizontal to vertical), the fluid pressure will be increased to 40 pounds per cubic foot. All required backfill behind walls below grade should be mechanically compacted in layers; flooding should not be permitted. Backfill should be compacted to at least 90% of the maximum density obtainable by the ASTM Designation D1557-64T method of compaction. If the backfill is compacted as recommended, infiltration of water into the backfill will be small and hydrostatic pressures are not expected to develop against the walls. Walls below grade, however, should at least be damp-proofed.

FLOOR SLAB SUPPORT

In areas of the buildings currently below the planned floor grade, we suggest the use of structurally supported floors in lieu of placing compacted fill for support of slabs on grade. In excavated areas, the natural soils will offer adequate support to slabs on grade. After excavating, the exposed subgrade should be carefully inspected and any existing fill deposits or disturbed natural soils should be excavated and replaced as properly compacted fill. All such fill should be placed in loose lifts not more than eight inches in thickness, brought to optimum moisture content, and compacted to at least 90% of the maximum density obtainable by the ASTM Designation D1557-64T method of compaction.

The materials are fine-grained, and we recommend that the floor slabs be supported on a four-inch-thick layer of gravel or on an impermeable membrane as a capillary break. The soils are somewhat expansive, and the subgrade should be uniformly moistened before placing the gravel or membrane. The subgrade should be approved by the Foundation Engineer before pouring floor slabs and adjacent walks. A suggested gradation for the gravel would be as follows:

Sieve Size	Percent Passing
3/4"	90 - 100
No. 4	0 - 10
No. 100	0 - 3

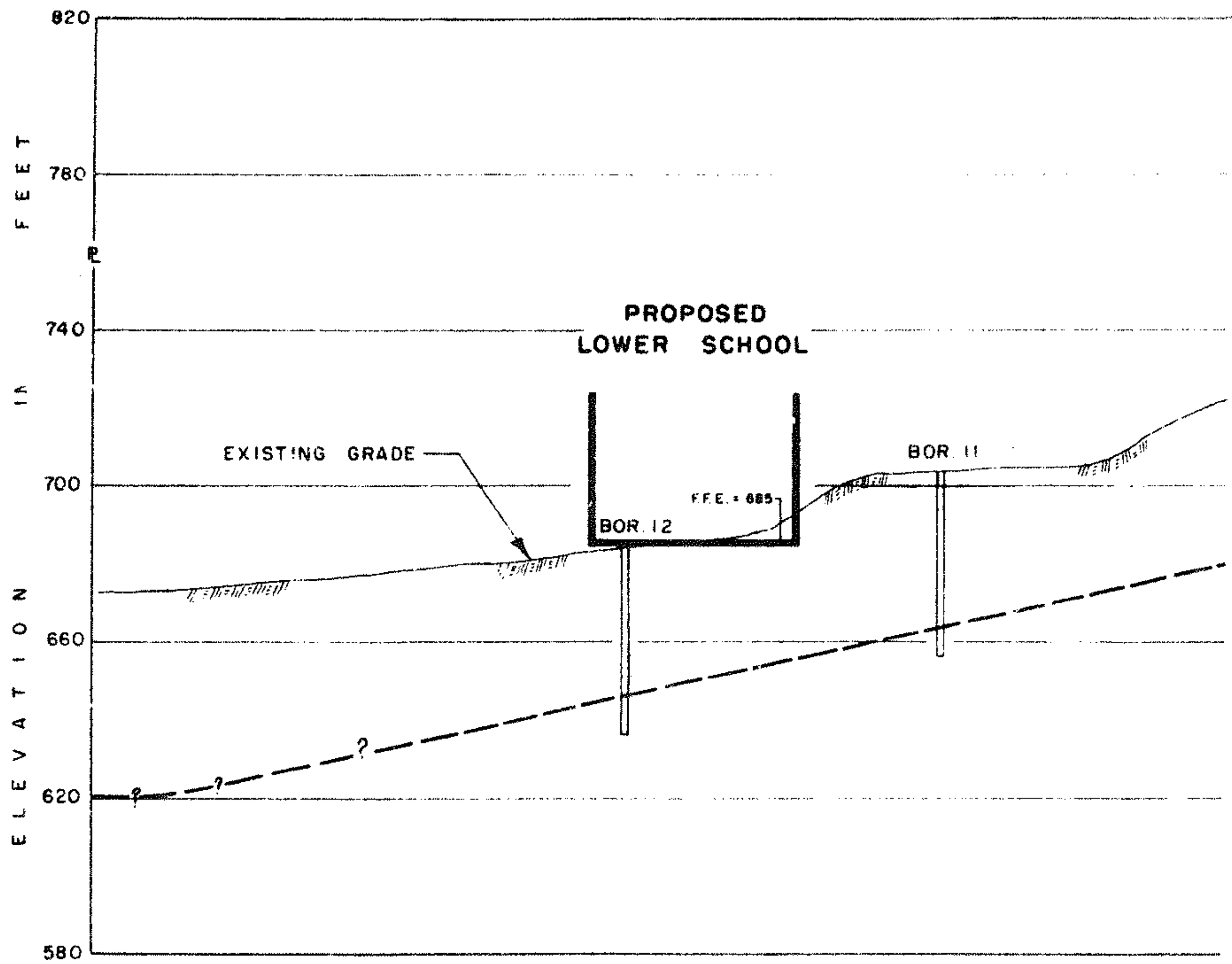
If a membrane is used instead of gravel, it should be covered with a thin layer of sand to allow curing of the concrete, or a low-slump concrete should be used to minimize possible slab curling.

-o0o-

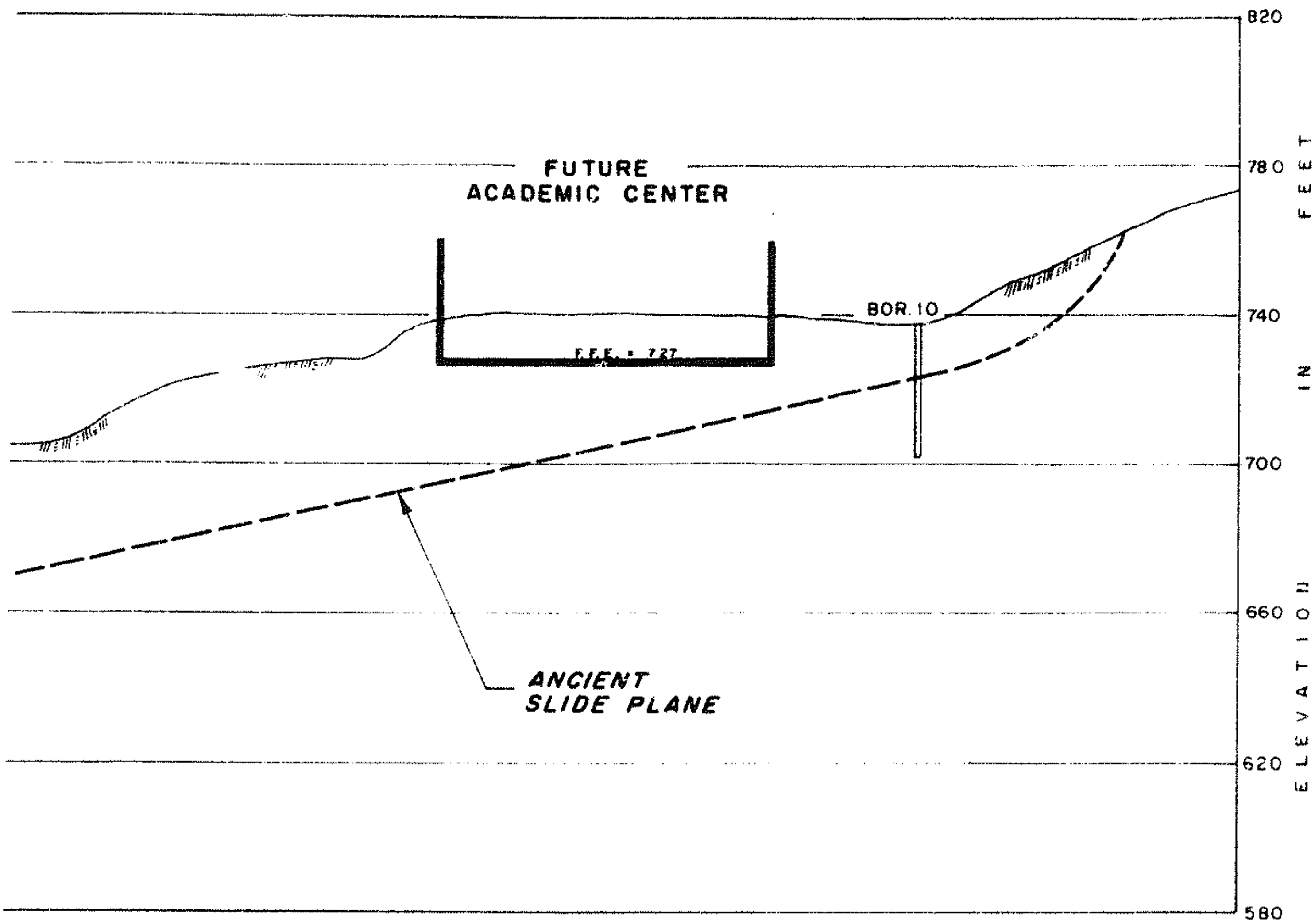
BIG MAP(152-44)

B & S

GRADING FILES


820
780
740
700
660
620
580
ELEVATION IN FEET
℄
PROPOSED
LOWER SCHOOL
EXISTING GRADE
F.F.E. = 685
BOR. 12
BOR. 11
?
?
?

SU

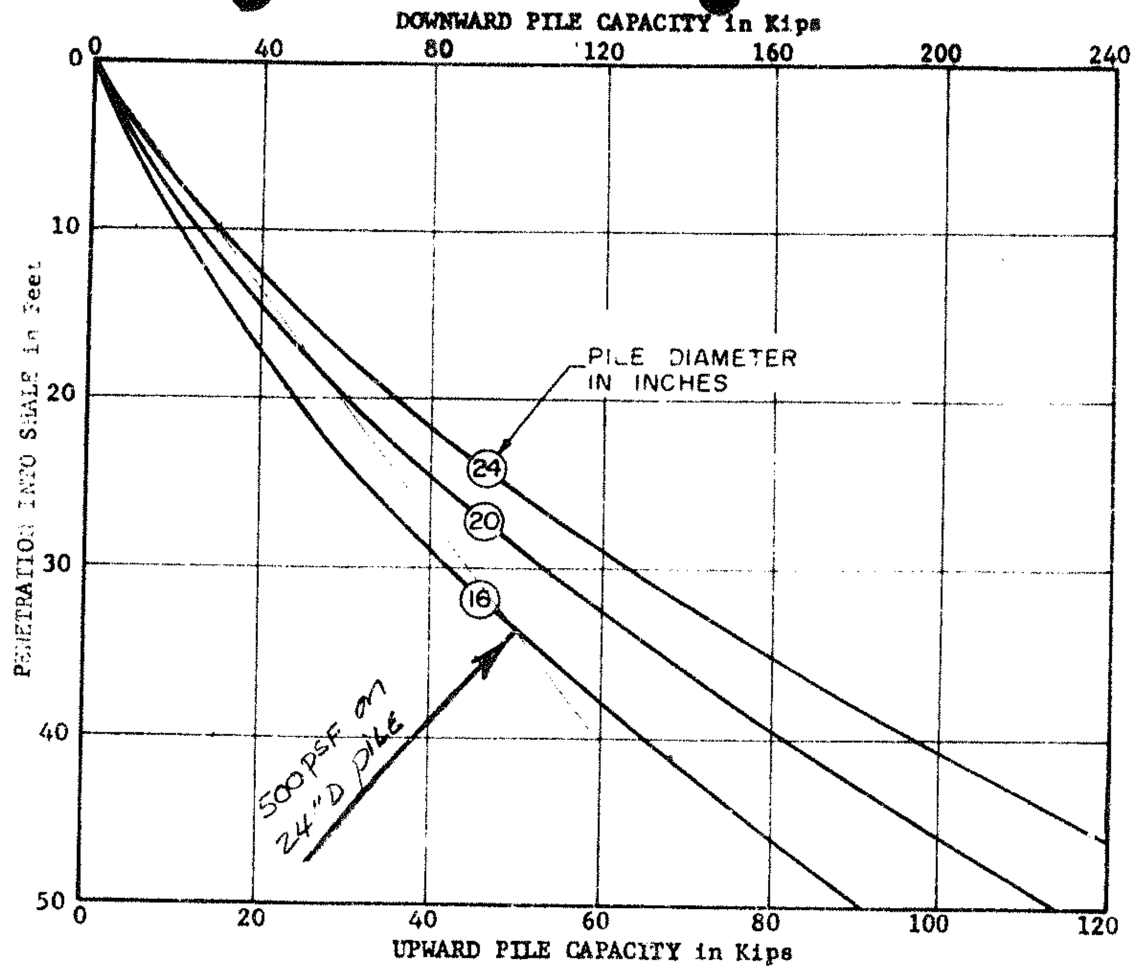


SUBSURFACE SECTION

SCALE 1" = 40'

LEROY CRANDALL & A

PLATE 2



NOTES:

(1) The indicated values refer to the total of dead plus live loads; a one-third increase may be used when considering wind or seismic loads.

(2) Piles in groups should be spaced a minimum of 2½ diameters on centers, and should be drilled and filled alternately with the concrete permitted to set at least 8 hours before drilling an adjacent hole.

(3) The indicated values are based on the strength of the soils; the actual pile capacities may be limited to lower values by the strength of the piles.

DRILLED PILE CAPACITIES

LEROY CRANDALL & ASSOCIATES

PLATE 3

APPENDIX A

APPENDIX A

EXPLORATIONS

The site of the proposed school additions was initially explored by drilling nine borings to depths ranging from 15 to 45 feet below the existing ground surface. In areas inaccessible to drilling equipment, seven pits were excavated to the shale using hand equipment. Additional explorations were subsequently considered necessary to complete our analyses of the stability of the site, and three additional borings, ranging in depth from 36 to 48 feet, were drilled. Also, one pit was excavated adjacent to a footing of an existing building where some settlement had been observed. The geological consultants, Hood & Schmidt, Inc., were involved in both the initial and supplementary field work, and their report is presented in Appendix C.

The borings were drilled using 18-inch-diameter bucket-type drilling equipment. Caving of the boring walls did not occur during drilling, and casing or drilling mud was not used to advance the borings. A gad and chopping bucket were used in Borings 3 and 10 to penetrate hard layers.

The borings were drilled under the supervision of our field engineer, who logged the soils encountered and obtained undisturbed samples for laboratory inspection and testing. Loose samples were also obtained for laboratory compaction tests. The boring and pit logs are presented on Plates A-1 through A-15; the depths at which undisturbed samples were obtained are indicated to the left of the boring logs. The overburden soils are classified in accordance with the Unified Soil Classification System described on Plate B.

In addition to logging the soils and obtaining undisturbed samples, several of the borings were entered to obtain measurements of the dip and direction of the dip of the shale; these data are presented to the left of the respective boring logs. Also indicated on the logs is the approximate depth to the slide plane.

LABORATORY TESTS

The field moisture content and dry density of the soils encountered were determined by performing tests on the undisturbed samples. The results of the tests are shown to the left of the boring logs.

Direct shear tests were performed on numerous undisturbed samples to determine the strength of the materials. To simulate the adverse conditions that may occur with moisture infiltration, most of the samples were tested after being submerged in water for a period of two days. The samples were sheared along the direction of the bedding planes which would have the least resistance. Also, to simulate an existing slide surface, several of the samples were purposely broken along the shear plane and submerged in water for a period of two days prior to testing. Most of the samples were tested at two different surcharge pressures to provide more complete data. The results of the direct shear tests are presented on Plates C-1 and C-2, Direct Shear Test Data.

Triaxial shear tests were performed on three undisturbed samples to provide additional data on the strength of the materials. The samples were tested after soaking for a period of two days and were tested at different confining pressures. The results of the triaxial shear tests are presented on Plate C-3, Triaxial Shear Test Data.

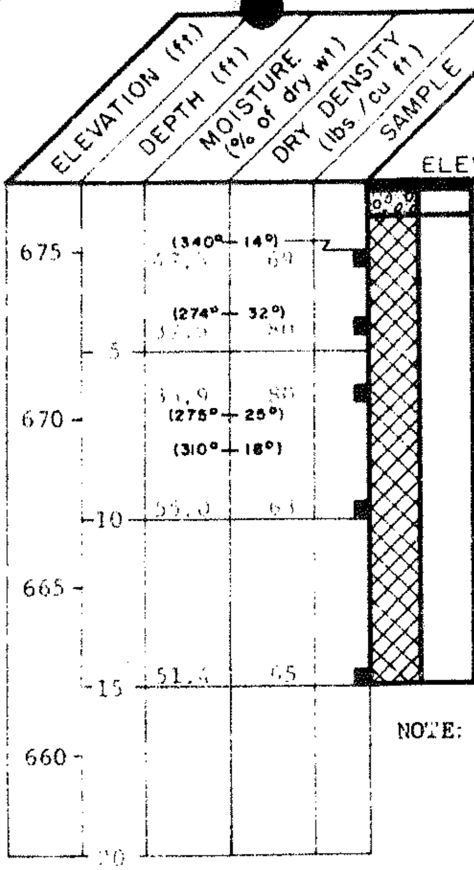
Confined consolidation tests were performed on five undisturbed samples to determine the compressibility of the soils. To illustrate the effect of moisture, water was added to three of the samples during the tests. The consolidation test results are presented on Plates D-1 through D-3, Consolidation Test Data.

-oOo-

BORING 1

DATE DRILLED: May 10, 1966
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION 677*



3" ASPHALTIC PAVING and 9" BASE COURSE

SHALE - weathered, fractured, mottled light brown and light greyish-brown

Lenses of Sandstone
Diatomaceous

NOTE: Water not encountered. No caving.

*Elevations refer to datum of reference topographic survey, see Plate 1.

Soils classified in accordance with the Unified Soil Classification System.

(340° — 14°)
- INDICATES ANGLE OF DIP BELOW HORIZONTAL
- INDICATES DIRECTION OF DIP (AZIMUTH)

LOG OF BORING

LEROY CRANDALL & ASSOCIATES

BORING 2

DATE DRILLED: May 10, 1966
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION 677

ELEVATION (ft)	DEPTH (ft)	MOISTURE (% of dry wt)	DRY DENSITY (lbs./cu ft)	SAMPLE
		24.1	[illegible]	
675		[illegible]	[illegible]	
		[illegible]	[illegible]	
	5	(321° — 7°)		
		[illegible]	[illegible]	
670		(295° — 8°)		
		31.2	[illegible]	
	10	(358° — 16°)		
665		51.4	65	
	15			
		[illegible]	53	
660				
	20			

SM SILTY SAND - well graded, oiled

CL SILTY CLAY - dark grey

Some shale gravel

SHALE - weathered, fractured, mottled light brown and light greyish-brown

Diatomaceous

Mottled greyish-white

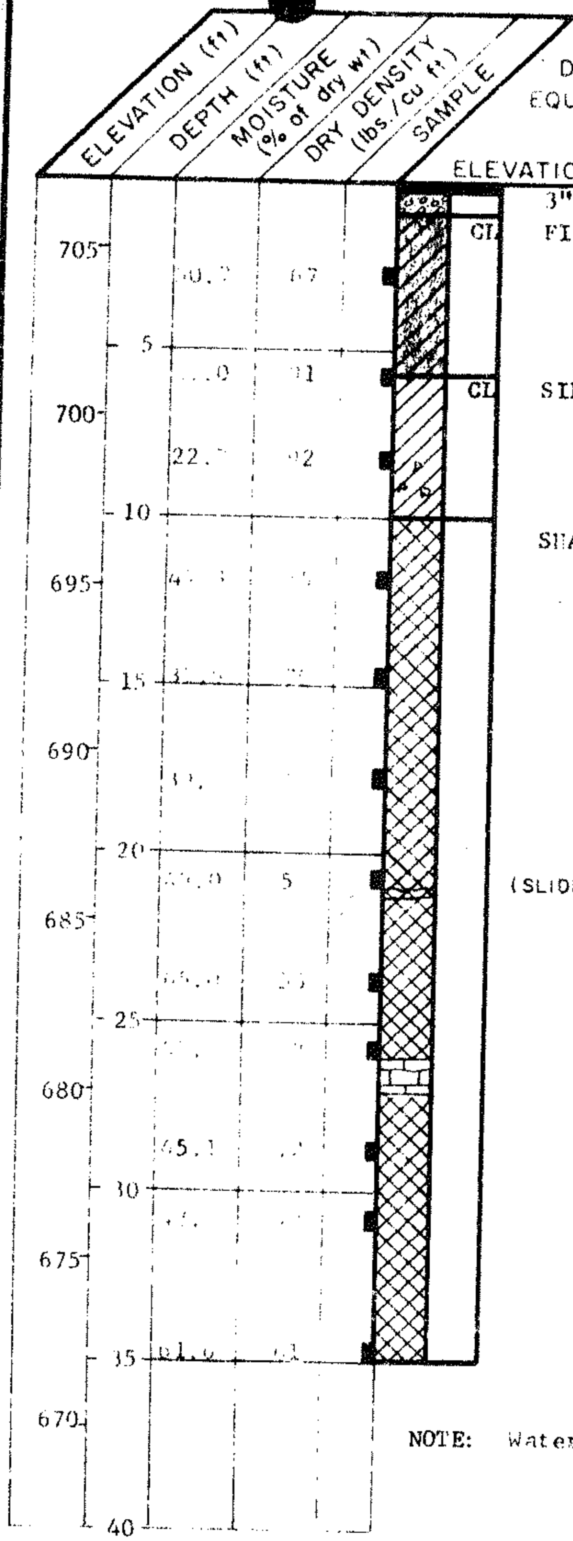
NOTE: Water not encountered. No caving.

JOB AG 61316 DATE 7-26-66 DR [illegible] CHKD [illegible]

LOG OF BORING

LEROY CRANDALL & ASSOCIATES

PLATE A-2



BORING 3

DATE DRILLED: May 10, 1966
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION 707

3" ASPHALTIC PAVING and 8" BASE COURSE

FILL - SILTY CLAY - lumps of shale, mottled brown and dark grey

SILTY CLAY - dark grey

Some Shale Gravel

SHALE - weathered, fractured, mottled light brown and yellowish-brown

Seams of Silty Clay - dark grey
Mottled light brown and light greyish-brown

Diatomaceous

(SLIDE PLANE)

Lenses of Sandstone

Layer of LIMESTONE (CHOPPING BUCKET USED FROM 26' TO 27')

NOTE: Water not encountered. No caving.

LOG OF BORING

LEROY CRANDALL & ASSOCIATES

PLATE A-3

JOB AGC 13/0 DATE 2 7-5-6 DR. CHKD

O 1 7 4

0 1 2 0 1 1 3 0 1 7 5

BORING 4

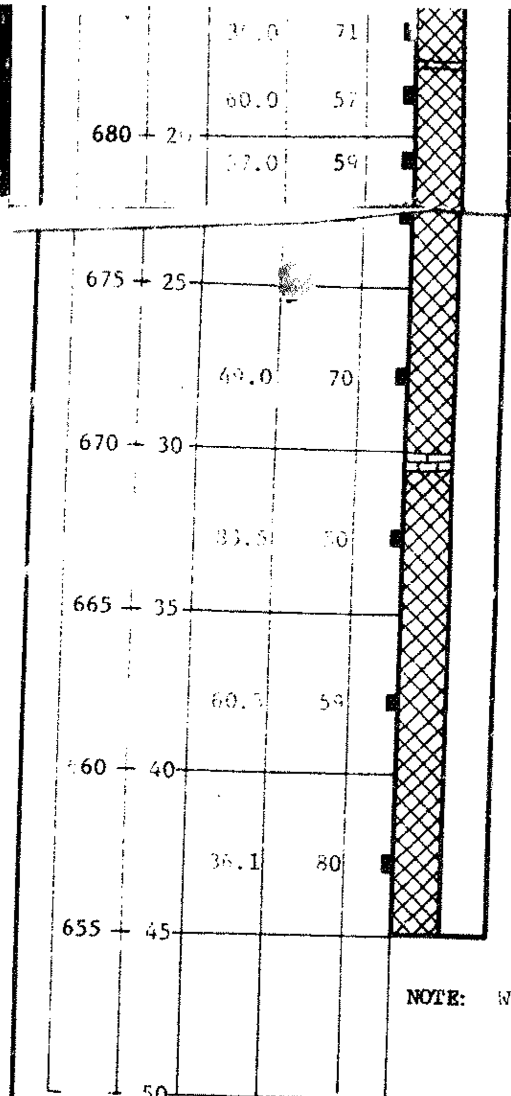
DATE DRILLED: May 11, 1966
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION 700

ELEVATION (ft)	DEPTH (ft)	MOISTURE (% of dry wt)	DRY DENSITY (lbs./cu ft)	SAMPLE	DESCRIPTION
					3" ASPHALTIC PAVING
		22.9	93	■	CL FILL - SILTY CLAY - lumps of shale, pieces of wood and concrete, mottled brown
					SHALE - weathered, fractured, mottled light brown and light greyish-brown
695	5	30.3	82	■	
		[illegible]	[illegible]	■	Diatomaceous
690	10	42.2	61	■	Mottled greyish-white
		[illegible]	[illegible]	■	
685	15				
		[illegible]	71	■	(SLIDE PLANE)
		60.0	57	■	Lenses of Sandstone Seams of Silty Clay - dark grey
680	20				
		[illegible].0	59	■	
675	25				
		[illegible].0	70	■	
670	30				Layer of LIMESTONE
		[illegible]	[illegible]	■	
665	35				
		[illegible]	[illegible]	■	Massive, dark brownish-grey
660	40				
		[illegible]	60	■	
655	45				
	50				

NOTE: Water not encountered. No caving.

mjm
Gmc OF HE CHKD



(SLIDE PLANE)
Lenses of Sandstone
Seams of Silty Clay - dark grey

Layer of LIMESTONE

Massive, dark brownish-grey

NOTE: Water not encountered. **No caving.**

JOB A66131 DATE 7-26-66 GMC O.E HE mjm CHKD FM

LOG OF BORING

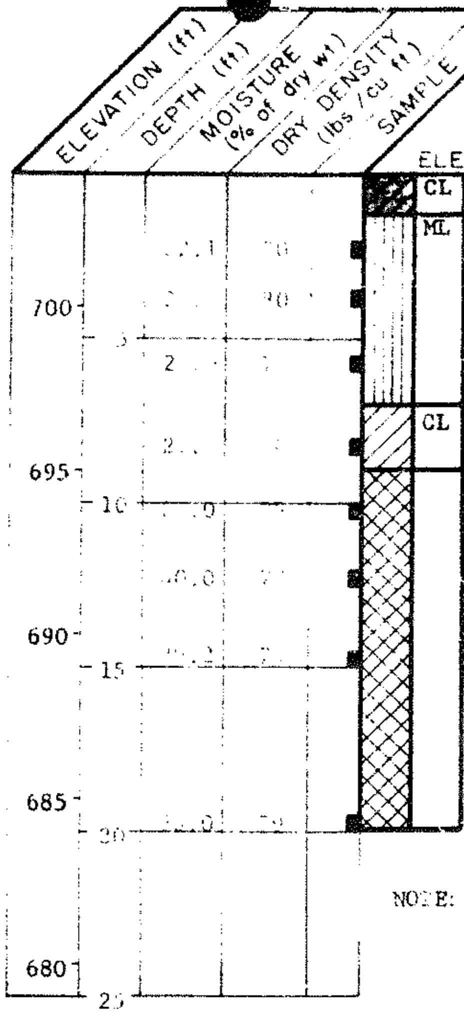
LEROY CRANDALL & ASSOCIATES

PLATE A-4

BORING 5

DATE DRILLED: May 11, 1966

EQUIPMENT USED: 18"-Diameter Bucket



ELEVATION 704

Symbol	Description
CL	FILL - SILTY CLAY - mottled brown and grey
ML	CLAYEY SILT - dark brown
	Root
CL	SILTY CLAY - mottled brown
	SHALE - weathered, highly fractured, mottled light brown and reddish-brown
	Greyish-brown

NOTE: Water not encountered. No caving.

LOG OF BORING

LEROY CRANDALL & ASSOCIATES

JOB [illegible] DATE 7-26-66 DR [illegible] CHKD [illegible]

BORING 6

DATE DRILLED May 11, 1966

EQUIPMENT USED 18"-Diameter Bucket

ELEVATION 690

ELEVATION (ft)	DEPTH (ft)	MOISTURE (% of dry wt)	DRY DENSITY (lbs/cu ft)	SAMPLE
		20.[illegible]	[illegible]5	■
		12.[illegible]	[illegible]2	■
685	5	[illegible].2	[illegible]	■
		25.5	70	■
680	10	[illegible].0	7[illegible]	■
		2[illegible] 1	70	■
675	15	[illegible]	[illegible]	■
670	20	[illegible]	[illegible]	■
665	25	[illegible].2	[illegible]	■
660	30			

FILL - SILTY SAND - fine, brown

CL FILL - SILTY CLAY - lumps of shale, few pieces of wood, dark greyish-brown

About 20% of diatomaceous shale

CL SILTY CLAY - brown

ML CLAYEY SILT - brown

Roots

CL SILTY CLAY - dark brown

Mottled brown

SHALE - weathered, diatomaceous, mottled light brown and light greyish-brown

NOTE: Water not encountered. No caving.

LOG OF BORING

LEROY CRANDALL & ASSOCIATES

BORING 7

DATE DRILLED: May 12, 1966
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION 714

ELEVATION (ft)	DEPTH (ft)	MOISTURE (% of dry wt)	DRY DENSITY (lbs/cu ft)	SAMPLE
		26.4	84	
710		26.0	95	
	5	4[illegible].7	67	
705		[illegible]3.0	57	
	10	[illegible]8.1	67	
700				
	15	3[illegible].2	79	
695	20			

SM CL — FILL - SILTY SAND and SILTY CLAY - mottled brown and dark greyish-brown

CL — SILTY CLAY - dark greyish-brown

SHALE - weathered, fractured, mottled light brown and light greyish-brown

(SLIDE PLANE)

Diatomaceous

Less fractured

NOTE: Water not encountered. No caving.

LOG OF BORING

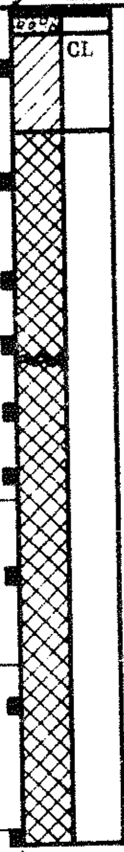
LEROY CRANDALL & ASSOCIATES

BORING 8

DATE DRILLED: May 12, 1966
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION 735

ELEVATION (ft)	DEPTH (ft)	MOISTURE (% of dry wt)	DRY DENSITY (lbs/cu ft)	SAMPLE
		22.[illegible]	77	■
730	5	[illegible]	[illegible]	■
		[illegible]	63	■
725	10	[illegible].0	70	■
		60.0	60	■
		[illegible].0	62	■
720	15			
		39.[illegible]	76	■
715	20			
		44.[illegible]	70	■
710	25	[illegible]	5[illegible]	■
705	30			



2" ASPHALTIC PAVING and 6" BASE COURSE

SILTY CLAY - dark greyish-brown

SHALE - weathered, fractured, mottled brown and light greyish-brown

Diatomaceous

Highly fractured

(SLIDE PLANE)

Less fractured

NOTE: Water not encountered. No caving.

LOG OF BORING

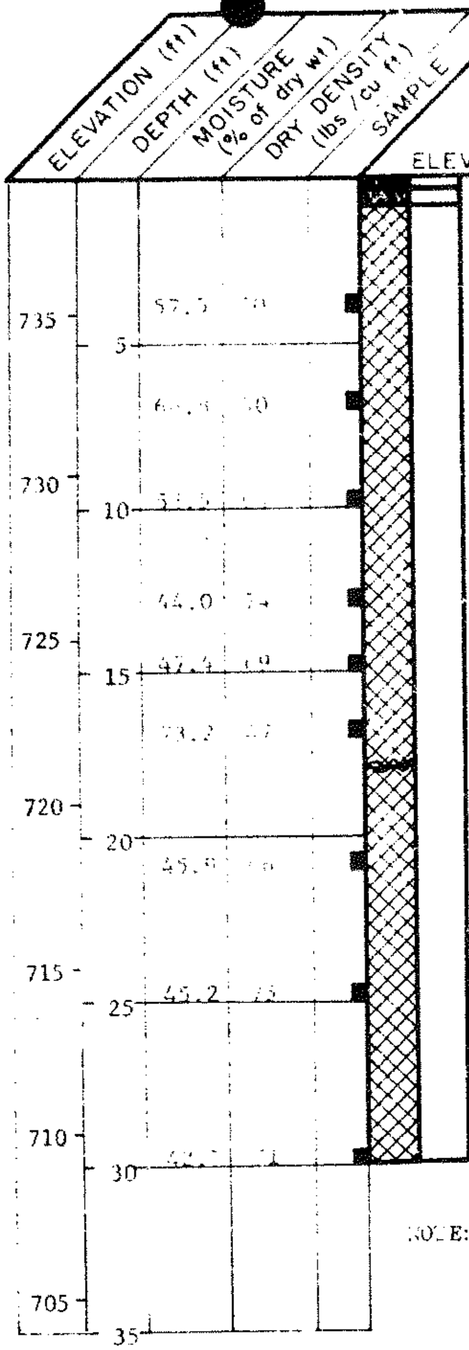
LEROY CRANDALL & ASSOCIATES

PLATE A-8

JOB AE-[illegible] DATE 7-[illegible]

BORING 9

DATE DRILLED: May 12, 1966
EQUIPMENT USED: 18"-Diameter Bucket



ELEVATION 739

4" ASPHALTIC PAVING and 5" BASE COURSE

SHALE - weathered, fractured, mottled light brown and light greyish-brown

Diatomaceous

Roots

Greyish-brown

Lenses of Sandstone

(SLIDE PLANE)

NOTE: Water not encountered. No caving.

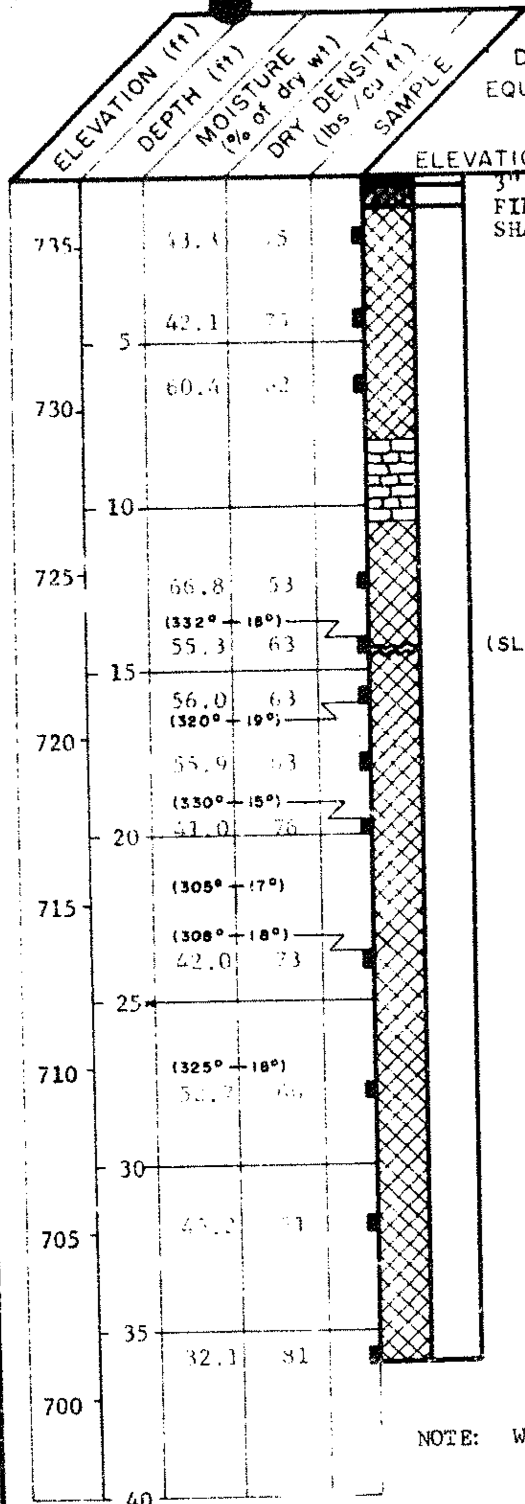
LOG OF BORING

LEROY CRANDALL & ASSOCIATES

PLATE A-9

BORING 10

DATE DRILLED July 15, 1966
EQUIPMENT USED 18"-Diameter Bucket



ELEVATION 737

3" ASPHALTIC PAVING
FILL - SILTY CLAY - mottled brown
SHALE - weathered, fractured, diatomaceous, mottled light brown and greyish-white

Layer of LIMESTONE (GAD AND CHOPPING BUCKET USED FROM 8' to 11')

(SLIDE PLANE)

Lenses of Sandstone

NOTE: Water not encountered. No caving.

LOG OF BORING

LEROY CRANDALL & ASSOCIATES

PLATE A-10

04201100182

BORING 11

DATE DRILLED July 15, 1966
EQUIPMENT USED. 18"-Diameter Bucket

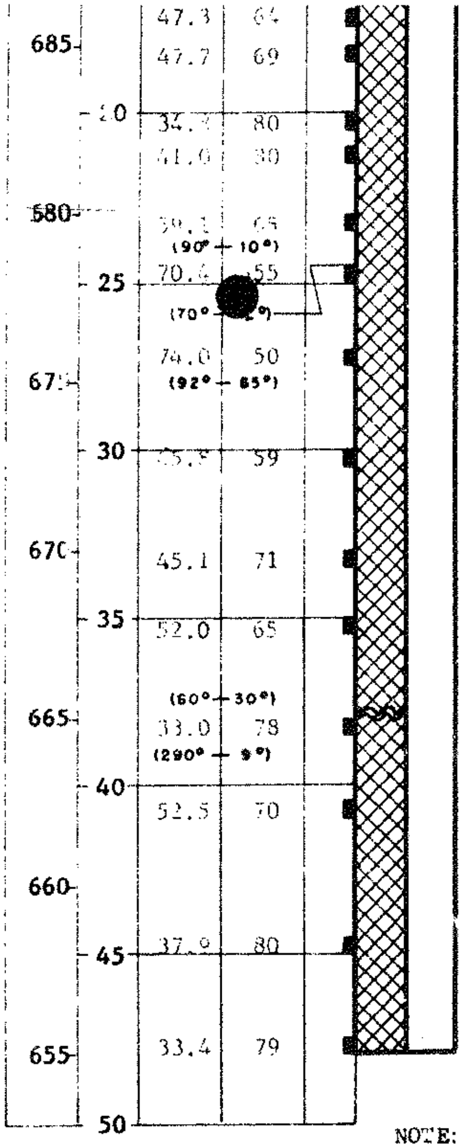
ELEVATION 703

ELEVATION (ft)	DEPTH (ft)	MOISTURE (% of dry wt)	DRY DENSITY (lbs./cu.ft.)	SAMPLE		
		32.0	84		CL SM	FILL - SILTY CLAY and SILTY SAND - mottled dark brown and brown
700		33.0	78			
		72.3	54			
	5				CL	SILTY CLAY - dar' grey
		(26° — 19°)				SHALE - weathere', fractured, mottled light brown and light greyish-brown Seams of SILTY CLAY - dark grey
695		26.0	8[illegible]			
		(30° — 13°)				
	10					Diatomaceous
		71.[illegible]	55			Lenses of Sandstone
690						
		43.1	69			
	15					
		(35° — 23°)				
		47.3	6[illegible]			
685		47.2	69			Hignly fractured
	20	34.[illegible]	80			
		41.0	80			Highly fractured
680						
		[illegible]	[illegible]			
		(90° — 10°)				
		70.2	55			
	25					
		(70° — [illegible]°)				
		76.0	50			
675		(92° — 85°)				
	30	[illegible]	59			
670		45.1	71			
	35	52.0	65			
		(60° — 30°)				Concretions (to 7" in diameter)
665		34.0	78			(SLIDE PLANE)
		(290° — 9°)				
	40	52.3	70			Massive, dark brownish-grey
660						
	45	37.2	80			
655		33.4	70			
	50					

NOTE: Slight water seepage encountered at a depth of 26'; no water in boring at completion of drilling. Slight raveling in the highly fractured zones; no caving.

CHKD

JOB A66131 DATE 7-26-66 GMC O.E. HE CHKD



Highly fractured

Highly fractured

Concretions (to 7" in diameter)

(SLIDE PLANE)

Massive, dark brownish-grey

NOTE: Slight water seepage encountered at a depth of 26'; no water in boring at completion of drilling. Slight raveling in the highly fractured zones; no caving.

LOG OF BORING

LEROY CRANDALL & ASSOCIATES

PLATE A-11

0 4 3 0 1 1 0 0 1 8 3

BORING 12

DATE DRILLED: July 16, 1966

EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION 683

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu.ft.)	SAMPLE
		23.1	83	
680		41.0	56	
	5	45.1	67	
675		64.2	62	
	10	42.0	72	
670		(330° – 20°)		
	15	37.0	78	
		40.5	79	
665		57.5	60	
		(310° – 23°)		
	20	[illegible]	[illegible]	
660		(310° – 13°)		
		71.8	63	
	25	63.5	62	
		(320° – 84°)		
655		62.7	54	
	30	(302° – 23°)		
		[illegible]	[illegible]	
		(295° – 17°)		
650		54.7	65	
	35	70.0	54	
645		(310° – 18°)		
		42.6	73	
		(280° – 10°)		
	40	54.5	77	
640		[illegible]	[illegible]	
	45			
635		[illegible]	[illegible]	
	50			

CL SILTY CLAY - dark grey

SHALE - weathered, fractured, mottled light brown and yellowish-brown

Lenses of Sandstone

Diatomaceous

Mottled light brown and light greyish-brown

Highly fractured

(SLIDE PLANE)

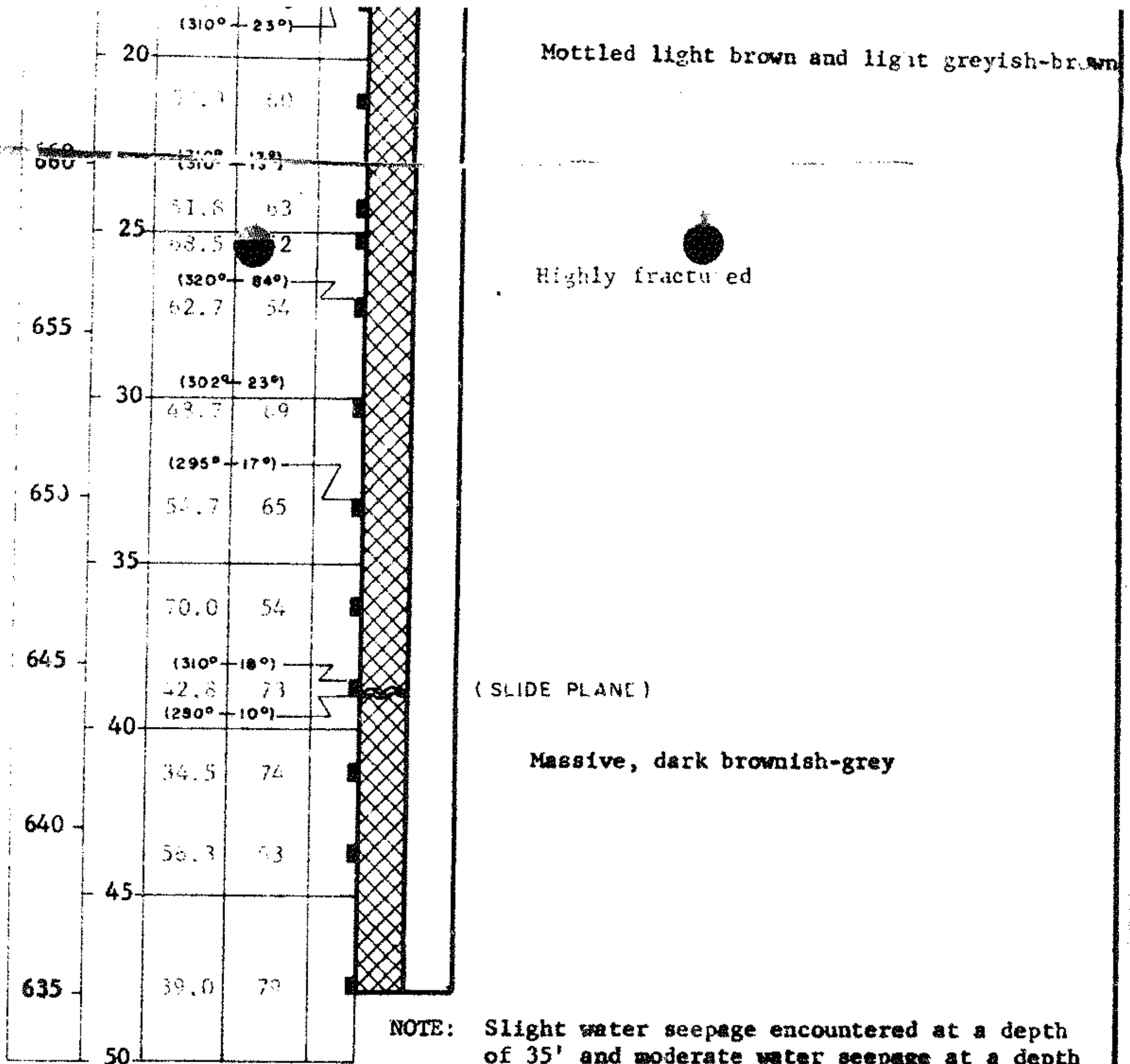
Massive, dark brownish-grey

NOTE: Slight water seepage encountered at a depth of 35' and moderate water seepage at a depth of 39'; no water in boring at completion of drilling. Slight raveling in the highly

cr CHKD

JOB AGG131 DATE 7-26-66 GMC O.E. HE CHKD [illegible]

cr



NOTE: Slight water seepage encountered at a depth of 35' and moderate water seepage at a depth of 39'; no water in boring at completion of drilling. Slight raveling in the highly fractured zones; no caving.

LOG OF BORING

LEROY CRANDALL & ASSOCIATES

PLATE A-12

BORING A (PIT)

DATE DUG May 12, 1966
EQUIPMENT USED Hand Shovel

ELEVATION 802

ELEVATION (ft)	DEPTH (ft)	MOISTURE (% of dry wt)	DRY DENSITY (lbs/cu ft)	SAMPLE
800		31.7	64	
	5			
795				
	10			

CL SILTY CLAY - roots, brown

SHALE - weathered, highly fractured, mottled brown

NOTE: Water not encountered.

BORING B (PIT)

DATE DUG May 12, 1966
EQUIPMENT USED Hand Shovel

ELEVATION 800

ELEVATION (ft)	DEPTH (ft)	MOISTURE (% of dry wt)	DRY DENSITY (lbs/cu ft)	SAMPLE
		27.9	78	
795	5			
790	10			

CL SILTY CLAY - some sand, roots, brown

SHALE - weathered, highly fractured, mottled brown

NOTE: Water not encountered.

BORING C (PIT)

DATE DUG May 12, 1966
EQUIPMENT USED Hand Shovel

ELEVATION 824

ELEVATION (ft)	DEPTH (ft)	MOISTURE (% of dry wt)	DRY DENSITY (lbs/cu ft)	SAMPLE
820		39.2	7[illegible]	
	5			
815				
	10			

CL SILTY CLAY - some sand, roots, brown

SHALE - weathered, highly fractured, mottled brown

NOTE: Water not encountered.

LOG OF BORING

LEROY CRANDALL & ASSOCIATES

BORING D (PIT)

DATE DUG May 12, 1966
EQUIPMENT USED : Hand Shovel

ELEVATION 842

ELEVATION (ft.)	DEPTH (ft)	MOISTURE (% of dry wt)	DRY DENSITY (lbs/cu ft)	SAMPLE
840				
	5	38.6	71	■
835				
	10			

CL SILTY CLAY - rootlets, brown

SHALE - weathered, highly fractured, mottled brown

NOTE: Water not encountered.

BORING E (PIT)

DATE DUG May 12, 1966
EQUIPMENT USED : Hand Shovel

ELEVATION 823

ELEVATION (ft.)	DEPTH (ft)	MOISTURE (% of dry wt)	DRY DENSITY (lbs/cu ft)	SAMPLE
820		25.7	82	■
	5			
815				
	10			

CL SILTY CLAY - brown

SHALE - weathered, highly fractured, mottled brown

NOTE: Water not encountered.

BORING F (PIT)

DATE DUG May 12, 1966
EQUIPMENT USED : Hand Shovel

ELEVATION 729

ELEVATION (ft.)	DEPTH (ft)	MOISTURE (% of dry wt)	DRY DENSITY (lbs/cu ft)	SAMPLE
725		33.1	69	■
	5			
720				
	10			

CL FILL - SILTY CLAY - chunks of shale, dark brown

CL SILTY CLAY - brown

SHALE - weathered, highly fractured, mottled brown

NOTE: Water not encountered.

LOG OF BORING

LEROY CRANDALL & ASSOCIATES

PLATE A-14

JOB AG0131 DATE 7-29-66 BY CHKD

ELEVATION (ft) | DEPTH (ft) | MOISTURE (% of dry wt) | DRY DENSITY (lbs/cu ft) | SAMPLE

BORING ...

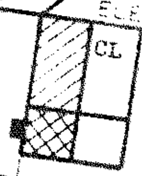
DATE DUG: May 1...

EQUIPMENT USED: Hand Sh...

ELEVATION 729

725 — 5

720 — 10



SILTY CLAY - roots (to 10"), da...

SHALE - weathered, fractured, mottle...

NOTE: Water not encountered.

BORING H (PIT)

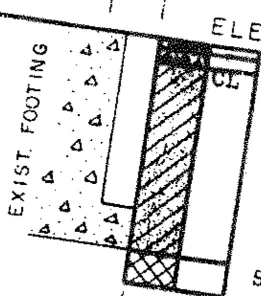
DATE DUG: July 21, 1966

EQUIPMENT USED: Hand Shovel to 3½' and 6"-Diameter Auger below 3½'

ELEVATION 740

735 — 5

730 — 10



3" BRICK and 5" CONCRETE

FILL - SILTY CLAY - lumps of shale, dark grey

SHALE - weathered, fractured, mottled light brown and light greyish-brown

NOTE: Water not encountered.

1-26-66 DR ... CHKD ... PM

LOG OF BORING

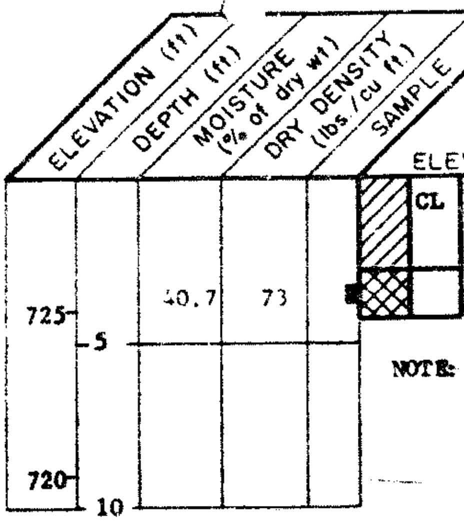
LEROY CRANDALL & ASSOCIATES

PLATE A-15

BORING G (PIT)

DATE DUG May 12, 1966
EQUIPMENT USED: Hand Shovel

ELEVATION 729



SILTY CLAY - roots (to 10"), dark greyish-brown

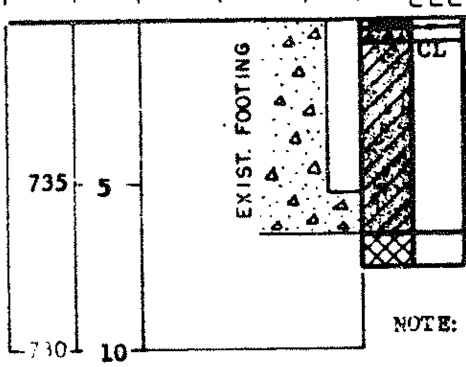
SHALE - weathered, fractured, mottled brown

NOTE: Water not encountered.

BORING H (PIT)

DATE DUG: July 21, 1966
EQUIPMENT USED: Hand Shovel to 3½' and 6"-Diameter Auger below 3½'

ELEVATION 740



3" BRICK and 5" CONCRETE
FILL - SILTY CLAY - lumps of shale, dark grey

SHALE - weathered, fractured, mottled light brown and light greyish-brown

NOTE: Water not encountered.

LOG OF BORING

LEROY CRANDALL & ASSOCIATES

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS (More than 50% of material is LARGER than No 200 sieve size)	GRAVELS (More than 50% of coarse fraction is LARGER than the No 4 sieve size)	CLEAN GRAVELS (Little or no fines)	GW	Well graded gravels, gravel sand mixtures, little or no fines
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines
		GRAVELS WITH FINES (Appreciable amt of fines)	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	SANDS (More than 50% of coarse fraction is SMALLER than the No 4 sieve size)	CLEAN SANDS (Little or no fines)	SW	Well graded sands, gravelly sands, little or no fines
			SP	Poorly graded sands or gravelly sands, little or no fines
		SANDS WITH FINES (Appreciable amt of fines)	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
FINE GRAINED SOILS (More than 50% of material is SMALLER than No 200 sieve size)	SILTS AND CLAYS (Liquid limit LESS than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silty clays of low plasticity
	SILTS AND CLAYS (Liquid limit GREATER than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity, organic silts
HIGHLY ORGANIC SOILS			Pt	Peat and other highly organic soils

BOUNDARY CLASSIFICATIONS Soils possessing characteristics of two groups are designated by combinations of group symbols

PARTICLE SIZE LIMITS

SILT OR CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

NO 200 NO 40 NO 10 NO 4 3/4 in 3 in 12 in

U S STANDARD SIEVE SIZE

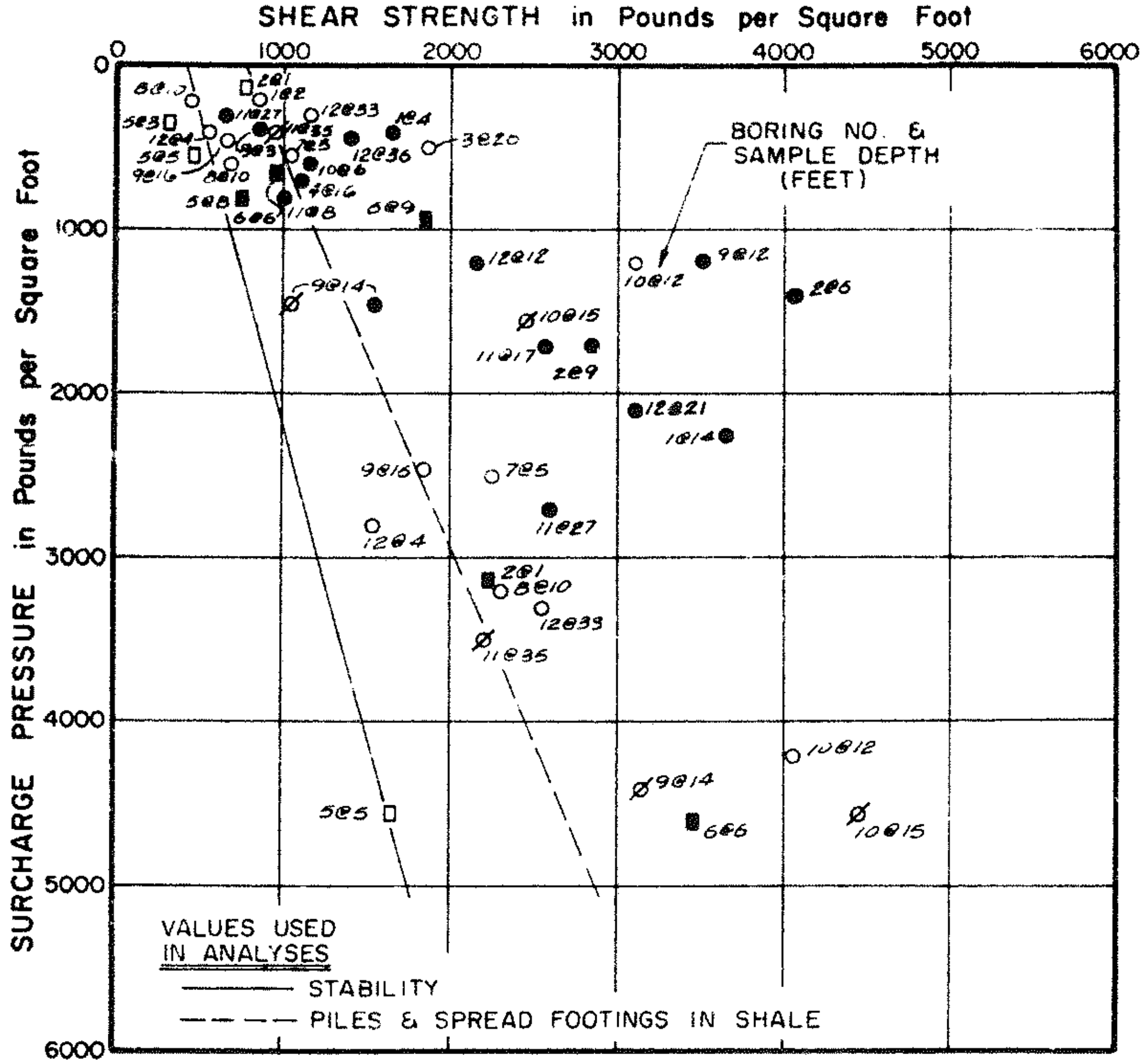
UNIFIED SOIL CLASSIFICATION SYSTEM

Reference
The Unified Soil Classification System, Corps of Engineers, U S Army Technical Memorandum No 3-357, Vol 1, March, 1953 (Revised April, 1960)

LEROY CRANDALL AND ASSOCIATES

PLATE B

04201100188



KEY:

□ ■ OVERBURDEN (Top soil)

○ ● SHALE within slide mass

ø SHALE within slide mass (Samples purposely broken along shear plane prior to soaking)

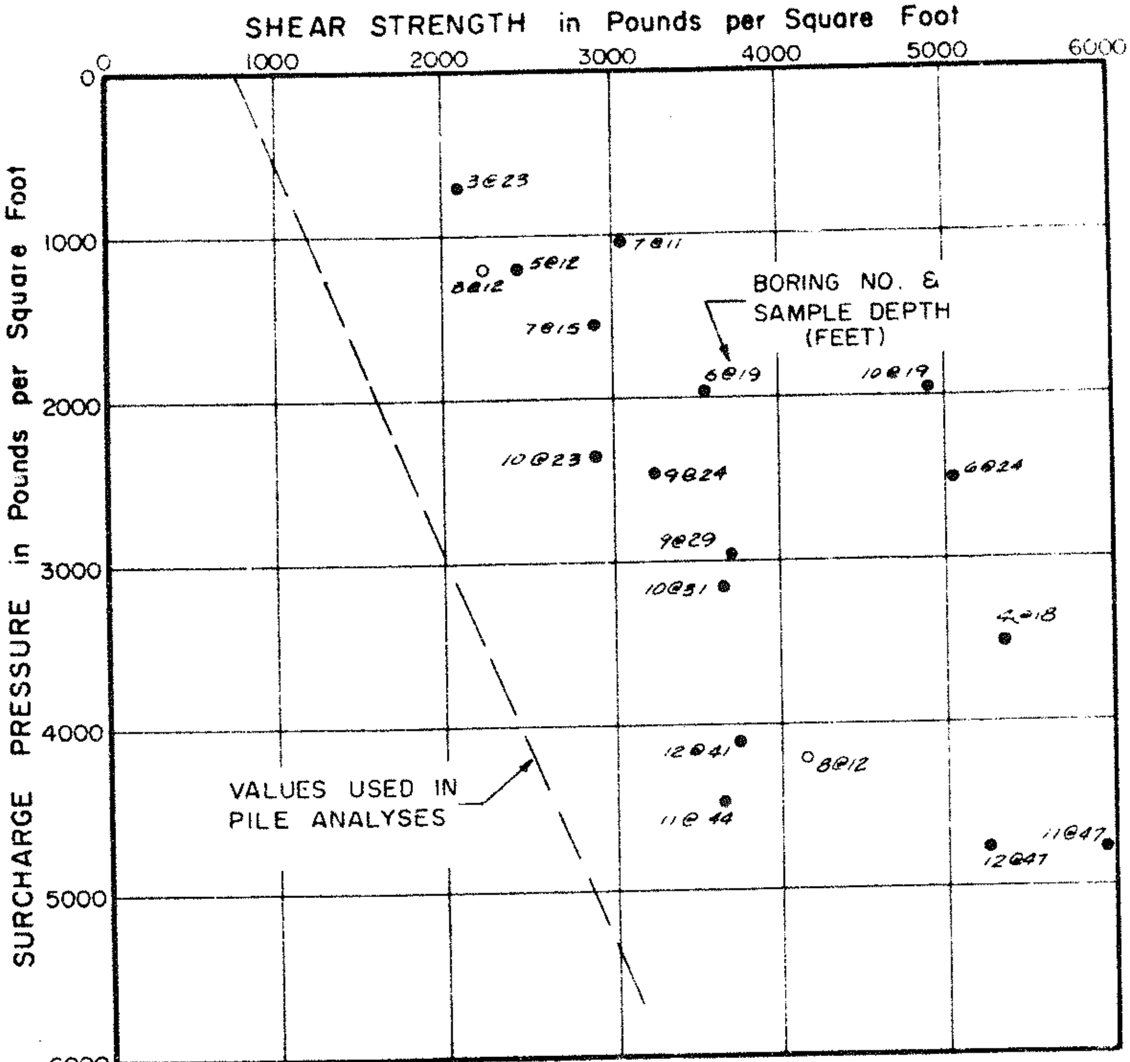
Tests at field moisture content

Tests at increased moisture content

DIRECT SHEAR TEST DATA

LeROY CRANDALL & ASSOCIATES

PLATE C-1



KEY:
- ● Tests at field moisture content
- ○ Tests at increased moisture content

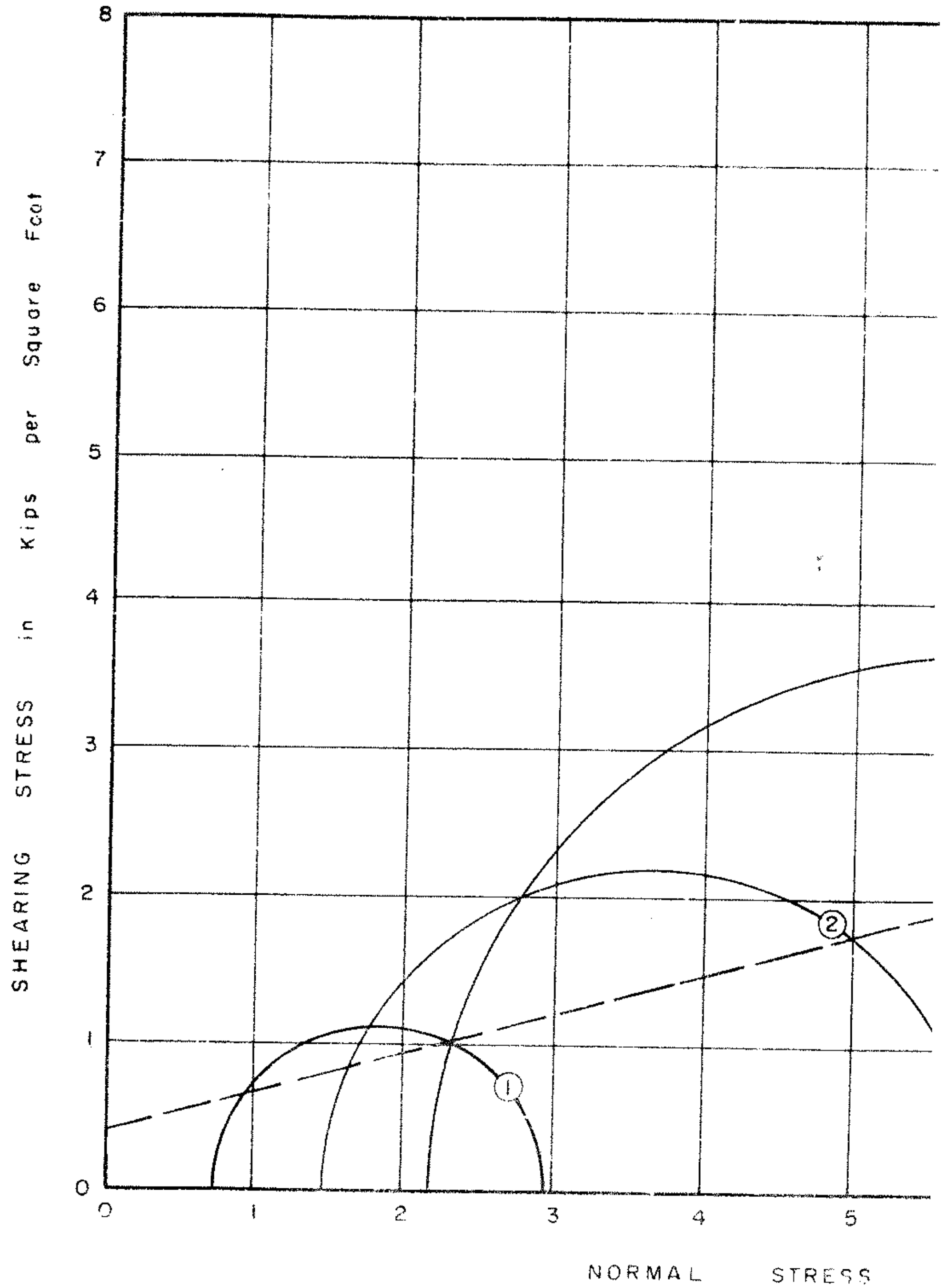
NOTE: SAMPLES TAKEN FROM SHALE BELOW SLIDE PLANE.

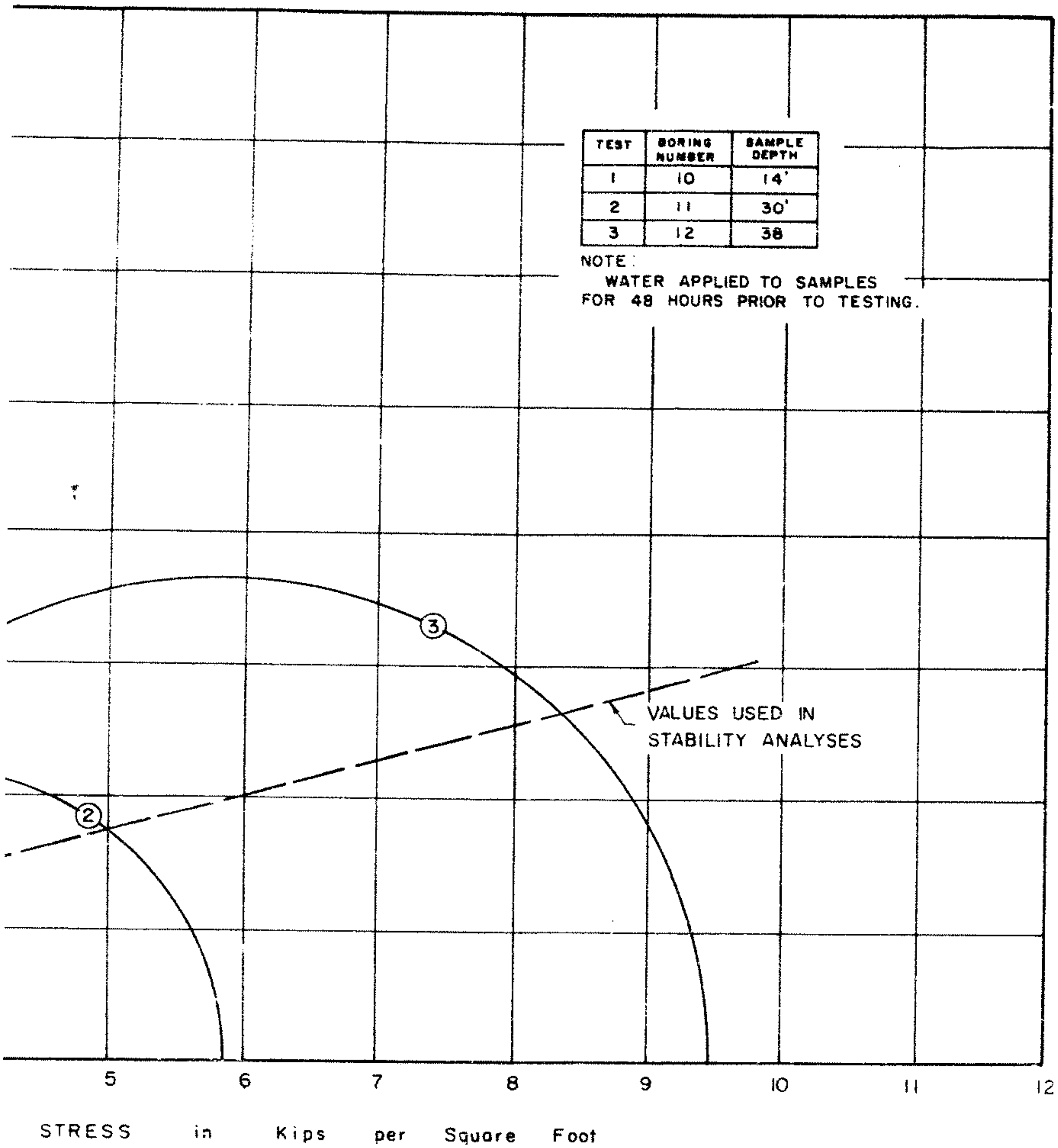
DIRECT SHEAR TEST DATA

LEROY CRANDALL & ASSOCIATES

PLATE C-2

JOB ____ DATE 2 0 1 OR 0 0 1 9 OF HE ____ CHKD. ____ PM

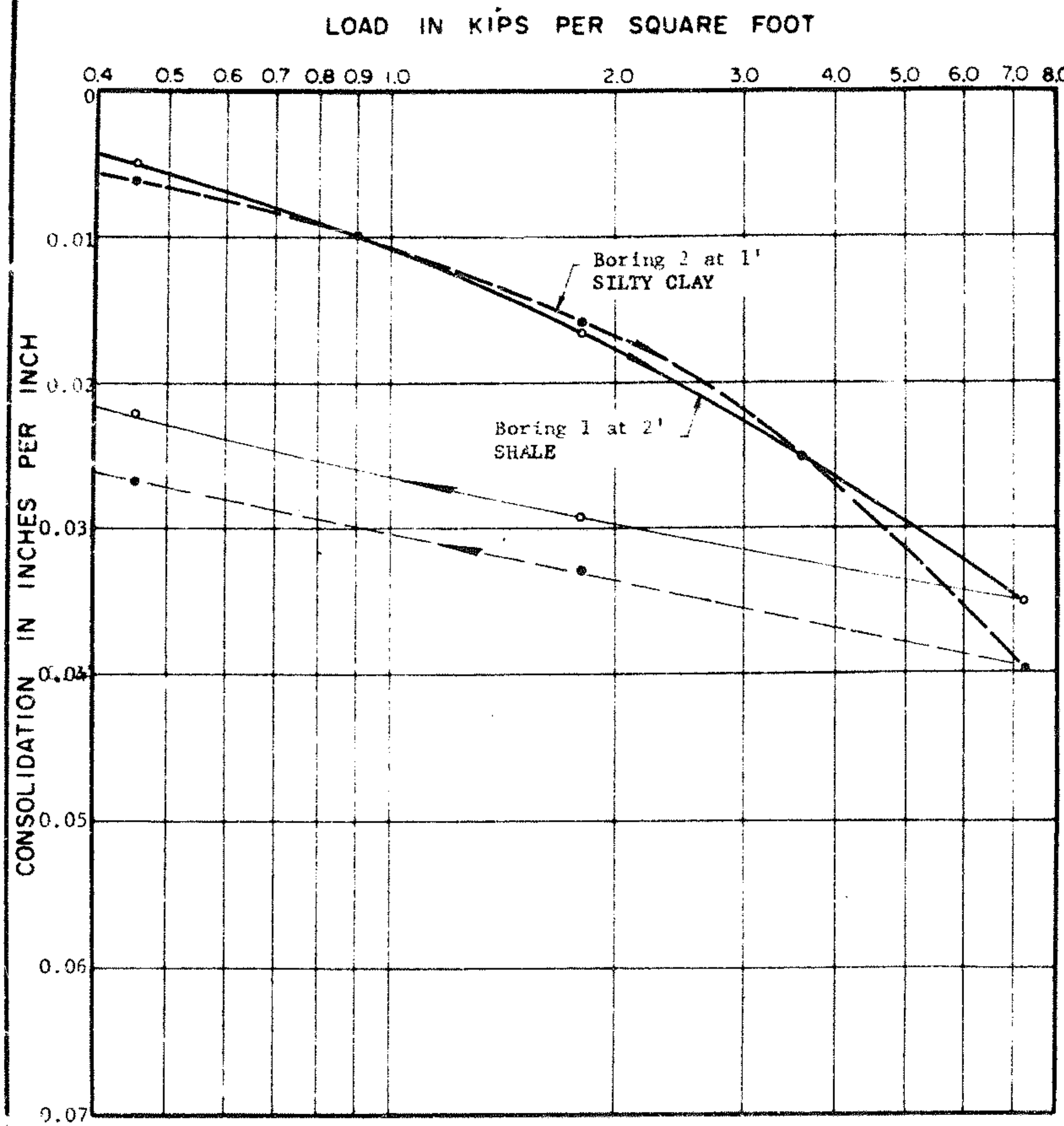




TRIAXIAL SHEAR TEST DATA

LeRoy Crandall & Associates

PLATE C-3

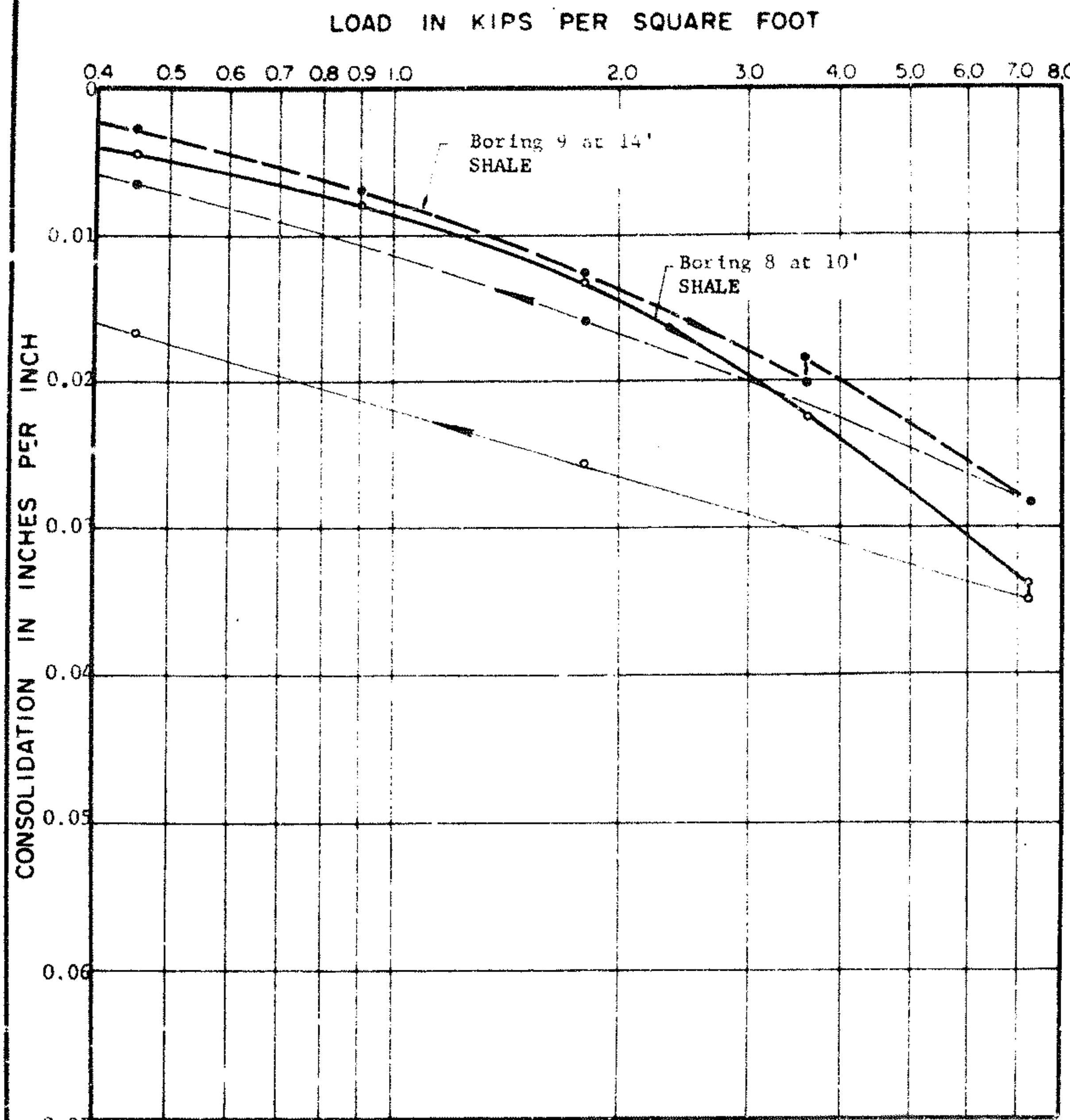


NOTE: Samples tested at field moisture content.

CONSOLIDATION TEST DATA

LEROY CRANDALL & ASSOCIATES

PLATE D-1

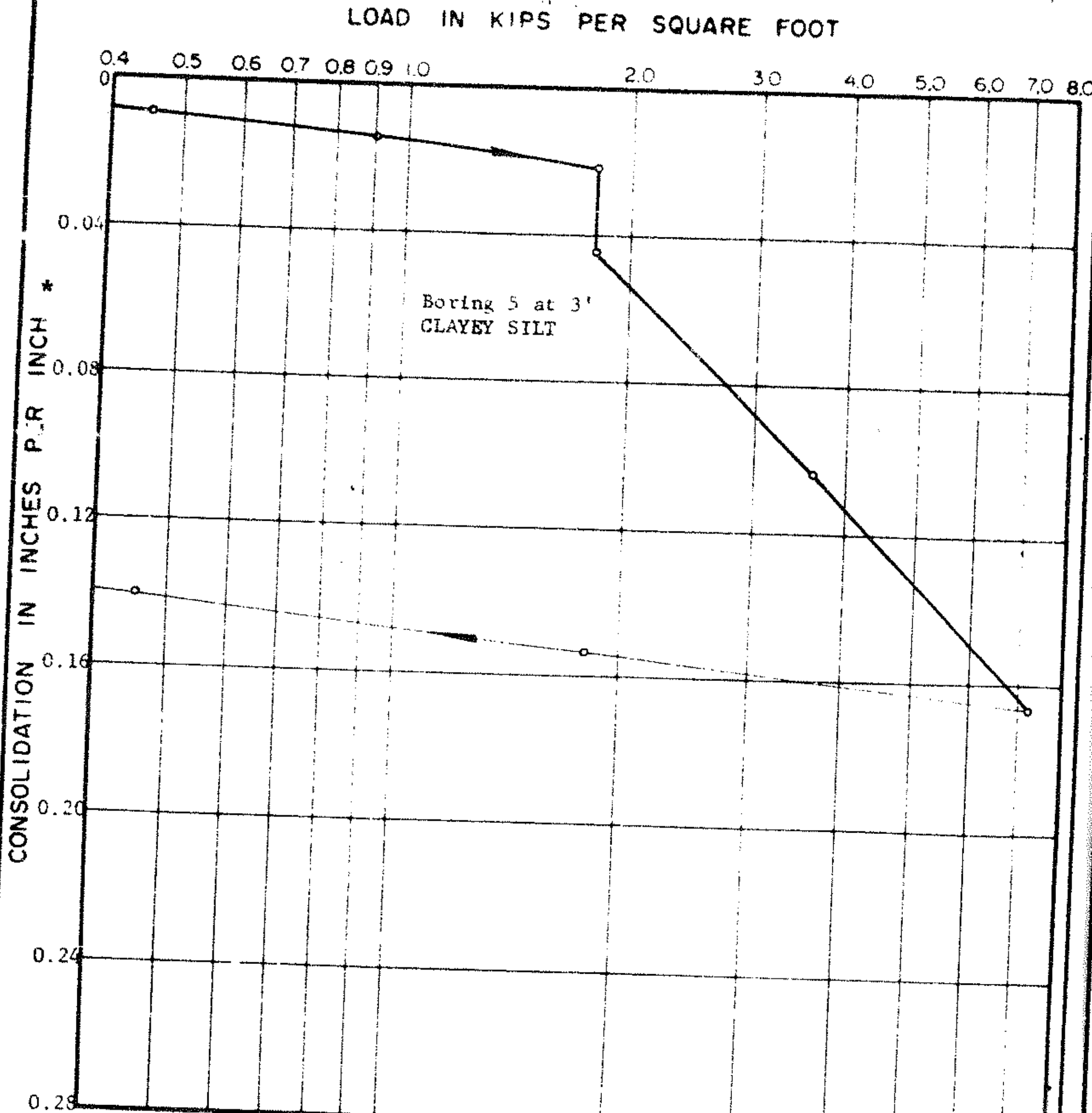


NOTE: Water added to samples from Borings 8 and 9 after consolidation under loads of 7.2 and 3.6 kips per square foot, respectively.

CONSOLIDATION TEST DATA

LEROY CRANDALL & ASSOCIATES

PLATE D-2



NOTE: Water added to sample after consolidation under a load of 1.8 kips per square foot.

*NOTE CHANGE IN SCALE

CONSOLIDATION TEST DATA

LEROY CRANDALL & ASSOCIATES

PLATE D-3

APPENDIX B

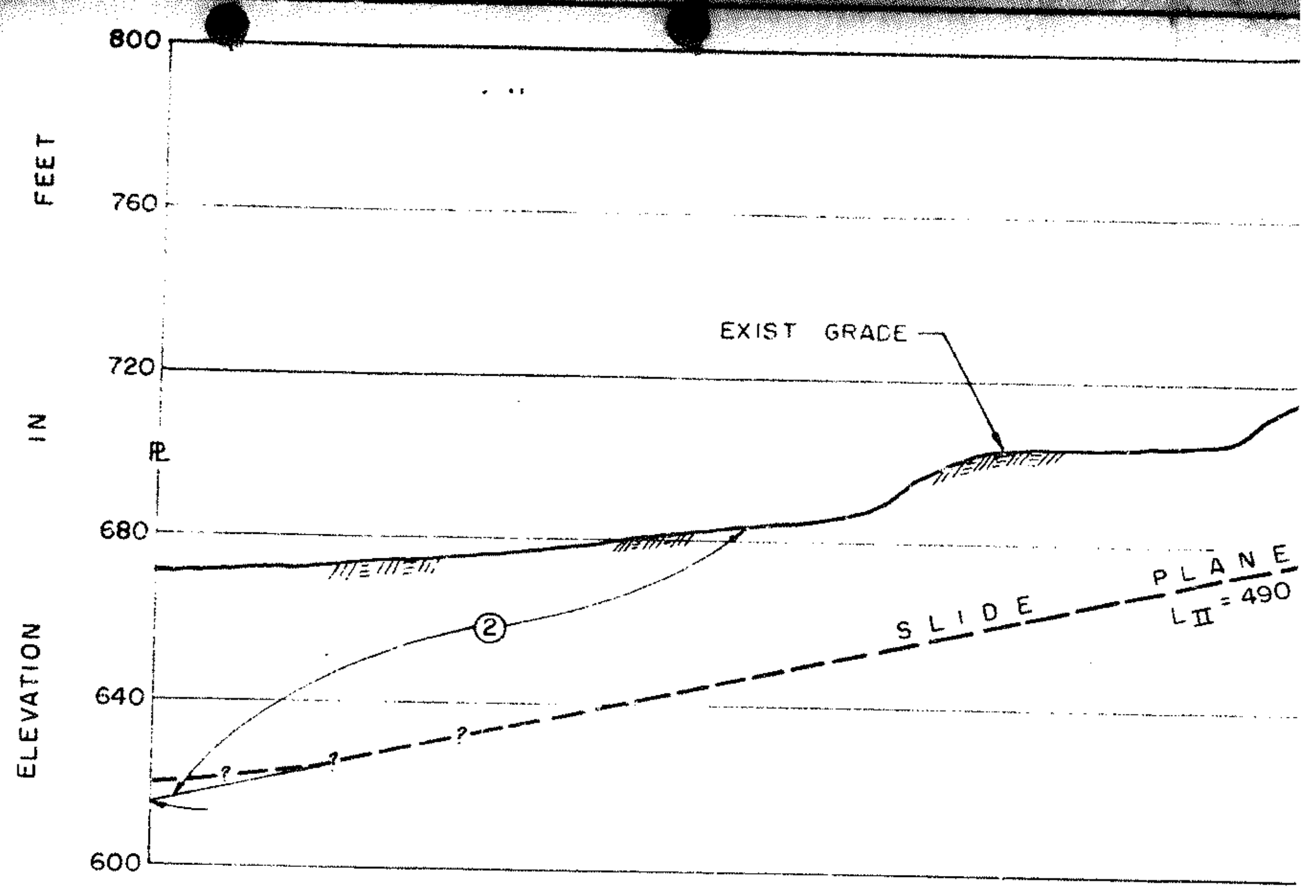
APPENDIX B

STABILITY ANALYSES

The stability of the ancient slide was analyzed along a section taken through the central portion of the slide mass; this section presents the most critical case. The analyses are presented on Plate E, Stability Analyses. For the calculations, a cohesion of 400 pounds per square foot and a friction angle of 15 degrees were used for the shale along the potential slide plane. These values are based on the conservative interpretation of the shear test data, and reflect the wet strength of the weaker materials along the bedding planes.

The analyses assumed that the potential slide would occur along the old slide plane. Since the extent of the slide at and beyond the property line is not known, we have considered, in our analyses, only that portion of the slide which is within the property. This is conservative because it does not include the additional resistance of the mass at the toe of the slide. Also, we have considered in our analyses the full weight of the slide mass without reduction for the proposed cut at the building locations and the surrounding areas. (The loads to be imposed by the proposed buildings would be less than the weight of the soil excavation.) Based on the results of our analyses, the slide mass has an overall factor of safety of 1.6 against sliding under static conditions.

-o0o-



Weight of the Slide Mass

$W = \text{Area} \times 1 \times \gamma$

$W_I = 700 \times 1 \times 0.11 = 77$ kips

$W_{II} = 18{,}960 \times 1 \times 0.11 = 2085$ kips

DRIVING FORCE

Horizontal Component

$= (W_I \sin 27.5^\circ)(\cos 27.5^\circ) +$
$(W_{II} \sin 12.5^\circ)(\cos 12.5^\circ)$

$= 77 \times 0.462 \times 0.887 +$
$2085 \times 0.216 \times 0.976$

$= 32 + 441 = 473$ kips

RESISTING FORCE

Horizontal Compone

$= (W_I \cos 27.5^\circ$

$(W_{II} \cos 12.5$

$= (77 \times 0.887 \times$
$(2085 \times 0.97$

$= 41 + 724$

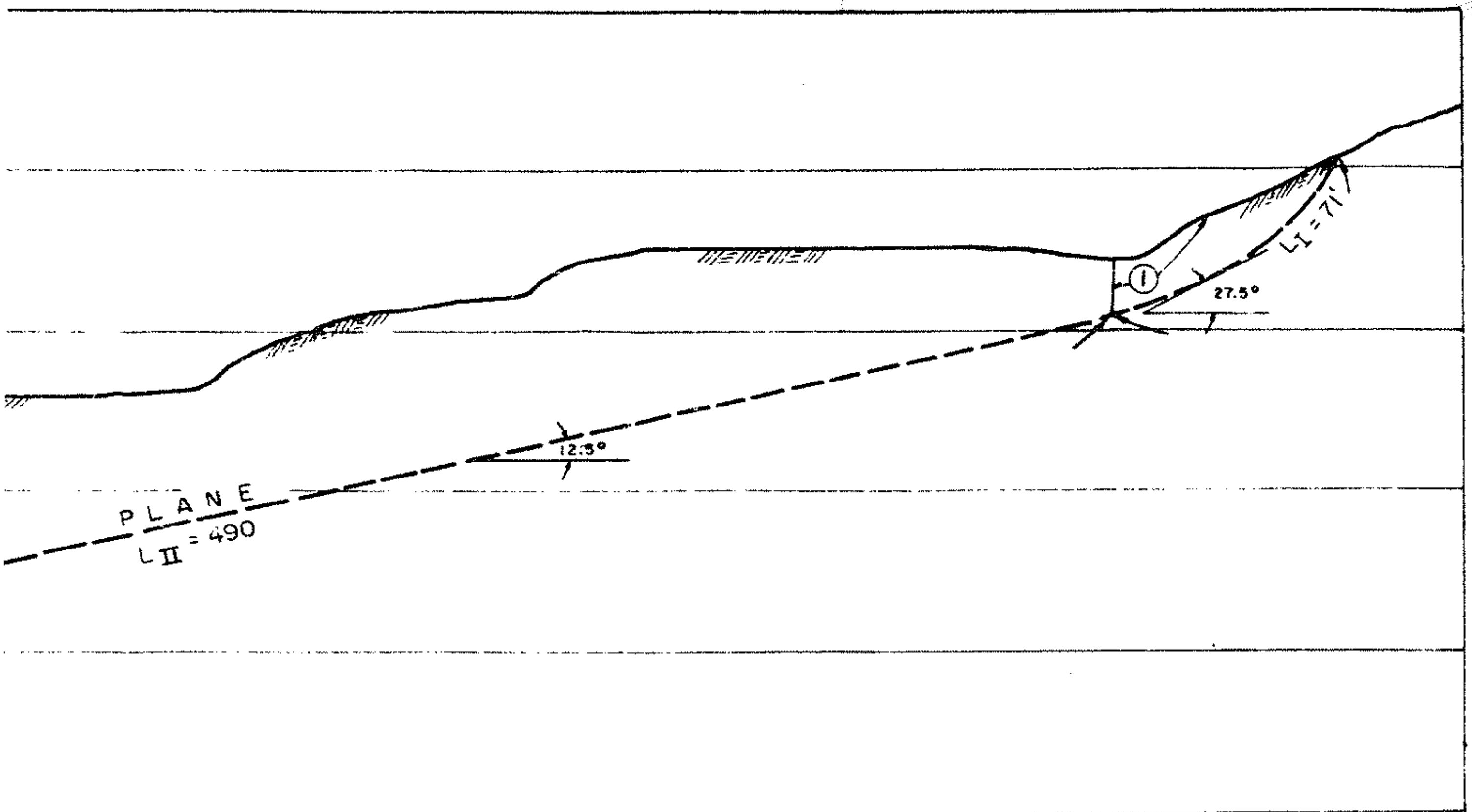
$= 765$ kips

FACTOR OF SAFETY

$F.S. = \frac{\text{Horizontal Com}}{\text{Horizontal Com}}$

$F.S. = \frac{765}{473} = 1.6$

NOTE: ANALYSIS BASED ON ONE FOOT WIDE STRIP



SOIL DATA

$C = 0.40$ ksf

$\phi = 15°$

$\gamma = 0.11$ kcf

ESISTING FORCE

Horizontal Component

$= (W_I \cos 27.5^\circ \tan 15^\circ + CL_I)(\cos 27.5^\circ) +$

$(W_{II} \cos 12.5^\circ \tan 15^\circ + CL_{II})(\cos 12.5^\circ)$

$= (77 \times 0.887 \times 0.268 + 0.40 \times 71)(0.887) +$

$(2085 \times 0.976 \times 0.268 + 0.40 \times 490)(0.976)$

$= 41 + 724$

$= 765$ kips

ACTOR OF SAFETY

$.S. = \dfrac{\text{Horizontal Component of Resisting Forces}}{\text{Horizontal Component of Driving Forces}}$

$.S. = \dfrac{765}{473} = \underline{1.6}$

STABILITY ANALYSES

SCALE 1" = 40'

LEROY CRANDALL & ASSOCIATES

PLATE E

APPENDIX C

FIELD PERCOLATION STUDIES
PROPOSED SUBSURFACE DRAINAGE SYSTEM
ANCIENT LANDSLIDE MASS
3700 COLDWATER CANYON DRIVE
LOS ANGELES, CALIFORNIA
FOR THE
HARVARD SCHOOL
(OUR JOB NO. A-66131-E)

LeROY CRANDALL

December 2, 1966

Johnson & Silvestri & Associates
13135 Ventura Boulevard
North Hollywood, California 91604

(Our Job No. A-66131-B)

Gentlemen:

Field Percolation Studies
Proposed Subsurface Drainage System
Ancient Landslide Mass
3700 Coldwater Canyon Drive
Los Angeles, California
for the Harvard School

SCOPE

This report presents the results of our field percolation studies at the subject site and our recommendations for subsurface drainage. We previously investigated the site and submitted our report of foundation investigation on August 26, 1966. In our initial investigation it was discovered that an ancient slide existed at the site of the proposed development; the extent of the slide is shown on Plate 1, Plot Plan.

From the tests and analyses presented in our report of August 26, 1966, it was found that the slide mass is stable under the present conditions, and that the site can be developed for the proposed buildings if hydrostatic pressures are not allowed to develop within the slide mass. We recommended that a suitable drainage system be devised to assure that future seepage would not jeopardize the stability of the site. It was then concluded that two rows of vertical sand drains would be [illegible]

04201100200

method of intercepting any future seepage. The present investigation was authorized to evaluate the possibility of the vertical sand drains functioning as dry wells to dispose of seepage below the slide plane. The results of the investigation were discussed with Messrs. Johnson and Silvestri, and with Mr. Harold King of King.Benioff.Steinmann.King, Consulting Engineers.

FIELD PERCOLATION STUDIES

To study the feasibility of seepage pits, it was planned to drill two borings, one in the upper part of the slide mass and one in the lower part. As discussed later, however, three borings were made. The logs of the three borings, numbered 13, 14, and 15, are presented on Plates 2-A through 2-C, Logs of Borings.

Boring 13 was drilled in the lower part of the slide to a depth of 70 feet. From 46 feet to 50 feet a layer of very hard limestone was encountered which required about 13 hours to penetrate with a gad and chopping bucket. The slide plane was found to be at 35 feet in depth. Slight water seepage occurred at a depth of 46 feet, and heavy seepage was encountered at a depth of 63 feet. As described on the boring log, water was added to the hole to check the percolation rate. However, it was found that the water level rose instead of dropping; this condition was due to the seepage at 63 feet being under pressure.

Because of the unfavorable seepage condition of Boring 13, Boring 14 was drilled near Boring 13. This boring was drilled to a depth of only 53 feet to avoid the heavy water seepage encountered at a depth of 63 feet in Boring 13. As noted on the boring log, the hard limestone layer at this location was only one foot thick; the slide plane was at a depth of 36 feet. As described on the log of Boring 14, a percolation test was also attempted in this boring. Although not so pronounced as in Boring 13, the water surface also rose rather than dropping.

Boring 15 was drilled in the upper portion of the slide to a depth of 50 feet below the existing grade. The slide plane was found at a depth of 19 feet; seepage water did not occur in this boring. Although the percolation test showed seepage from the boring, the rate was nominal. At the end of the percolation tests, the remaining water was removed from the borings and the holes backfilled with compacted earth.

CONCLUSIONS AND RECOMMENDATIONS

As discussed above, the two percolation tests at the lower end of the slide mass were completely unsuccessful, since there was no seepage of water into the soils. While the percolation test at Boring 15 indicated some seepage into the soils, the rate was disappointing. From these results, it is apparent that seepage pits will not be feasible to dispose of any subsurface seepage.

Since the dry well technique is not feasible, some means of collecting water from the vertical sand drains must be devised. While individual pumps could be put into each vertical drain, we believe that it will be more economical to interconnect each of the two rows of vertical drains as shown on Plate 3, Subsurface Drainage System. The drains should extend at least six feet below the slide plane and will connect at their bottoms to drain to a common sump. The vertical drains are shown at a spacing of [illegible] feet center to center on Plate 5; this spacing is not final and may be modified depending on the contractor's method of installation. The spacing at seven feet is based on the assumption that the drains will be connected by either belling or by slot-cutting. If the contractor chooses to connect the drains by boring or tunneling, a larger spacing may be used up to a maximum spacing of 15 feet on centers.

The drains should be filled with washed concrete sand, except for the top six feet, which should be plugged with impermeable soils compacted into place. If the drains are connected by boring or tunneling, then a section of pipe could be installed to connect the bottoms of the drains rather than the sand. An adequate slope should be provided along the drain connections to allow the water to flow to a sump located at the end of each line; the sumps should be equipped with automatic pumps to discharge any intercepted seepage.

In addition to the suggested subsurface drainage system, the surface drainage should be carefully planned to minimize infiltration of surface water into the slide mass. With proper planning and installation of the subsurface drains, it is our opinion that the site may be safely developed as planned. We will be pleased to assist you in developing the detailed design of the drainage system.

Respectfully submitted,

LeROY CRANDALL AND ASSOCIATES

by
LeRoy Crandall

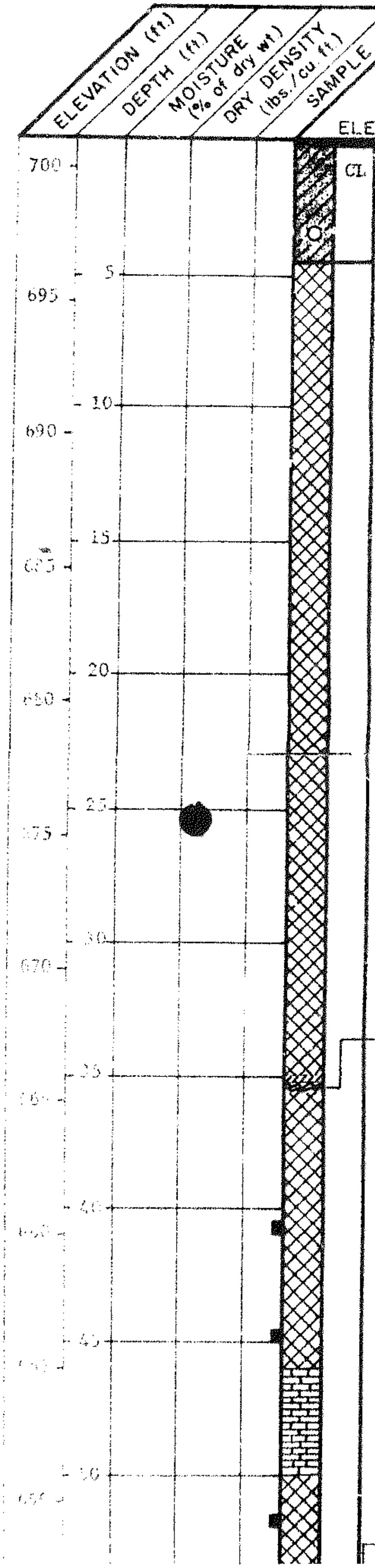
LC-[illegible]
Attachments (5)
(3 copies submitted)
cc: (2) King-Benioff-Steinmann-King

0 1 2 0 1 1 0 0 2 0 4

BORING 13*

DATE DRILLED November 1, 2, & 3, 1966

EQUIPMENT USED: 18"-Diameter Bucket



ELEVATION 701

3" ASPHALTIC PAVING

FILL - SILTY CLAY - lumps of Shale, mottled brown

5"-diameter pipe

SHALE - weathered, fractured, mottled light brown and light greyish-brown

Diatomaceous

Mottled greyish-white and light brown

Highly fractured

Seams of Silty Clay - dark grey

(SLIDE PLANE)

Lens of SILTY CLAY - brownish-grey

Massive, dark brownish-grey

Layer of LIMESTONE - (GAD AND CHOPPING EQUIPMENT USED FROM 46' TO 50')

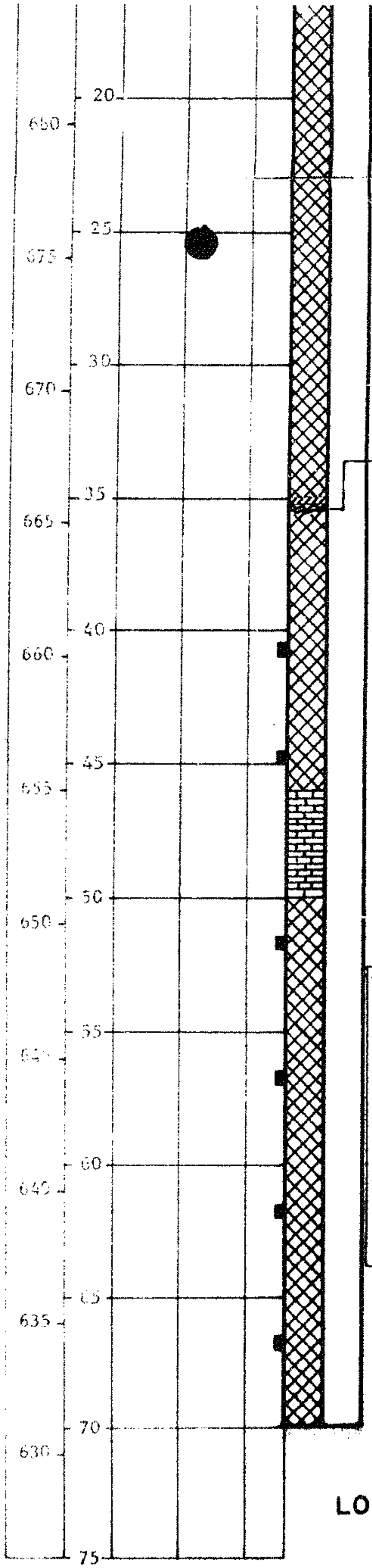
Lenses of Sandstone

NOTE: Slight water seepage encountered at -

CHKD

JOB A66131-B DATE 11-16-66 GMC OE [illegible] CHKD

1b



Seams of Si[illegible] Cl v - dark grey

Lens of SILTY CLAY - brownish-grey

Massive, dark brownish-grey

Layer of LIMESTONE - (GAD AND CHOPPING BUCKET USED FROM 46' TO 50')

Lenses of Sandstone

NOTE: Slight water seepage encountered at a depth of 46'; moderate water seepage encountered at a depth of 63'. No water in the boring at completion of drilling; 1.3' of water in the boring 20 minutes after completion of drilling.

Field Percolation Test Data: 20 minutes after completion of drilling, water was added to the hole to bring the water level to Elevation 642.3. 2½ hours later, the water level had risen to Elevation 643.5 and 13½ hours later to Elevation 650.3.

* Borings 1 through 12 presented in previous report (our Job No. A-66131).

LOG OF BORING

LEROY CRANDALL & ASSOCIATES

PLATE 2-A

BORING 14

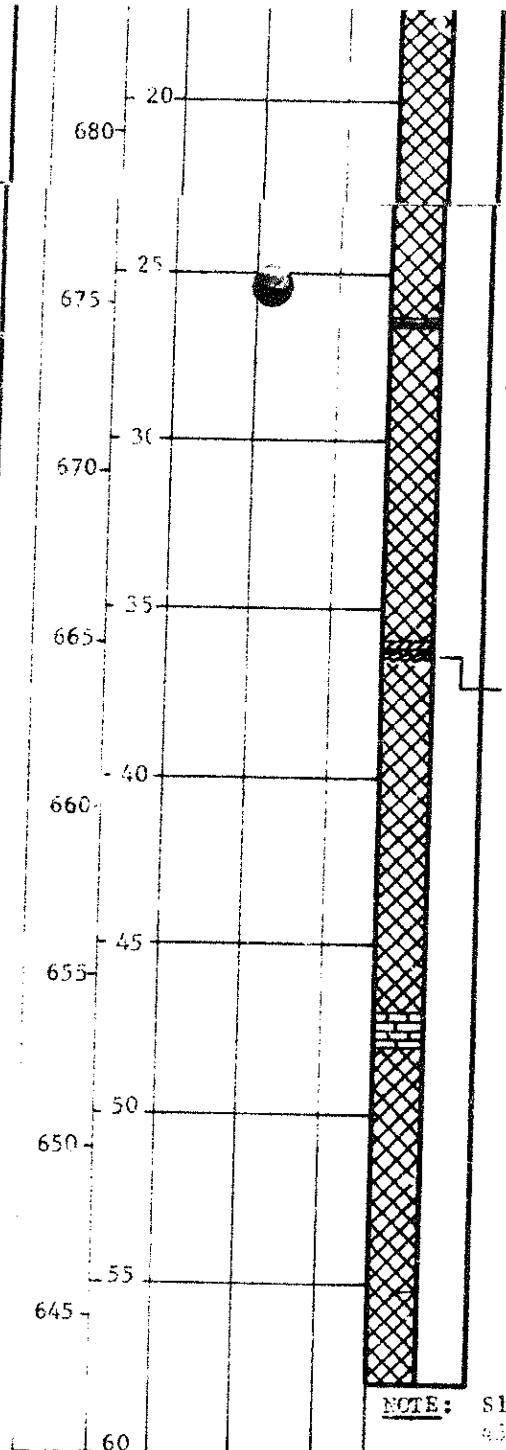
DATE DRILLED: November 5, 1966
EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION (ft)	DEPTH (ft)	MOISTURE (% of dry wt)	DRY DENSITY (lbs/cu ft)	SAMPLE

ELEVATION 701

Elevation (ft)	Depth (ft)	Description
700		3" ASPHALTIC PAVING
		CL FILL - SILTY CLAY - some Sand, mottl d brown
		SHALE - weathered, fractured, mottled light brown and light greyish-brown
695	5	Diatomaceous
		Mottled greyish-white and light brown
690	10	
685	15	
680	20	highly fractured
675	25	Lens of LIMESTONE
670	30	
665	35	Layer of SILTY CLAY (SLIDE PLANE)
660	40	Massive, dark brownish-grey
655	45	Layer of LIMESTONE (GAD USED FROM 47' TO 48')
650	50	Lenses of Sandstone

GMC DE CHKD



Highly fractured

Lens of LIMESTONE

Layer of SILTY CLAY

(SLIDE PLANE)

Massive, dark brownish-grey

Layer of LIMESTONE (GAD USED FROM 47' TO 48')

Lenses of Sandstone

NOTE: Slight water seepage encountered at a depth of 43½'; no water in the hole at the completion of drilling. No caving.

Field Percolation Test Data: After completion of drilling, water was added to the hole to bring the water level to Elevation 659.9; 22 hours later, the water level was at Elevation 660.5.

JOB AG6131-B DATE 11-16-66 R Gmc OE Jm CHKD

1b

LOG OF BORING

LEROY CRANDALL & ASSOCIATES

PLATE 2-B

0 4 2 0 1 1 0 0 2 0 6

BORING 15

DATE DRILLED: November 4, 1966

EQUIPMENT USED: 18"-Diameter Bucket

ELEVATION (ft)	DEPTH (ft)	MOISTURE (% of dry wt)	DRY DENSITY (lbs/cu ft)	SAMPLE

ELEVATION 739

3" ASPHALTIC PAVING and 9" BASE COURSE

3" ASPHALTIC PAVING

SHALE - weathered, fractured, mottled light brown and light greyish-brown

Seams of Silty Clay - dark grey

Diatoms occur

Moist

Greyish-brown

Highly fractured

Layer of SILTY CLAY - brownish-grey

(SLIDE PLANE)

Layer of LIMESTONE

Layer of Sandstone

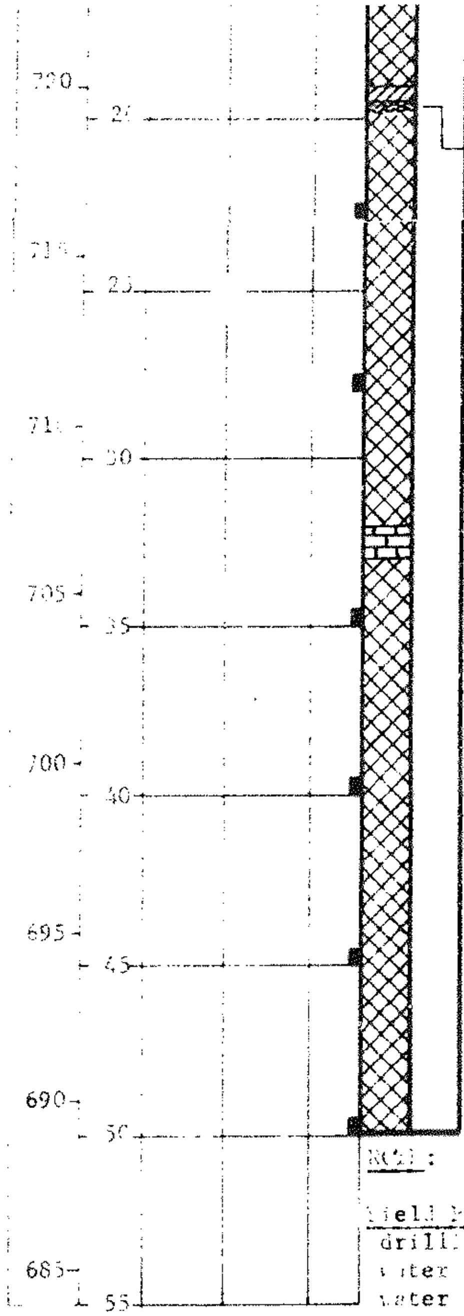
NOTE: Water not encountered. No caving.

Field Percolation Test Data: After completion of drilling, water was added to the hole to bring the water level to Elevation 698.7; 18 hours later, the

735, 730, 725, 720, 715, 710, 705, 700, 695, 690, 685

5, 10, 15, 20, 25, 30, 35, 40, 45, 50

1b



Layer of SILTY CLAY - brownish-grey

(SLIDE PLANE)

Layer of LIMESTONE

Layer of Sandstone

NOTE: Water not encountered. No caving.

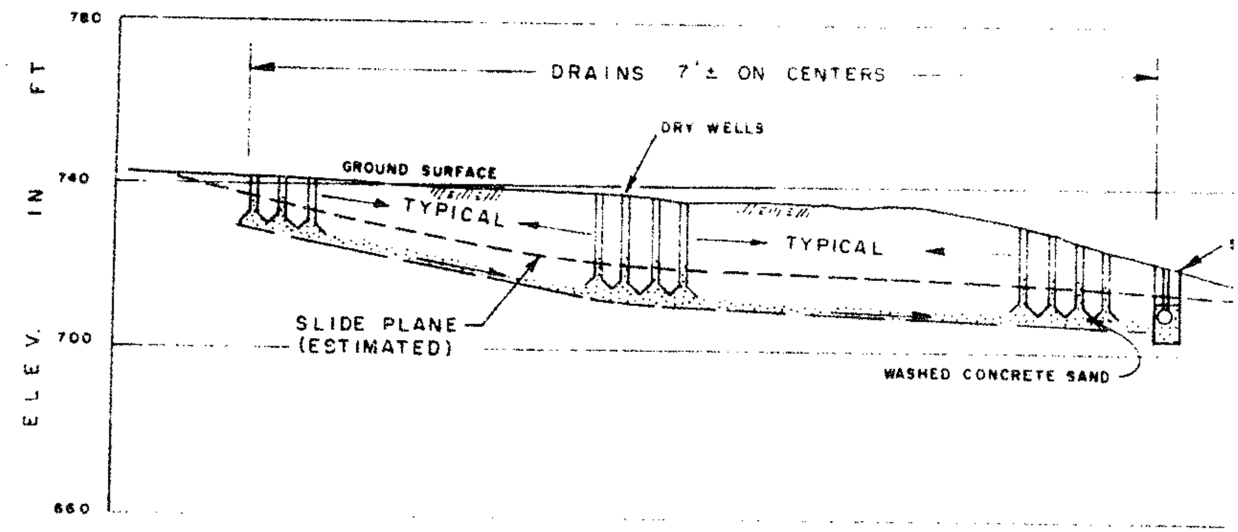
Field Percolation Test Data: After completion of drilling, water was added to the hole to bring the water level to Elevation 698.7; 18 hours later, the water level was at Elevation 697.5. Water added to bring the water level to Elevation 708.5; 26 hours later, the water level was at Elevation 703.1.

LOG OF BORING

LEROY CRANDALL & ASSOCIATES

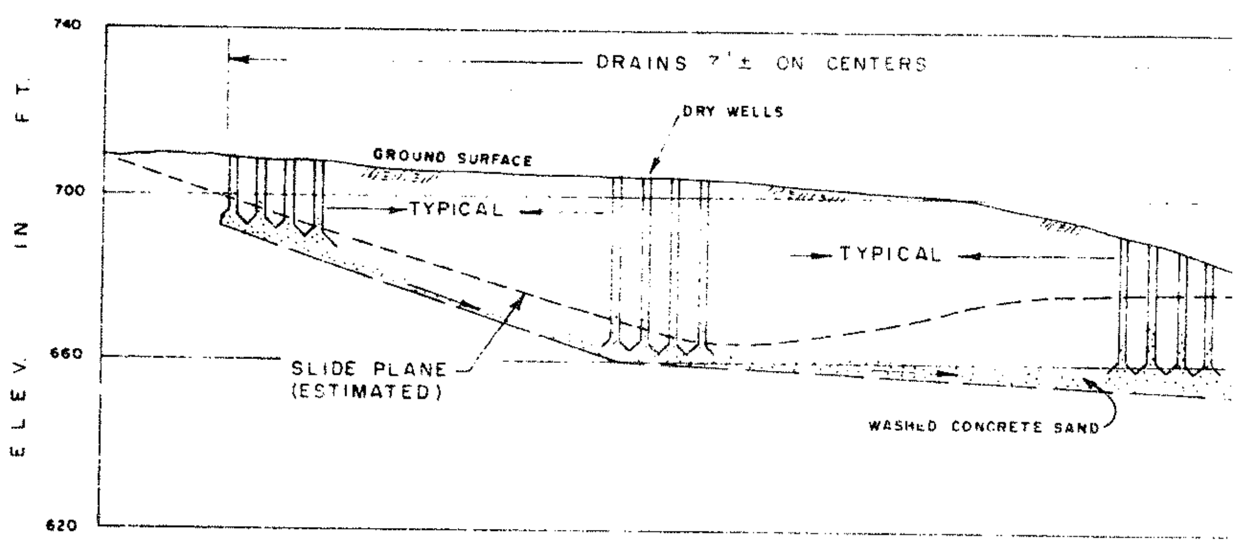
PLATE 2-C

JOB 46601.13 DATE ... OE ... 1b CHKD



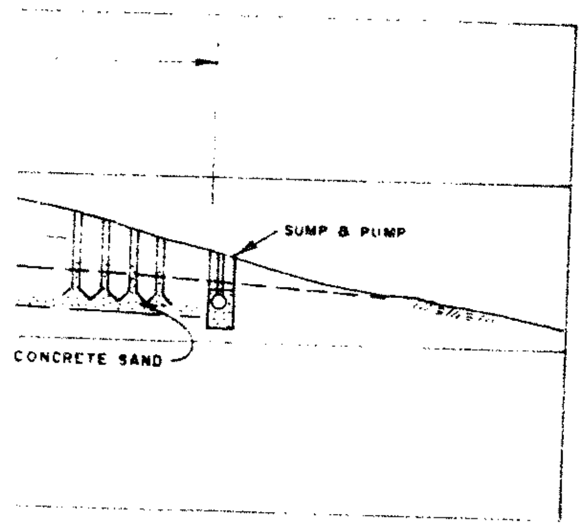
SECTION A-A

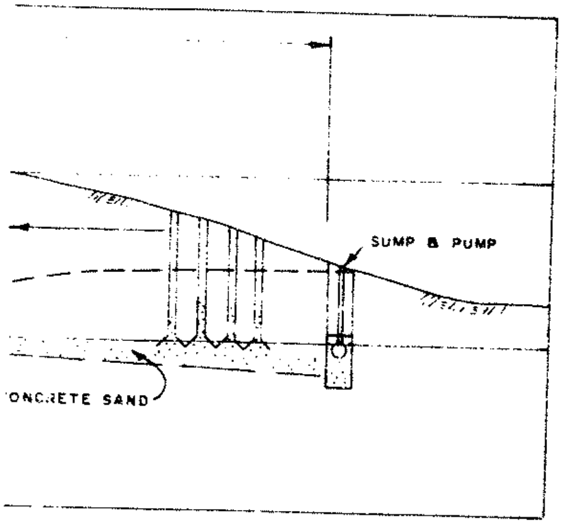
SCALE 1" = 40'

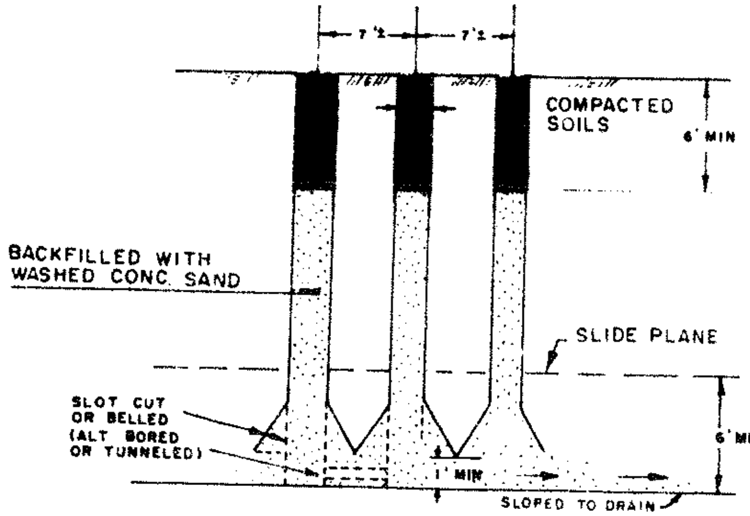


SECTION B-B

SCALE 1" = 40'





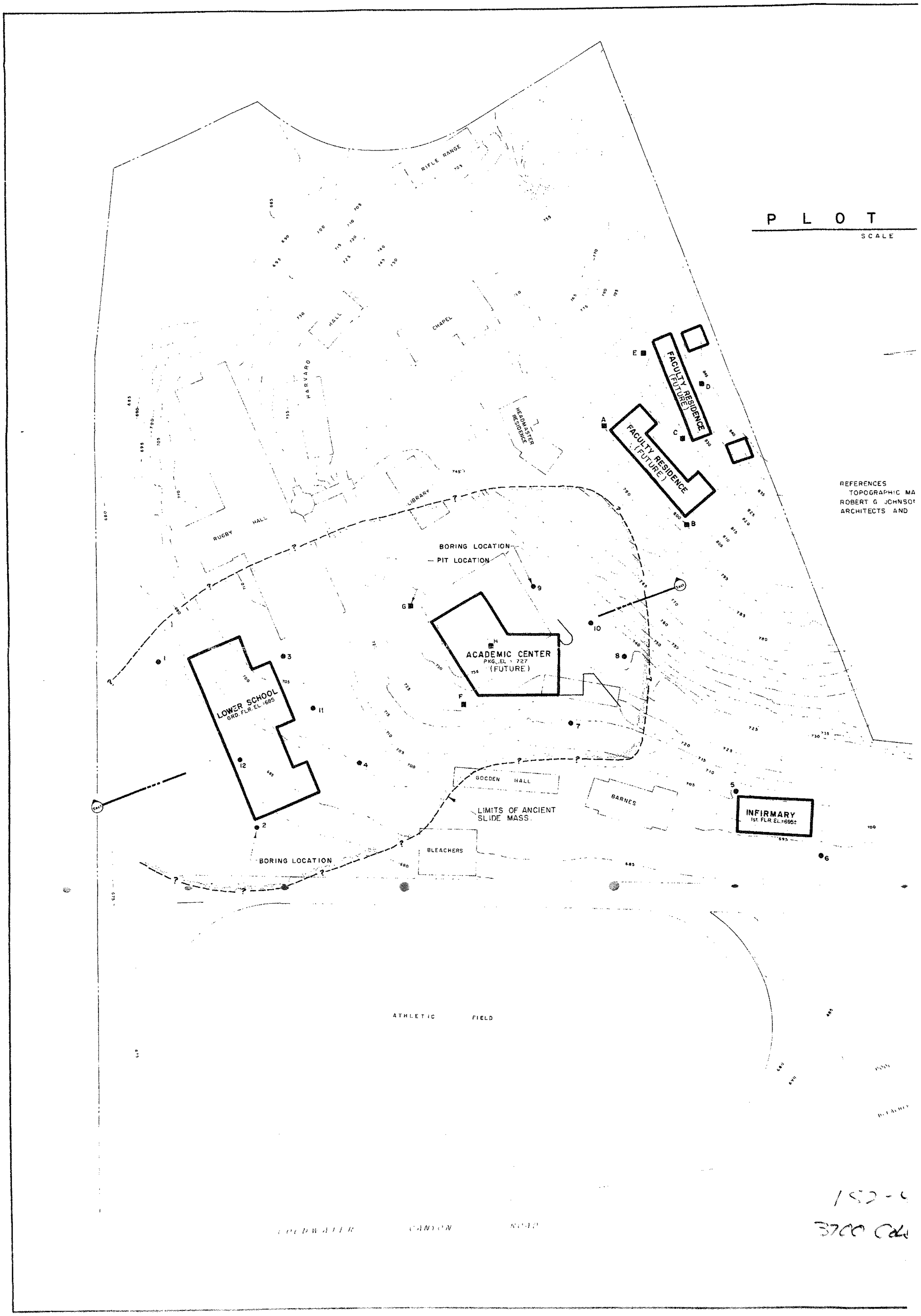


DRAIN DETAIL
NOT TO SCALE

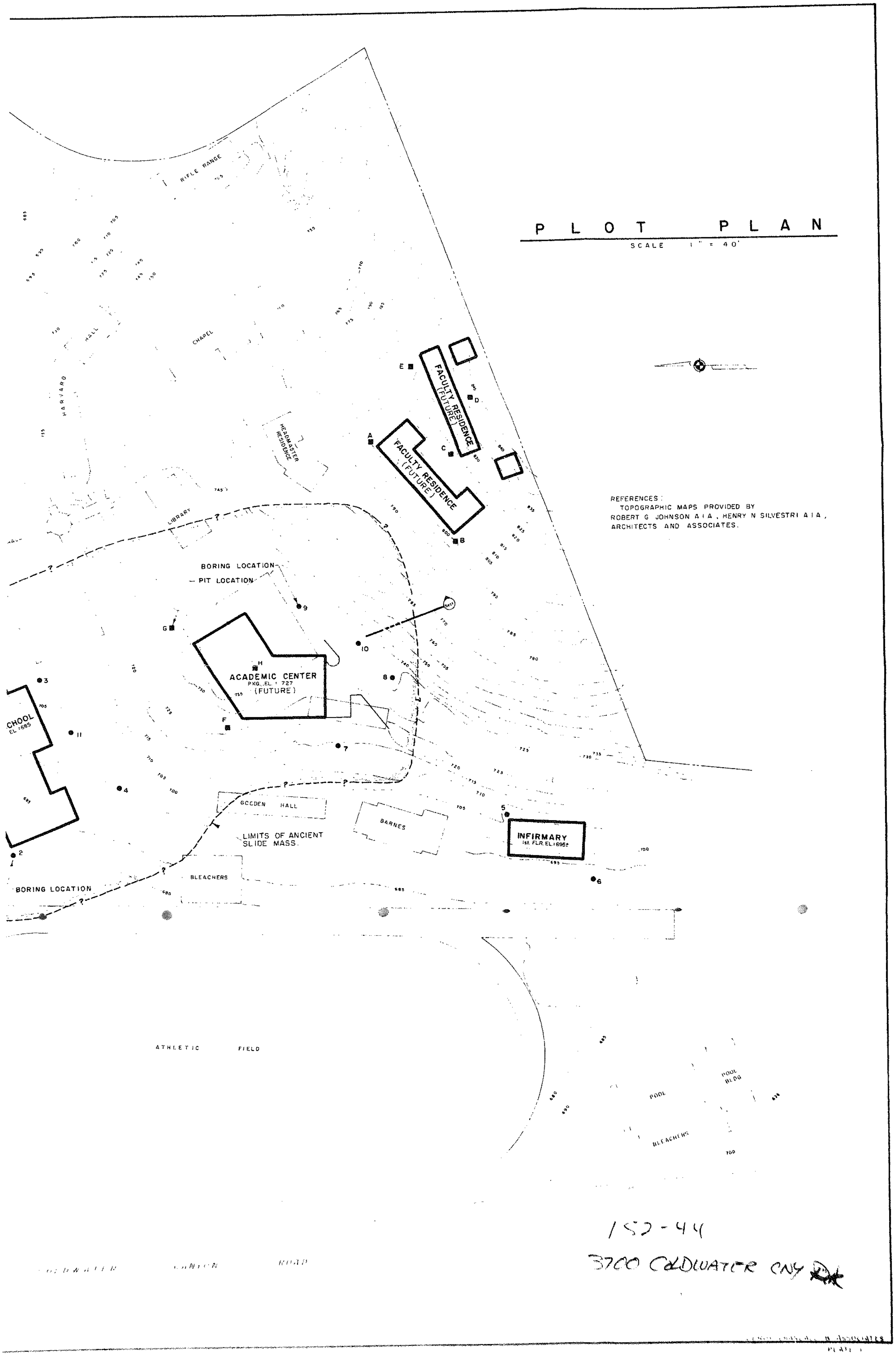
SUBSURFACE DRAINAGE SYSTEM

LEROY CRANDALL & ASSOCIATES

PLATE 3


PLOT
SCALE
REFERENCES
TOPOGRAPHIC MA
ROBERT G. JOHNSO
ARCHITECTS AND
RIFLE RANGE
HALL
CHAPEL
HARVARD
HEADMASTER RESIDENCE
FACULTY RESIDENCE (FUTURE)
FACULTY RESIDENCE (FUTURE)
LIBRARY
RUGBY HALL
BORING LOCATION
PIT LOCATION
ACADEMIC CENTER
PKG. EL. = 727
(FUTURE)
LOWER SCHOOL
GRD. FLR. EL. 685
GOODEN HALL
BARNES
INFIRMARY
1st. FLR. EL. 695±
LIMITS OF ANCIENT SLIDE MASS.
BLEACHERS
BORING LOCATION
ATHLETIC FIELD
COLDWATER CANYON ROAD
3700 Col

6/11/82


PLOT PLAN
SCALE 1" = 40'
REFERENCES:
TOPOGRAPHIC MAPS PROVIDED BY
ROBERT G. JOHNSON A.I.A., HENRY N. SILVESTRI A.I.A.,
ARCHITECTS AND ASSOCIATES.
RIFLE RANGE
CHAPEL
HARVARD HALL
HEADMASTER RESIDENCE
LIBRARY
FACULTY RESIDENCE (FUTURE)
FACULTY RESIDENCE (FUTURE)
BORING LOCATION
PIT LOCATION
ACADEMIC CENTER
PKG. EL. ± 727
(FUTURE)
SCHOOL
EL 685
GOLDEN HALL
BARNES
INFIRMARY
1st. FLR. EL. ± 695±
LIMITS OF ANCIENT
SLIDE MASS.
BLEACHERS
BORING LOCATION
ATHLETIC FIELD
POOL
POOL BLDG
BLEACHERS
ROAD
152-44
3700 COLDWATER CNY DR

Appendix II

Geology and Soils Consultants, Inc., report dated January 29, 1973

Geology & Soils Consultants Inc.

JB17866

GEOLOGIC ENGINEERING INVESTIGATION

Proposed Library and Field House
3700 Coldwater Canyon Avenue
Los Angeles, California

for

HARVARD SCHOOL

GSC 614 January 29, 1973

G.S. Kovacs
Civil Engineering
John W. Byer
Engineering Geology

ENGINEERING GEOLOGY ▪ SOILS & FOUNDATION ENGINEERING
12526 VENTURA BLVD STUDIO CITY CALIFORNIA 91604 (213)980-0825 (L.A.)877-2757

January 29, 1973
GSC 614

GEOLOGIC ENGINEERING INVESTIGATION
PROPOSED LIBRARY AND FIELD HOUSE
3700 COLDWATER CANYON AVENUE
LOS ANGELES, CALIFORNIA

INTRODUCTION

This report presents the results of our foundation investigation performed on the subject property. The purpose of this investigation was to determine the nature of the soils underlying the site, to ascertain their engineering properties, and to provide recommendations for foundations, lateral design, and slabs on grade.

This investigation included reviewing pertinent engineering geology and soils reports (see Reference List), drilling exploratory test borings and pits, obtaining representative soil samples, and the preparation of this report. The exploratory boring locations are shown on the enclosed Plot Plan and Geologic Map. Also attached are the laboratory results, a geologic section and stability analysis.

STRUCTURAL CONSIDERATIONS

Information concerning the proposed development was furnished by Mr. Robert Johnson of Johnson & Silvestri, Architects. The proposed structures are a two-story concrete, tilt-up panel Field House and a two-story, woodframe Library. The Field House is a heavy masonry structure with column loads in the range of 550 kips. The Library will be a conventional frame structure with a wall load not expected to exceed three kips per lineal foot. The column loads are not expected to exceed 50 kips. The Library site will require some grading as it will be on a cut and fill area. The Field House will require extensive grading since its location will require excavation.

SITE CONDITIONS

The property is located at 3700 Coldwater Canyon Avenue in the City of Los Angeles, California. For the Library structure: the surface at present is covered by several trees, shrubs, lawn and the existing Library. The site slopes from the south to the north and drainage is by sheetflow down the slopes to the north and down the street to the west.

ENGINEERING GEOLOGY ▪ SOILS & FOUNDATION ENGINEERING
12526 VENTURA BLVD., STUDIO CITY, CALIFORNIA 91604 (213) 980-0825 (L.A.) 877-2757

The adjacent structures are the Academic Center to the west, a plaza to the east, a roadway to the south, and a rose garden to the north. There is a buried foundation to the west of the existing Library in the parking lot. There are also several underground utilities in and about the existing Library structure (see Plot Plan and Geologic Map).

For the Field House: the surface at present has a parking lot on approximately 15 feet of fill, several trees, shrubs, and a retaining wall down at the track level. The parking area is approximately 15 feet above the track level and all drainage from the parking area is by sheetflow to the storm drains which pipe the water down to the storm drains at the track level. There are also several electrical conduits, sewer lines, and storm drains running through the site (see Plot Plan and Geologic Map).

The site was explored on the 11th, 12, and 15th of January, 1973, by digging two test pits and by drilling seven exploratory test borings with a truck-mounted, rotary drilling machine. The borings varied in depth from 20 to 51 feet, and the test pits were six to eight feet deep. The boring and test pit locations are shown on the Plot Plan and Geologic Map and the soils encountered are logged on Plates A-1 through A-19.

ENGINEERING GEOLOGY ■ SOILS & FOUNDATION ENGINEERING
12526 VENTURA BLVD., STUDIO CITY, CALIFORNIA 91604 (213)980-0825 (L.A.)877-2757

The soils encountered will be broken into those found at the Field House site and those found at the Library.

Library Building

The soil profile for the Library was consistent throughout the site. The profile is a sloping profile going from a cut to a fill condition across the site in a south to north direction. In Borings 6, 7, and 9-A, Modelo Formation was encountered below the asphalt and crusher run base to the completed depth of these borings (46 feet for Boring 6, 35 feet for Boring 7, and 30 feet in Boring 9-A). Down the slope in Test Pits A, B, and GA, a typical fill soil, natural soils, then Modelo Formation profile was encountered with the Modelo continuing to completion of these test pits (6 feet for Test Pit A, 8 feet for Test Pit B, and 4 feet for Test Pit GA).

Field House

The soil profile for the Field House was consistent once the 15 feet of fill for the parking lot was penetrated. The Field House, from the data collected in the borings, appears to be situated across an alluviated canyon. The soils in Borings 1 through 4, below the parking lot fill, were alternating layers of sands, silts, and clayey sands which are indicative of alluvium.

Bedrock was encountered in all borings. The depth of bedrock ranged from 10 feet in Boring 5, and 15 feet in Boring 6A, which were on a flank of a south-trending ridge; 24 to 28 feet in Borings 1 and 2, down on the track; and 36 to 37 feet in Borings 3 and 4, up on the parking lot fill.

GROUND WATER

Library

Ground water was not encountered in any of the borings or test pits dug for the Library Building. As such, no problems are to be anticipated during the construction of the structure.

Field House

Ground water was encountered in Borings 1 through 4. The water was flowing in at the bedrock-alluvium contact. The water had filled each hole to this elevation by the time the hole was ten feet into the bedrock. Ground water concentrates on the bedrock due to its impermeable nature.

GEOLOGIC STRUCTURE

Geologic conditions affecting the proposed Library were ascertained from downhole inspection in Borings 6 and 7. Geologic data presented in the Crandall reports were reviewed and incorporated into the regional geologic picture.

As described in the referenced geologic reports, a pre-historic landslide mass was found to underlie a majority of the existing Harvard School. The slide debris ranges in thickness from 25 to 40 feet. The slide can be classified as a block glide which failed along northwest-dipping bedding planes within the Miocene Modelo Formation. Subsequent to the failure, the surrounding terrain has been modified by erosion, grading, and more significantly, alluvial deposition along the toe of the slide.

Geologic data gathered from Borings 6 and 7 were incorporated with the existing data which were used to draw Geologic Section A. This section indicates the slide to be larger than originally mapped. Section A was then used for a stability analysis. The analysis indicates that the pre-historic landslide has reached equilibrium and possesses a safety factor in excess of 1.5.

ENGINEERING GEOLOGY ▪ SOILS & FOUNDATION ENGINEERING
12526 VENTURA BLVD., STUDIO CITY, CALIFORNIA 91604 (213)980-0825 (L.A.)877-2757

Downhole inspection indicated the slide debris to consist of diatomaceous and silty shale of the Modelo Formation. The slide debris was found to be mostly tight with localized areas containing open fractures and soil filled fractures. The slide plane in Boring 6 was found at a depth of 24.5 feet. The attitude of the slide plane in this area is strike N40E, with a 14 degree dip to the northwest. The slide plane consists of wet silty clay that is firm and no active seepage was occurring at the slide plane.

Immediately below the slide plane, the bedrock encountered consisted of diatomaceous and silty shale with interbedded, fine-grained sandstone. The bedrock is very firm, tight, with no open fractures or joints. Bedding within the Modelo Formation was found to strike N50E and dip 12 to 15 degrees to the northwest.

In Boring 7, two slide planes were recognized. The first plane was found at a depth of four feet in which the rock above was very weathered, crushed, and broken. The slide debris below four feet was tight and resembled in-place bedrock except for the presence of open fractures. The fractures were open as much as 1/4 inch to a depth of approximately 20 feet. At this depth, a second slide plane was recognized striking N55E and dipping 14 degrees to the

ENGINEERING GEOLOGY ▪ SOILS & FOUNDATION ENGINEERING
12526 VENTURA BLVD., STUDIO CITY, CALIFORNIA 91604 (213)980-0825 (L.A.)877-2757

northwest. Below the slide plane, the bedrock was found to be very firm, tight, and closed, similar to the rock found in Boring 6.

Based upon the geologic data submitted to date and the additional information gained from Borings 6 and 7, the school site is underlain by a pre-historic landslide that is now grossly stable and should not affect future construction. Reconnaissance visual inspection of buildings currently constructed on the slide area reveals the presence of no adverse cracking or separations which would indicate that the slide is creeping.

RECOMMENDATIONS - LIBRARY STRUCTURE

Foundations

Conventional spread footings may be used to support the structure provided they are founded on the Modelo Formation (within the prehistoric landslide) which is found nine inches below existing grade on the south side and eight feet below existing grade on the north side. Wall footings may be designed for a bearing value of 3,000 pounds per square foot and should be a minimum of 12 inches in width and two feet into the Modelo Formation. Column footings may be designed for a bearing value of 5,000 pounds per square foot and should be a minimum of two feet in width and two feet into the Modelo Formation. The structure should not be supported on isolated piers. The wall footings and column piers should all be tied together.

Lateral Design

The bearing values indicated above are for the total of dead and frequently applied live loads and may be increased by one third for short duration loading, which includes the effects of wind or seismic forces. Resistance to lateral loading may be provided by friction acting at the base of foundations and by passive earth pressure. An allowable coefficient of friction of 0.4 may be used with the dead load forces. Passive earth pressure may be computed as an equivalent fluid having a density of 400 pounds per cubic foot with a maximum earth pressure of 5,000 pounds per square foot for the Modelo Formation. When combining passive and friction for lateral resistance, the passive component should be reduced by one third. For design of isolated poles, the allowable passive earth pressure may be increased by 100 percent.

Foundation Settlement

Settlement of the foundation system is expected to occur on initial application of loading. The maximum settlement is expected to be 0.5 inches and occur below the columns. Differential settlement is not expected to exceed 1/4 inch.

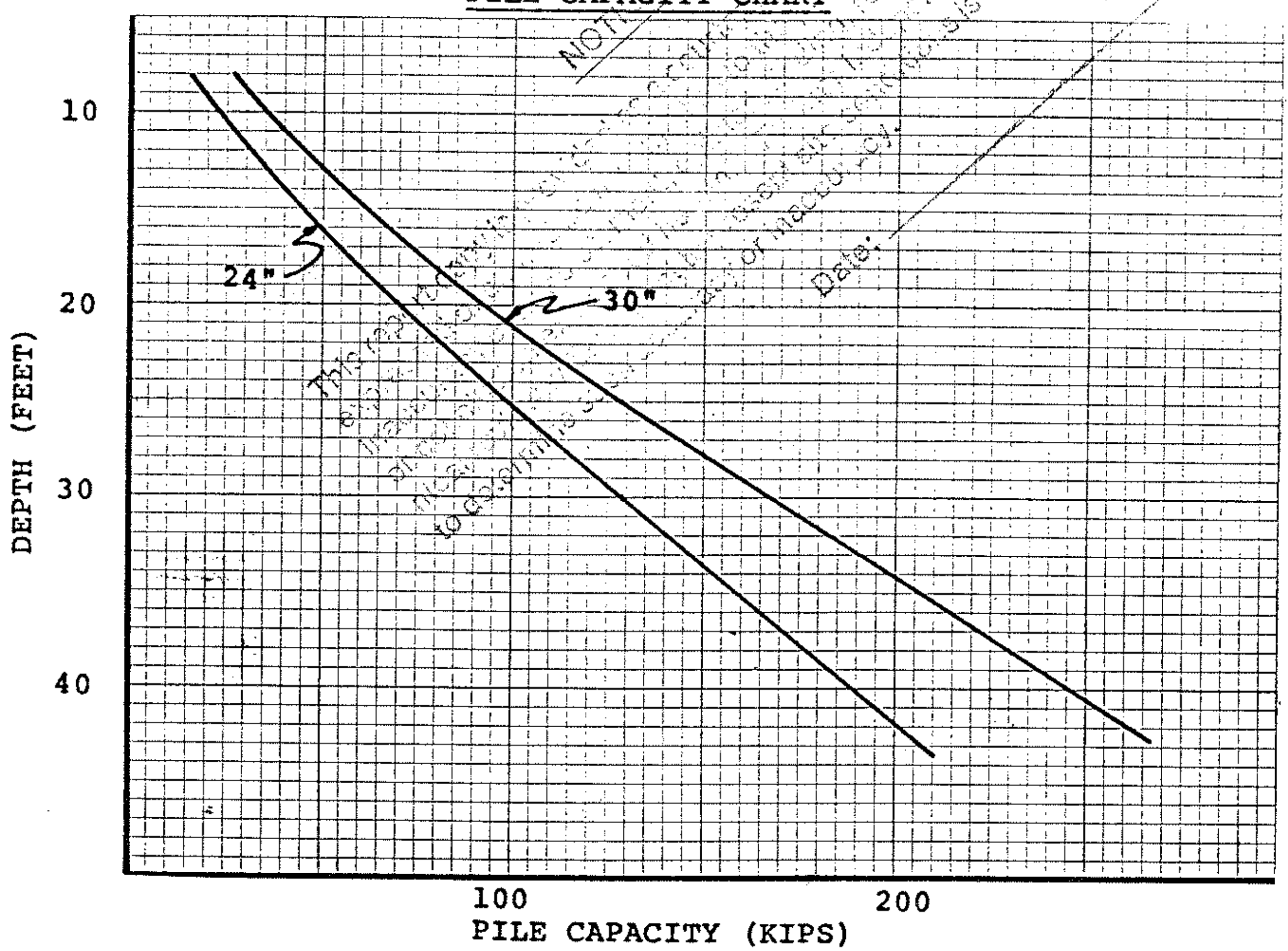
RECOMMENDATIONS - FIELD HOUSE

Foundations

The most economical foundation system which can satisfy the structural consideration is drilled, cast-in-place friction piles. While some difficulty is expected due to the ground water condition, the cost should prove less when considering driven displacement piles of either timber or concrete. The latter cause considerable noise and ground motion which is objectionable in this area.

The piles may be sized per the following Pile Capacity Chart.

PILE CAPACITY CHART



NOTES:
1. Capacity for Drilled, Cast-in-Place, Pile of Diameter Shown. Other Diameter Piles Have Capacity in Direct Proportion to Diameter.
2. For Uplift Capacity, Multiply by One-Half.
3. All Piles to be Placed in "Dry" Holes.
4. No Pile Excavation to Remain Open Overnight.
5. Space Piles at 2-1/2 Pile Diameters, Center to Center.
6. Place No More Than One Pile Per Group Per Day.
7. For Lateral Capacity, See Text.
8. Minimum Recommended Depth, Ten Feet Into Bedrock.

Placement

Placement of the concrete will require a coordinated effort by the foundation contractor. Using 24-inch piles as an example, the following steps should be taken: Piles should be excavated using a 30-inch bucket down to the bedrock. Once in the bedrock, a 28-inch steel casing should be placed in the hole and driven into the bedrock so as to effect a seal to keep the perched ground water from entering the excavation. A 24-inch bucket should then be used to excavate the pile to the required depth. Any water that has collected in the excavated shaft should be removed prior to placing any concrete. The concrete should be placed in the shaft keeping the concrete level five feet above the tip of the steel casing as it is being extracted.

Inspection by the foundation engineer will be required to determine the required penetration and allow continuous placement of concrete. No pile excavation should be allowed to stand open overnight. Only one pile per cluster may be placed in any one day.

Group Action

Where multiple piles are required and they can be placed at 2-1/2 pile diameter spacing center to center, no reduction in capacity is necessary.

ENGINEERING GEOLOGY ■ SOILS & FOUNDATION ENGINEERING
12526 VENTURA BLVD., STUDIO CITY, CALIFORNIA 91604 (213)980-0825 (L.A.)877-2757

Settlement

Foundation settlement is expected during the course of construction. Differential settlement is not expected to occur and the maximum settlement of the piles is not expected to exceed 3/4 inch.

Lateral Design

Resistance to lateral loads may be calculated from the following table for fixed and free head piles 24 inches in diameter.

	Fixed Head	Free Head
Lateral Capacity (kips)	32	13
Max. Neg. Moment (kip-in)	66P*	--
Inflection Point (feet)	7	--
Depth to Max. Moment (feet)	13	8
Max. Pos. Moment (kip-in)	19P*	58P*
Depth to Zero Moment (feet)	28	24

*P is in kips

Retaining Walls

The Field House is going to be cut into the existing parking lot fill on the west and south sides. Also, it is going to be cut into the side of the south-trending ridge on the east side. These cuts will most likely cause the walls of the building to act as retaining walls. Retaining walls can be designed for an equivalent fluid pressure of 30 pounds per square foot per foot of depth. The retaining walls shall have a sufficient number of weepholes and these weepholes shall be covered with a sufficient amount of gravel to allow the weepholes to function properly. Footings may be designed for 3,000 pounds per square foot and lateral resistance may be provided by friction and passive earth pressure as indicated in the "Lateral Design" section of the recommendations for the Field House.

Excavations

There will be excavations required for the Field House ranging in depth from 12 feet to 16 feet. These excavations should be stabilized within 30 days of initial excavating. Water should not be allowed to pond on top of the excavation nor to flow towards it.

Temporary excavations can be made if the upper five feet is cut at a slope of 1:1. The remaining 11 feet may be cut vertically.

Floor Slabs and Paving

Prior to placing slabs-on-grade or paving, the existing grade should be scarified to a depth of six inches, moistened as required to obtain optimum moisture content and recompacted to 90 percent of the maximum dry density, as determined by ASTM D 1557-70. The following pavement sections are recommended:

Service	Pavement Thickness (Inches)	Base Course (Inches)
Light Passenger Cars	2	
Trucks, Moderate Truck Driveways (Storage, etc.)	3	4

Base course should be crusher run base (CRB) or decomposed granite.

Floor slabs should be reinforced with a minimum of 6x6-10x10 welded wire fabric. Slabs which will be provided with a floor covering should be protected by a polyethelene plastic vapor barrier. The barrier should be covered with a thin layer of sand, about one inch, to prevent punctures and aid in the concrete cure.

Inspection

It is recommended that all footings be inspected by our representative prior to placing concrete or steel. Any fill which is placed should be inspected, tested, and certified.

Respectfully submitted,

GARY C. MASTERMAN

G. S. KOVACS
R.C.E. 13503

JOHN W. BYER
E.G. 883

GCM:GSK:JWB:mm

Enc: Plot Plan and Geologic Map
Section A-A
Plates A-1 thru A-19
Plates B-1 thru B-4
Plates C-1 thru C-7
Plate D
Reference List

xc: (2) Addressee
(1) King, Benioff, Steinmann, & King
(4) Johnson & Silvestri, AIA

BORING LOG NUMBER 1

Drilling Date 1/12/73 **Elevation** 680.0

Project Harvard - GSC 614

Sample Depth ft.	Blows per ft.	Moisture Content %	Dry Unit Weight p.c.f.	Depth in feet.	Graphic Log	Surface: Dirt Track **Description**
				2		FILL, Clayey Sand, brown, moist, dense, with diatomaceous Shale 1/2"
5.0	3	23.0	90.0			
				6	ML ML	Clayey Silt, tan, moist, firm, with Shale fragments 1/2"
					—	— — grades to red-brown
10.0	4	21.0	88.0	10		
					SM	Silty Sand, red-brown, slightly moist, dense, medium with Slate fragments 1/2"
				14		
15.0	1	35.6	80.3			Clayey Silt, tan, very moist, soft
						grades to red-brown
				18	ML	
20.0	2	29.0	93.0			
				22		water in hole at 22'
23.0	** 17/6"	75.0	54.5			
						BEDROCK, Modelo Shale, dark brown, dense
				26		
				30		
31.0	** 46/10	54.7	65.5			
				34		End boring at 33.0 feet; No Caving; Water level at 23.0 feet.
						1500# Kelly **750# Inner Kelly 12" Drop

BORING LOG NUMBER 2

Drilling Date 1/12/73 **Elevation** 680.3

Project Harvard - GSC 614

Sample Depth ft.	Blows per ft.	Misture Content %	Dry Unit Weight p.c.f.	Depth in feet	Graphic Log	Surface: Dirt Track **Description**
						FILL, Clayey Sand, red-brown, moist, dense
5.0	6	22.7	98.0	2 6	ML	Clayey Silt, tan, moist, stiff with Shale fragments 1/2", grades drier
10.0	3	16.7	90.6	10	SM	Silty Sand, brown, moist, dense, medium with some Clay binder grades clayier
15.0	5	19.0	98.7	14		Sandy Clay, tan, moist, dense, medium
20.0	3	15.9	102.5	18 22	CL	1/2" angular Shale fragments red-brown Sand stringer
25.0	5**	30.8	92.3	26	ML	Clayey Silt, tan, moist, stiff caving at 28.5 feet water running in at 29 feet
30.0	6**	46.0	75.9	30		BEDROCK, Modelo Shale, tan
35.0	12**	55.0	68.4	35		
40.0	20**	47.0	70.0	40		grades denser and has a green color
				45 50		
						End at 51 feet; Water at 29 feet; Caving at 30 feet

BORING LOG NUMBER 3

Drilling Date 1/12/73 **Elevation** 693.7

Project Harvard - GSC 614

Sample Depth ft.	Blows per ft.	Moisture Content %	Dry Unit Weight p.c.f.	Depth in feet	Graphic Log	Description
						Surface: AC Parking Lot, 4" AC
				2		FILL, Sandy Gravel, brown, moist, dense with subrounded boulders, Gravel 1½" maximum, bricks and concrete present
5.0	5	14.1	115.0			Silt, tan, slightly moist, loose, with Shale fragments to 1½"
				6		Clayey Sand, dark brown, moist, dense, medium, with Shale fragments plaster and bricks present
10.0	3	24.2	84.5	10		Silty Sand, yellow-brown, moist, dense, medium with Shale fragments and Clay binder
				14		
15.0	2	13.7	89.2			
				18		grades clayier
20.0	2	16.4	88.0			Clayey Gravel, brown, moist, dense, 2" maximum
				22		cobble-size Shale fragments present
25.0	** 6	20.3	92.5		GC	grades sandy
				26		
30.0	** 7	24.1	97.0	30		
				34	CL	Clayey Silt, tan, moist, soft, has some Gravel present, hard to drill; cemented limestone boulders, grades very moist, rapid caving
35.0	** 4	34.4	86.3			
				38		BEDROCK, highly weathered
40.0	** 4	58.4	64.3			
				42		End boring at 41.5 feet; Water and Caving at 39 feet; unable to advance

BORING LOG NUMBER 4

Drilling Date 1/12/73 **Elevation** 694.0

Project Harvard - GSC 614

Sample Depth ft.	Blows per ft.	Moisture Content %	Dry Unit Weight p.c.f.	Depth in feet	Graphic Log	Description Surface: Dirt Area Just Off Parking Area
						FILL, Clayey Sand, red-brown, moist, dense, medium
				2		Clayey Silt, dark brown, moist, soft with Shale fragments 1½" maximum
5.0	2	22.5	87.0			
				6		
						Clayey Sand, red-brown, moist, dense, medium to coarse with Shale fragments 1½"
10.0	2	14.7	92.0	10		
				14		
17.5	4	14.6	104.5			Clayey Sand, dark brown, moist, dense, medium
				18		
						Gravel present, Shale fragments 1½" maximum
				22		
24.0	** 6	21.4	91.5			grades sandier
				26		
					CL	Clayey Silt, dark brown, moist, stiff, with angular Shale fragments 1½"
				30	GC	Clayey Sand, tan, moist, dense, medium with some angular Shale fragments 1½" -grades very sandy & to a dark red brown
31.5	** 5	27.9	90.0			
				34	SM	Silty Sand, red-brown, moist, dense, medium, with some Clay binder
36.0	** 4	33.2	84.8		CL	Clayey Silt, gray, very moist, soft Water at 36 feet
				38		BEDROCK, Modelo Formation, weathered
				40		good, hard, Shale
				45		End at 45.0 feet; Water at 38.0 feet at Conclusion of Drilling; no Caving

BORING LOG NUMBER 5

Drilling Date 1/15/73 **Elevation** 702.0

Project Harvard - GSC 614

Sample Depth ft.	Blows per ft.	Moisture Content %	Dry Unit Weight p.c.f.	Depth in feet	Graphic Log	Description
						Surface: Dirt Road
				2		FILL, Clayey Silt, brown, moist, firm, with Sand, concrete, bricks, Shale, cobble-sized subrounded fragments cobbles of very hard Shale present
5.0	2	24.6	66.5			Silt, Tan, slightly moist, loose
				6	ML	
10.0	4	38.8	70.0	10		BEDROCK, Modelo Formation, tan, dense
				14		
15.0	7	60.0	62.8			
				18		
20.0	8	60.6	59.8			
				22		
						End at 21.0 feet; no Water; no Caving

BORING LOG NUMBER 6

Drilling Date 1/15/73 **Elevation** 740.0

Project Harvard - GSC 614

Sample Depth ft.	Blows per ft.	Moisture Content %	Dry Unit Weight p.c.f.	Depth in feet	Graphic Log	Description
						Surface: AC 4", CRB 6"
				2		FILL, Clayey Silt, tan, moist, firm, with bits of Clay and concrete, and Shale
5.0	4	35.2	85.4			BEDROCK, Modelo Formation, weathered
				6		well bedded
				10		
13.0	7	65.0	57.0	14		
				18		
				22		
24.0	** 18					slide plane at 25 feet
				26		
30.0	16	45.0	77.3	30		
				34		
35.0	** 16	47.0	73.5			fine Sand lense
				38		
40.0	23	78.0	54.5	40		grades moist

(continues)

BORING LOG NUMBER 6 (continued)

Drilling Date 1/15/73 **Elevation** 740.0

Project Harvard - GSC 614

Sample Depth ft.	Blows per ft.	Moisture Content %	Dry Unit Weight p.c.f.	Depth in feet.	Graphic Log	Description
				42		Modelo Formation continues
				46		End at 46.0 feet; no Water; no Caving <u>ATTITUDES</u> Bedding in Slide: N40E; 38NW @ 8' N60E; 25N @ 9' N65E; 30N @ 9.5' N60E; 26N @ 11' N60E; 27N @ 15' N25E; 46W @ 20' N50E; 17N @ 25' Slide Plane 24.5 Feet: N40E; 14NW Bedding Below Slide: N50E; 14N @ 30' N45E; 12N @ 35' N50E; 15N @ 40'

BORING LOG NUMBER 7

Drilling Date 1/15/73 **Elevation** 745.0

Project Harvard - GSC 614

Sample Depth ft.	Blows per ft.	Moisture Content %	Dry Unit Weight p.c.f.	Depth in feet	Graphic Log	Description Surface: AC 4", CRB 4"
						BEDROCK at surface
				2		thinly bedded **Slide Plane at 4 feet**
5.0	8	87.0	48.4	6		
10.0	6	46.3	72.5	10		grades moist
				14		
15.0	8	46.6	71.0	18		
20.0	4	76.0	56.6			limestone layer slide plane at 19 feet grades drier
				22		
25.0	**22	43.8	75.5	26		
30.0	**16	42.0	78.5	30		
				34		Siliceous layer, very hard
				38		End at 35.0 feet; no Water; no Caving See Plate A-9 for Attitudes.

BORING LOG NUMBER 7 (continued)

Drilling Date 1/15/73 **Elevation** 745.0

Project Harvard - GSC 614

Sample Depth ft.	Blows per ft.	Moisture Content %	Dry Unit Weight p.c.f.	Depth in feet.	Graphic Log	Description
						<u>ATTITUDES</u> **In Slide Plane:** N55E; 14N @ 4' Bedding: N50E; 12N @ 5' N75W; 19N @ 10' N55E; 12N @ 15' N45E; 16N @ 18' Slide Plane (?): N55E; 14N @ 20' Bedding: N60E; 15N; @ 22' N60E; 18N @ 31'

BORING LOG NUMBER A, B

Drilling Date 1/11/73 **Elevation** 742.2, 736.2

Project Harvard - GSC 614

Sample Depth ft.	Blows per ft.	Moisture Content %	Dry Unit Weight p.c.f.	Depth in feet	Graphic Log	Description
						A (742.2)
				0		SOIL, Clayey Sand, brown, moist, dense
				1		BEDROCK, Modelo Formation, weathered
2.0	-	40.6	71.0	2		
				3		
				4		
				5		
6.0	-	53.0	67.0	6		End pit at 6.0 feet; no Water; no Caving
						B (736.2)
				0		FILL, Clayey Sand, brown, moist, dense
				1		Clayey Silt, dark brown, moist, stiff, with Diatomaceous Shale fragments
2.0	-	23.4	86.0	2		Silty Clay, dark brown, moist, hard
				3		
				4		
				5		BEDROCK, Modelo Formation, weathered
				6		
				7		
				8		End pit at 8.0 feet; no Water; no Caving

BORING LOG NUMBER 2A

(See Ref. List No. 2)

Drilling Date 5/10/66 **Elevation** 677.0

Project Harvard - GSC 614

Sample Depth ft.	Blows per ft.	Moisture Content %	Dry Unit Weight p.c.f.	Depth in feet.	Graphic Log	Description
				0	SM	Silty Sand, well graded, oiled
					CL	Silty Clay, dark gray
						some Shale Gravel
5.0	-	33.6	71.0	5		Shale, weathered, fractured, mottled light brown and light grayish brown
10.0	-	31.2	87.0	10		diatomaceous
						mottled grayish white
15.0	-	75.1	53.0	15		
						End at 15 feet; no Water; no Caving

BORING LOG NUMBER 4A

(See Ref. List No. 2)

Drilling Date 5/11/66 **Elevation** 700.0

Project Harvard - GSC 614

Sample Depth ft.	Blows per ft.	Moisture Content %	Dry Unit Weight p.c.f.	Depth in feet	Graphic Log	Description
						3" Asphaltic Paving
				0	CL	FILL, Silty Clay, lumps of Shale, pieces of wood and concrete, mottled brown
						Shale, weathered, fractured, mottled light brown and light grayish brown
5.0	-	30.5	82.0	5		
						diatomaceous
10.0	-	42.2	61.0	10		mottled grayish-white
15.0	-	36.0	71.0	15		
						(Slide Plane) lenses of Sandstone seams of Silty Clay, dark gray
20.0	-	57.0	59.0	20		
25.0	-	49.0	70.0	25		
30.0	-	83.5	50.0	30		layer of limestone
35.0	-	60.3	59.0	35		
						massive, dark brownish gray
40.0	-	36.1	80.0	40		
				45		
						End at 45 feet; no Water; no Caving

BORING LOG NUMBER 6A

(See Ref. List No. 2)

Drilling Date 5/11/66 **Elevation** 690.0

Project Harvard - GSC 614

Sample Depth ft.	Blows per ft.	Moisture Content %	Dry Unit Weight p.c.f.	Depth in feet	Graphic Log	Description
				0		FILL, Silty Sand, fine, brown
					CL	FILL, Silty Clay, lumps of Shale, few pieces of wood, dark grayish brown, about 20% of diatomaceous Shale
5.0	-	23.2	85.0	5		
					CL	Silty Clay, brown
					ML	Clayey Silt, brown roots
10.0	-	28.0	79.0	10		
					CL	Silty Clay, dark brown mottled brown
15.0	-	44.6	68.0	15		
						Shale, weathered, diatomaceous, mottled light brown and light grayish brown
20.0	-	50.4	68.0	20		
25.0	-	61.2	60.0	25		
						End at 25 feet; no Water; no Caving

BORING LOG NUMBER 9A

(See Ref. List No. 2)

Drilling Date 5/12/66 **Elevation** 739.0

Project Harvard - GSC 614

Sample Depth ft.	Blows per ft.	Misture Content %	Dry Unit Weight p.c.f.	Depth in feet.	Graphic Log	Description
						4"Asphaltic Paving and 5" Base Course
				0		Shale, weathered, fractured, mottled light brown and light grayish brown
5.0	-	57.5	50.0	5		diatomaceous
10.0	-	53.5	64.0	10		roots
						grayish brown
15.0	-	47.4	69.0	15		lenses of Sandstone
						(Slide Plane)
20.0	-	45.9	75.0	20		
25.0	-	45.2	75.0	25		
30.0	-	42.7	71.0	30		
						End at 30 feet; no Water; no Caving

BORING LOG NUMBER 10A

(See Ref. List No. 2)

Drilling Date 7/15/66 **Elevation** 737.0

Project Harvard - GSC 614

Sample Depth ft.	Blows per ft.	Moisture Content %	Dry Unit Weight p.c.f.	Depth in feet	Graphic Log	Description
						3" Asphaltic Paving
				0		FILL, Silty Clay, mottled brown
						Shale, weathered, fractured, diatomaceous, mottled light brown and grayish white
5.0	-	42.1	75.0	5		
						layer of limestone
10.0				10		
						(Slide Plane)
15.0	-	55.3	63.0	15		
						lenses of Sandstone
20.0	-	41.0	76.0	20		
25.0	-	42.0	73.0	25		
30.0	-	52.7	66.0	30		
35.0	-	32.1	81.0	35		
						End at 35 feet; no Water; no Caving

BORING LOG NUMBER AA, EA, GA
(See Ref. List No. 2)

Drilling Date 5/12/66 **Elevation** 802.0, 823.0, 729.0

Project Harvard - GSC 614

Sample Depth ft.	Blows per ft.	Moisture Content %	Dry Unit Weight p.c.f.	Depth in feet	Graphic Log	Description
						AA (802.0)
				0	CL	Silty Clay, roots, brown
2.0	-	37.7	64.0			Shale, weathered, highly fractured, mottled brown
				5		End at 2.5 feet; no Water
						EA (823.0)
				0	CL	Silty Clay, brown
4.0	-	25.7	82.0			Shale, weathered, highly fractured, mottled brown
				5		End at 4.0 feet; no Water
						GA (729.0)
				0	CL	Silty Clay, roots (to 10"), dark grayish brown
4.0	-	40.7	73.0			Shale, weathered, fractured, mottled brown
				5		End at 4.0 feet; no Water

BORING LOG NUMBER 16A'

(See Ref. List No. 3)

Drilling Date 7/8/68 **Elevation** 800.0

Project Harvard - GSC 614

Sample Depth ft.	Blows per ft.	Moisture Content %	Dry Unit Weight p.c.f.	Depth in feet.	Graphic Log	Description
				0	CL	Silty Clay, lumps of Shale, roots, mottled gray and brown
						Shale, weathered, fractured, mottled light brown and light grayish brown Bedded
5.0	-	34.8	58.0	5		
10.0	-	24.0	82.0	10		
15.0	-	38.4	50.0	15		seams of Clay, roots
20.0	-	38.4	50.0	20		
25.0	-	44.1	55.0	25		
30.0	-	36.1	65.0	30		
35.0	-	42.3	56.0	35		
40.0	-	41.0	64.0	40		
45.0	-	38.5	77.0	45		grayish brown
50.0	-	32.8	83.0	50		
						End at 50 feet; no Water; no Caving

BORING LOG NUMBER 17A'

(See Ref. List No. 3)

Drilling Date 7/9/68 **Elevation** 825.0

Project Harvard - GSC 614

Sample Depth ft.	Blows per ft.	Moisture Content %	Dry Unit Weight p.c.f.	Depth in feet.	Graphic Log	Description
				0	CL	Silty Clay, lumps of Shale, few Gravel, roots, mottled brown and dary gray Few cobbles
5.0	-	21.9	82.0	5		Shale, weathered, fractured, mottled, light brown and light grayish brown Bedded
10.0	-	27.1	70.0	10		roots
15.0	-	28.5	65.0	15		
20.0	-	27.0	54.0	20		
25.0	-	28.0	80.0	25		
						End at 25 feet; no Water; Raveling from 0'-3' (to 30" in diameter)

BORING LOG NUMBER 18A'

(See Ref. List No. 3)

Drilling Date 7/9/68 **Elevation** 850.0

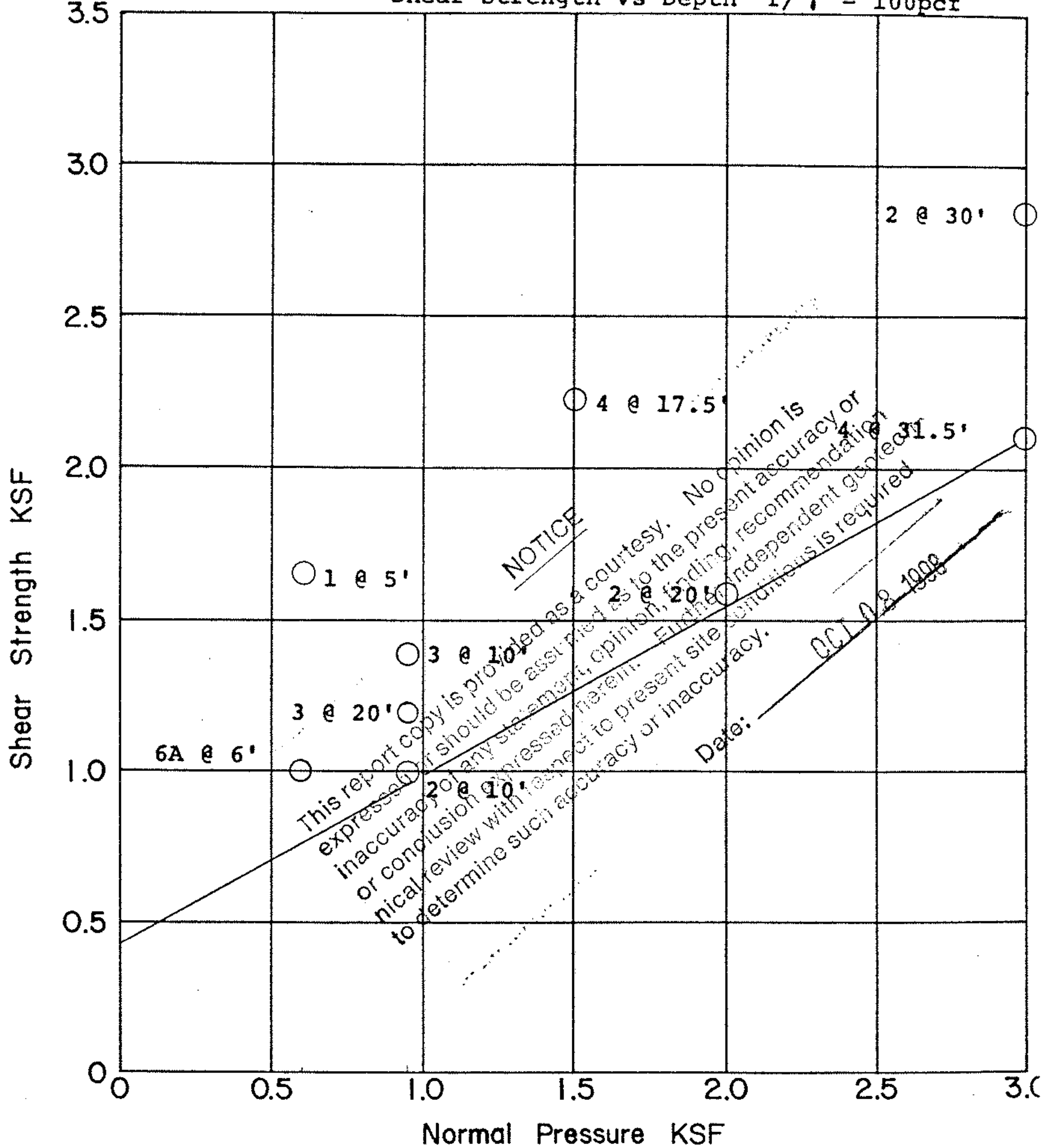
Project Harvard - GSC 614

Sample Depth ft.	Blows per ft.	Moisture Content %	Dry Unit Weight p.c.f.	Depth in feet	Graphic Log	Description
				0	CL	FILL, Silty Clay, lumps of Shale, mottled brown and gray
						Decomposed wood
5.0	-	24.6	59.0	5		Shale, weathered, fractured, roots, mottled light brown and light grayish brown Bedded
10.0	-	31.3	71.0	10		
15.0	-	34.6	68.0	15		
20.0	-	33.9	57.0	20		
				25		
						End at 25 feet; no Water; Raveling from 0'-5' (to 30" in diameter)

SHEAR TEST DIAGRAM

Project Harvard - GSC 614

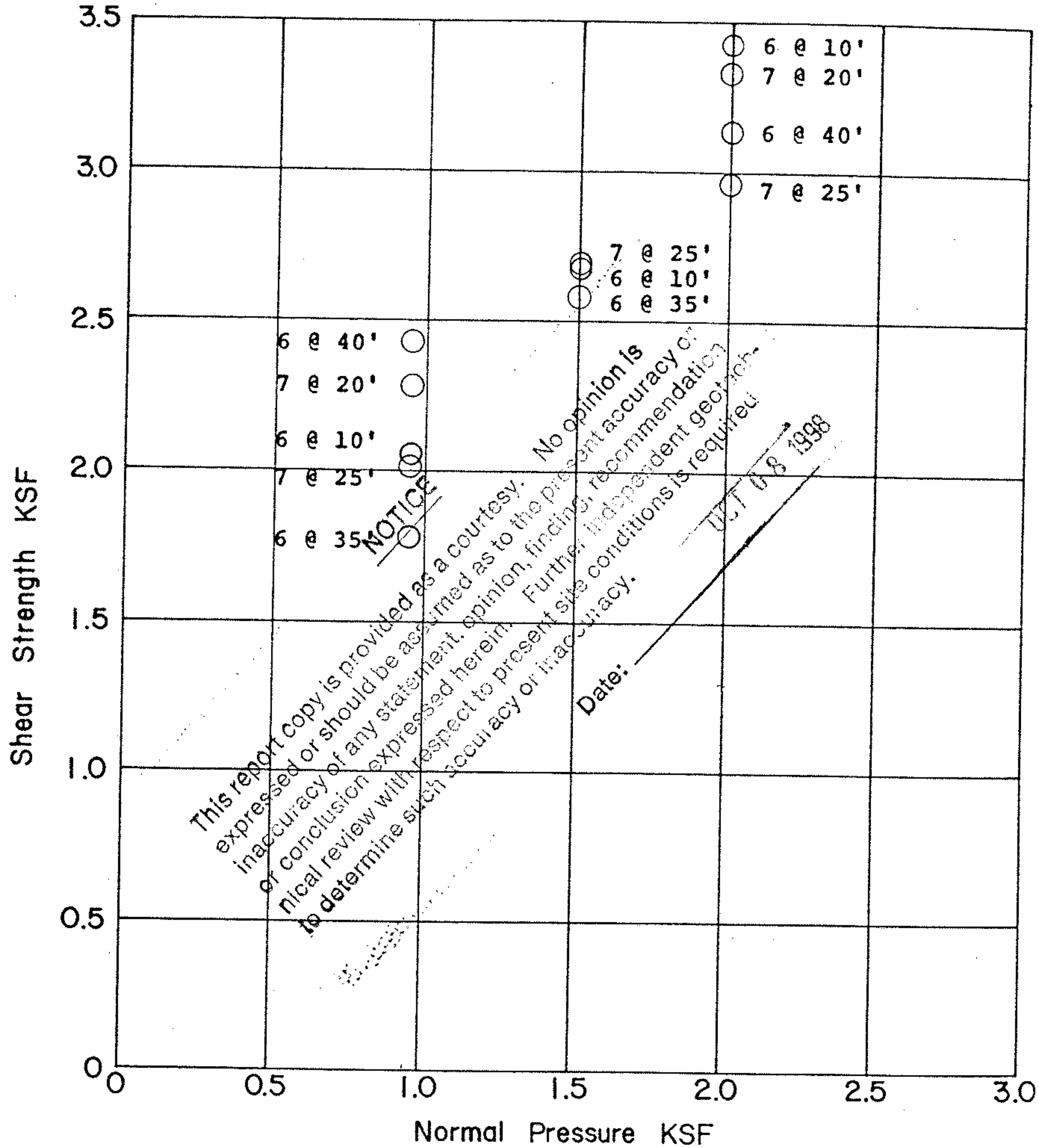
Shear Strength vs Depth f/ γ = 100pcf



- ○ Direct Shear at Field Moisture
- ● Direct Shear, Saturated
- ◡ Unconfined Compression Test
- ⊕ Vane Shear Test
- ⊙ Penetrometer

SHEAR TEST DIAGRAM

Project Harvard - GSC 614 (Library)

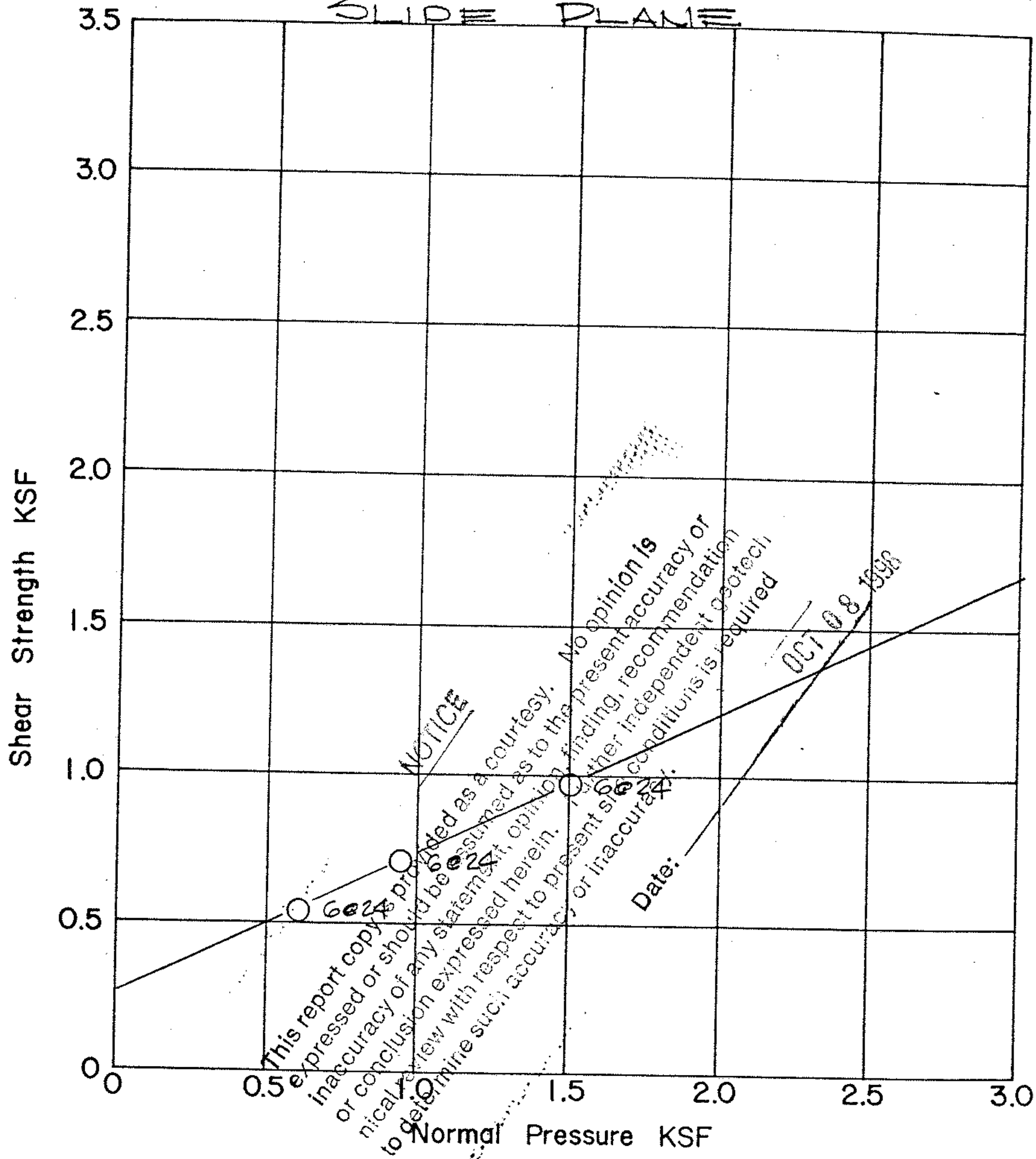


- ○ Direct Shear at Field Moisture
- ● Direct Shear, Saturated
- ◡ Unconfined Compression Test
- ⊕ Vane Shear Test
- ⊙ Penetrometer

SHEAR TEST DIAGRAM

Project HARVARD SCHOOL GSC 614

SLIDE PLANE

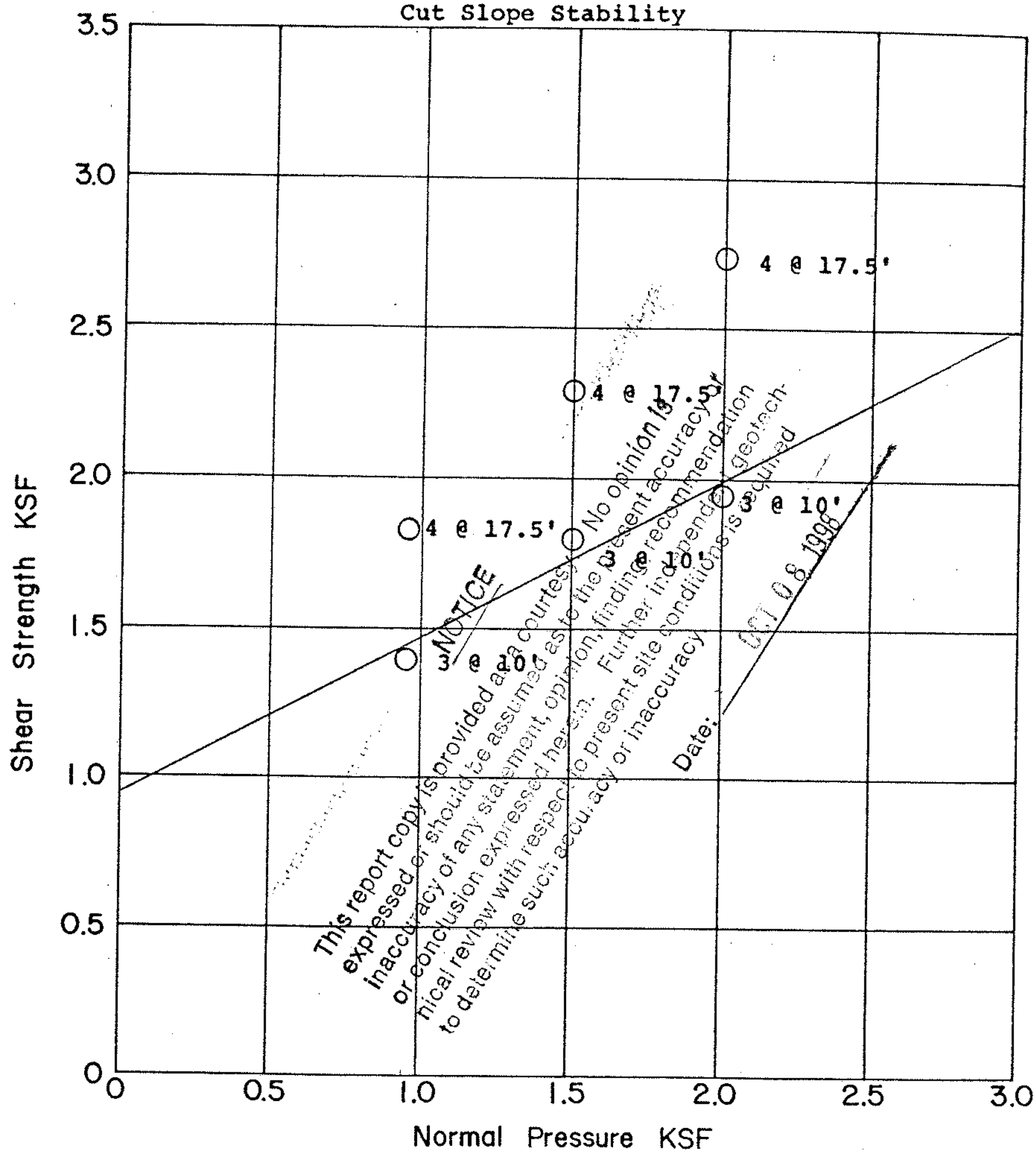


- ○ Direct Shear at Field Moisture
- ● Direct Shear, Saturated
- ◡ Unconfined Compression Test
- ⊕ Vane Shear Test
- ⊙ Penetrometer

SHEAR TEST DIAGRAM

Project Harvard - GSC 614

Cut Slope Stability

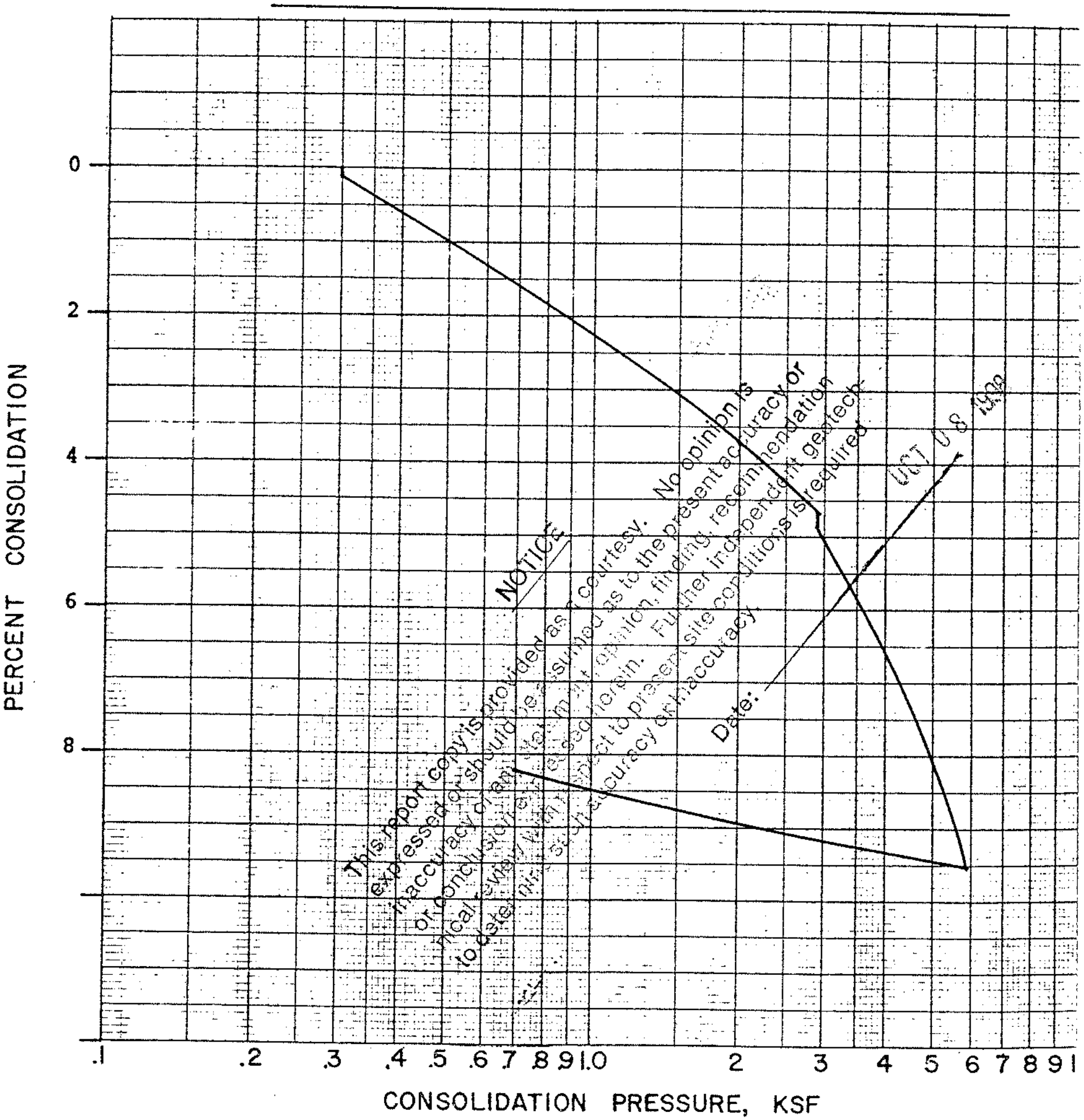


○ Direct Shear at Field Moisture
● Direct Shear, Saturated
◡ Unconfined Compression Test
⊕ Vane Shear Test
⊙ Penetrometer

CONSOLIDATION TEST

Project Harvard - GSC 614

Sample 1 @ 15' Water added at 3ksf

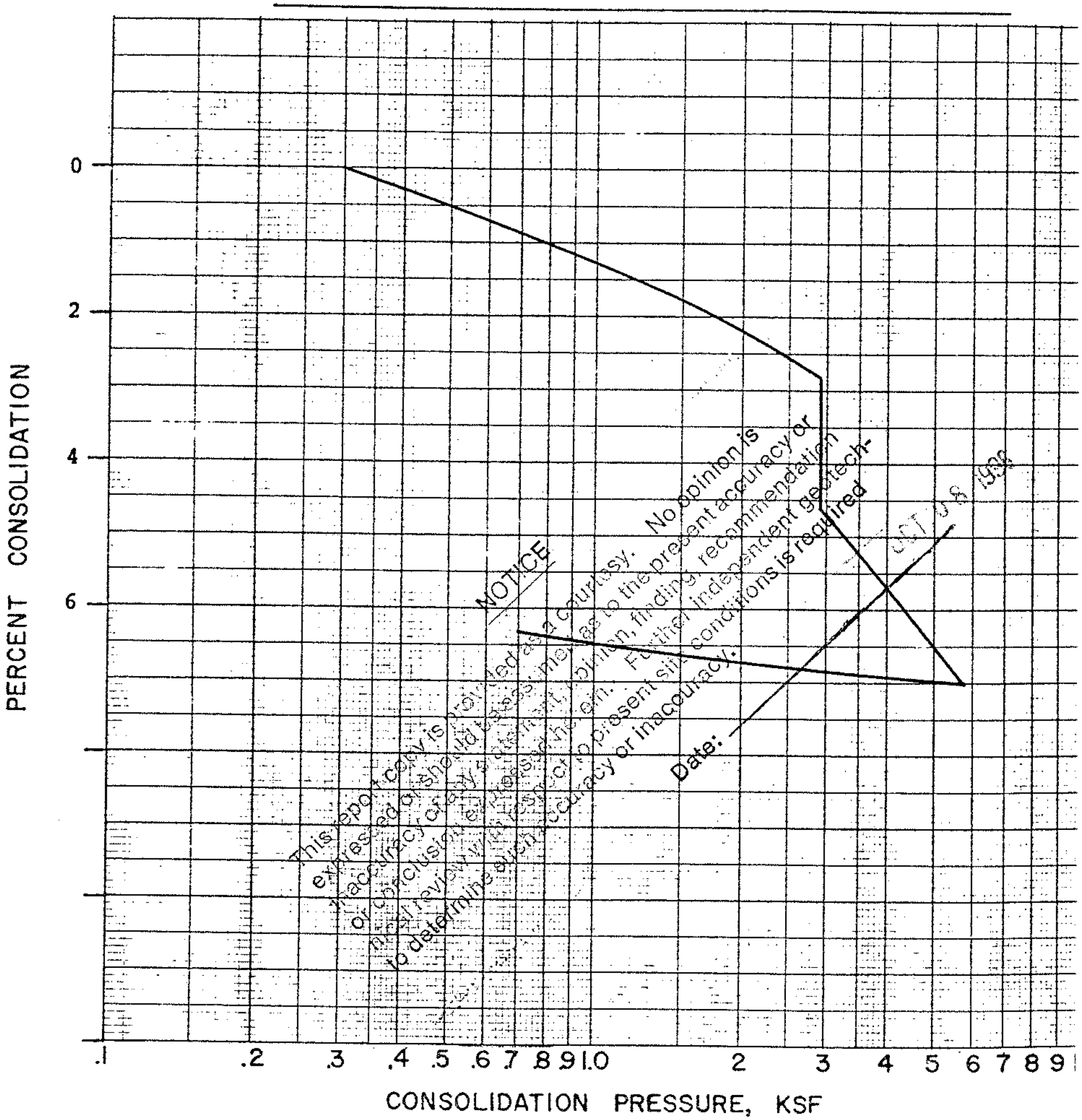


GEOLOGY and SOILS CONSULTANTS, Inc. PLATE C-1

CONSOLIDATION TEST

Project Harvard - GSC 614

Sample 2 @ 10' Water added at 3ksf

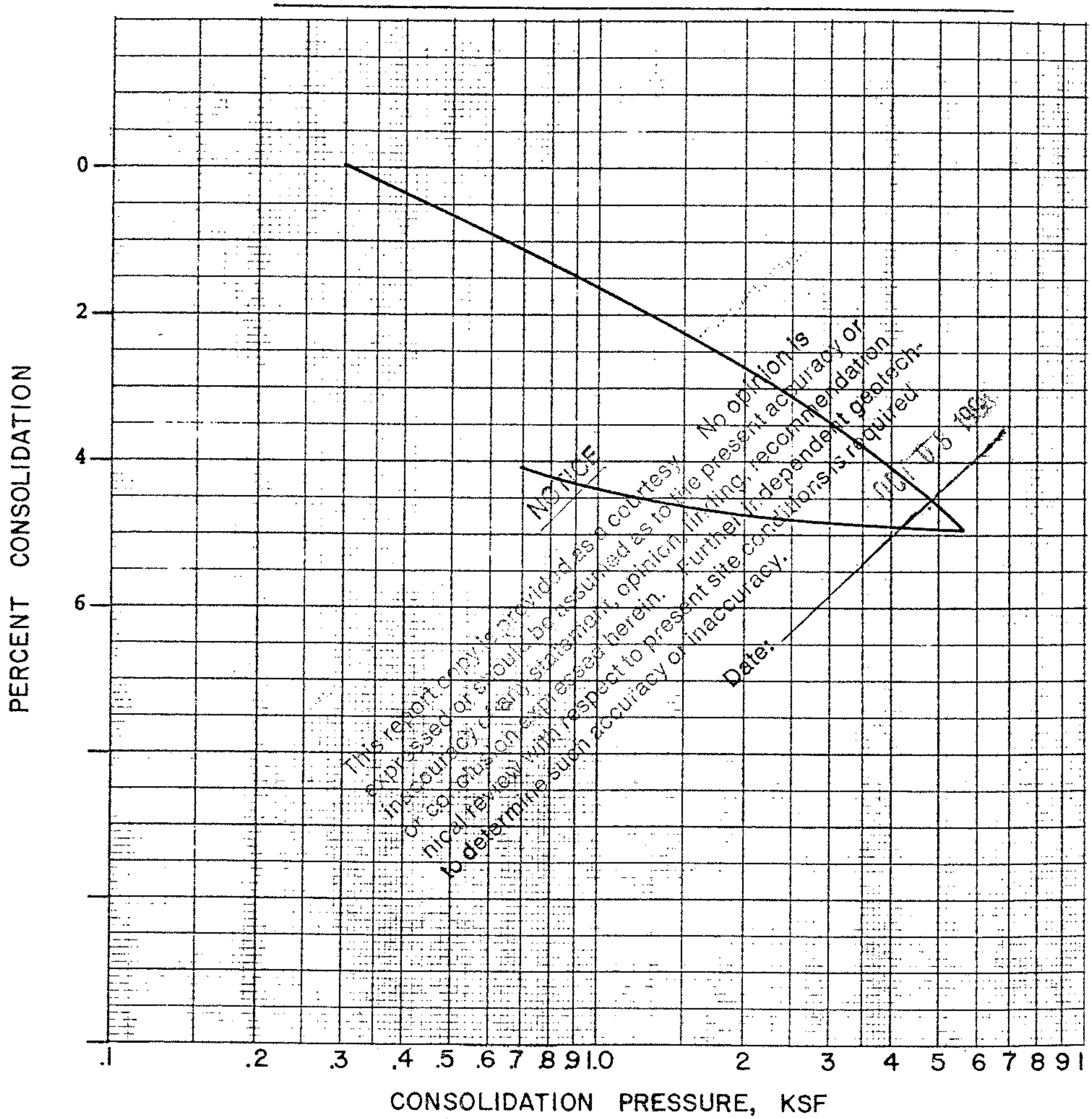


GEOLOGY and SOILS CONSULTANTS, Inc. PLATE C-2

CONSOLIDATION TEST

Project Harvard - GSC 614

Sample 2 @ 20' Water added at 3ksf

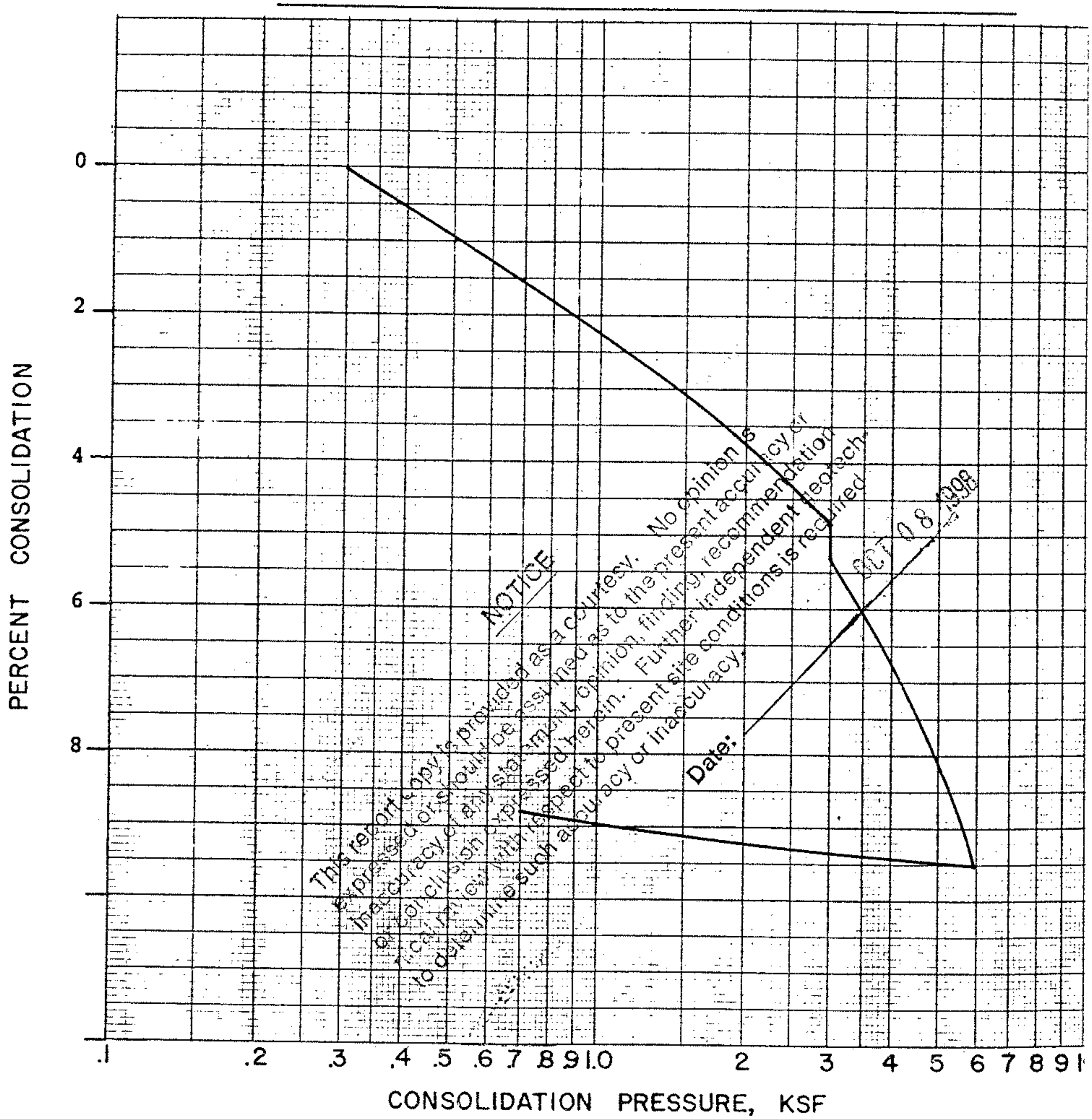


GEOLOGY and SOILS CONSULTANTS, Inc. PLATE C-3

CONSOLIDATION TEST

Project Harvard - GSC 614

Sample 3 @ 20' Water added at 3ksf

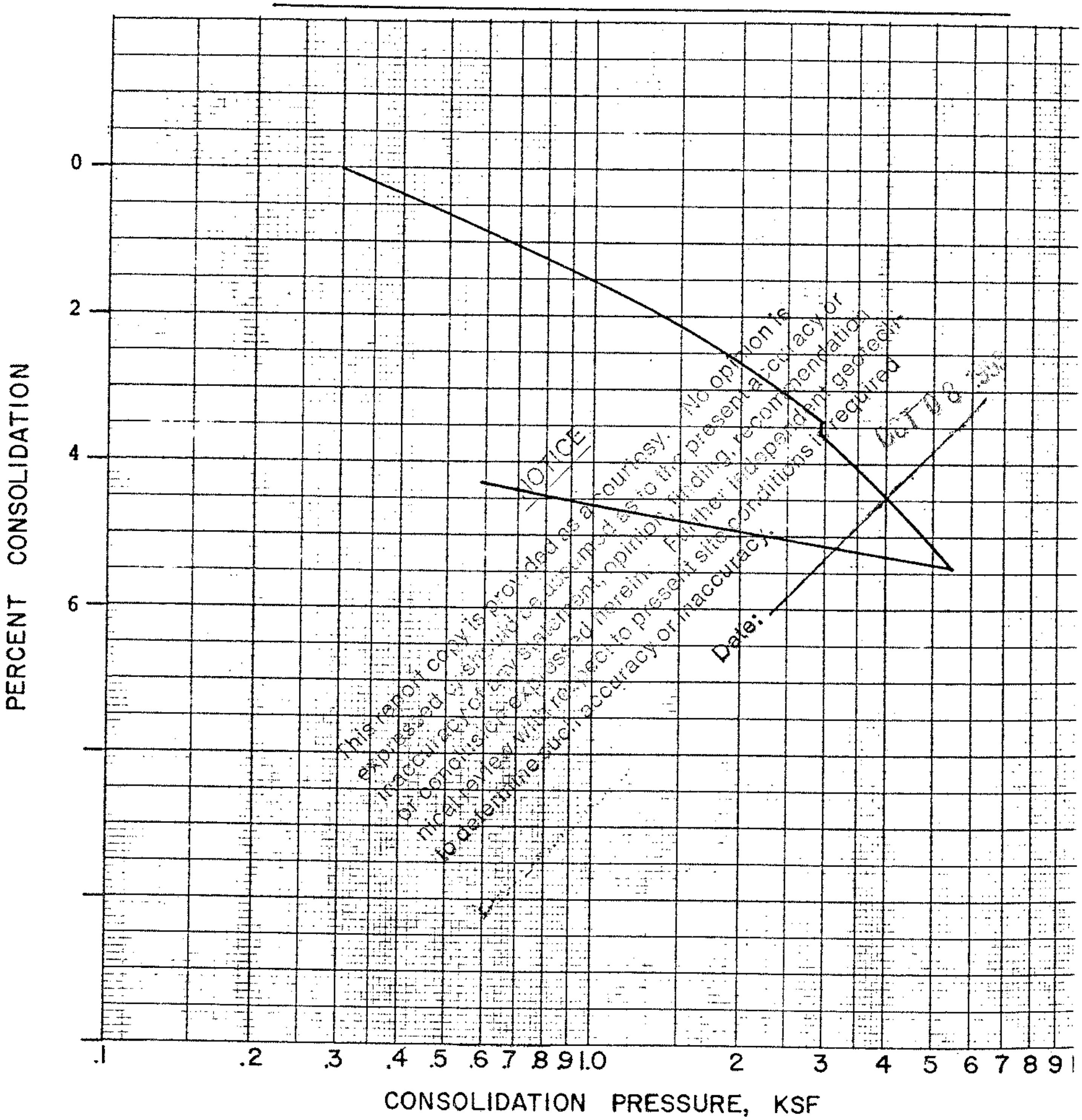


GEOLOGY and SOILS CONSULTANTS, Inc. PLATE C-4

CONSOLIDATION TEST

Project Harvard - GSC 614

Sample 2 @ 30' Water added at 3ksf

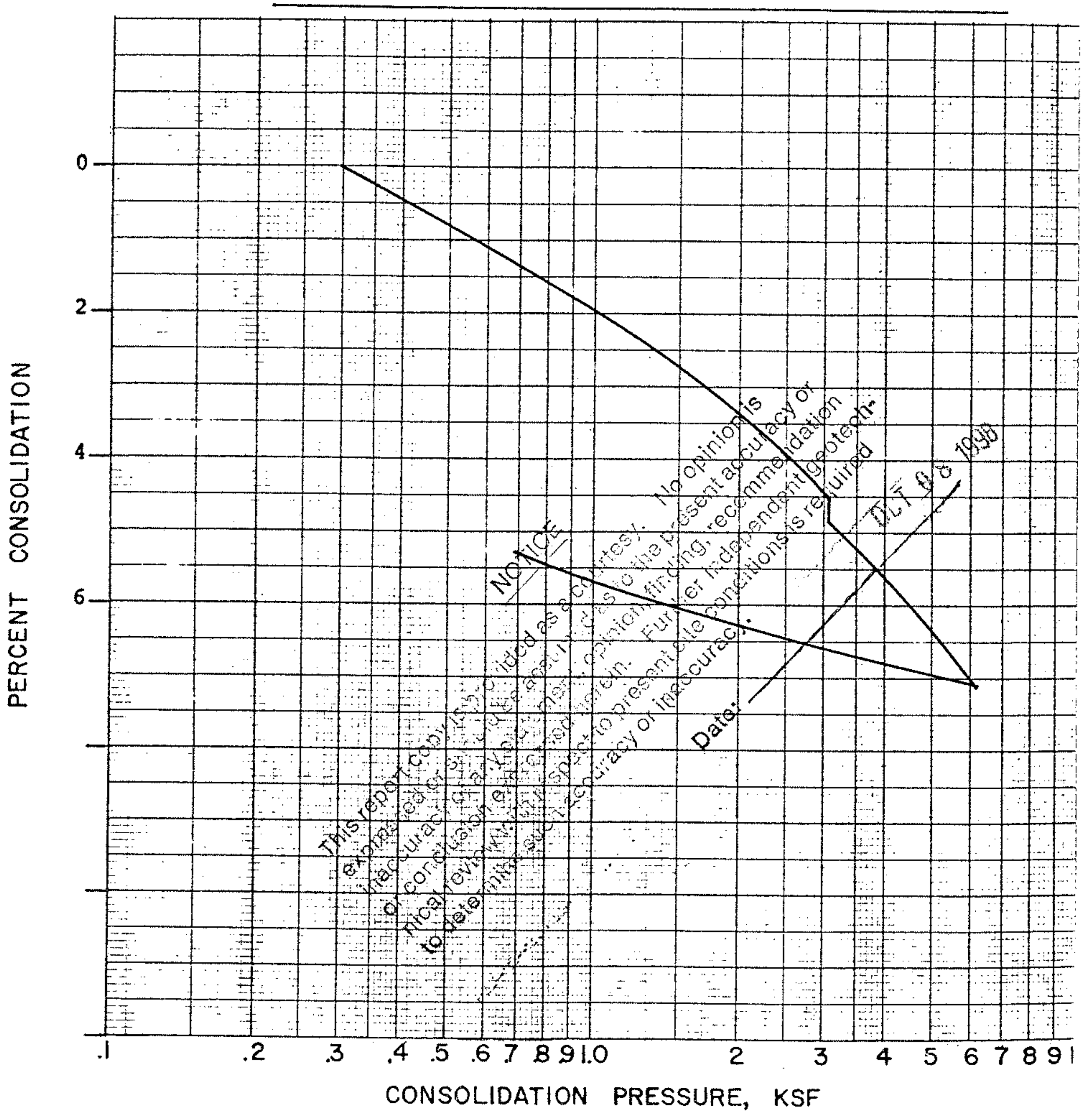


GEOLOGY and SOILS CONSULTANTS, Inc. PLATE C-5

CONSOLIDATION TEST

Project Harvard - GSC 614

Sample 4 @ 31.5' Water added at 3ksf

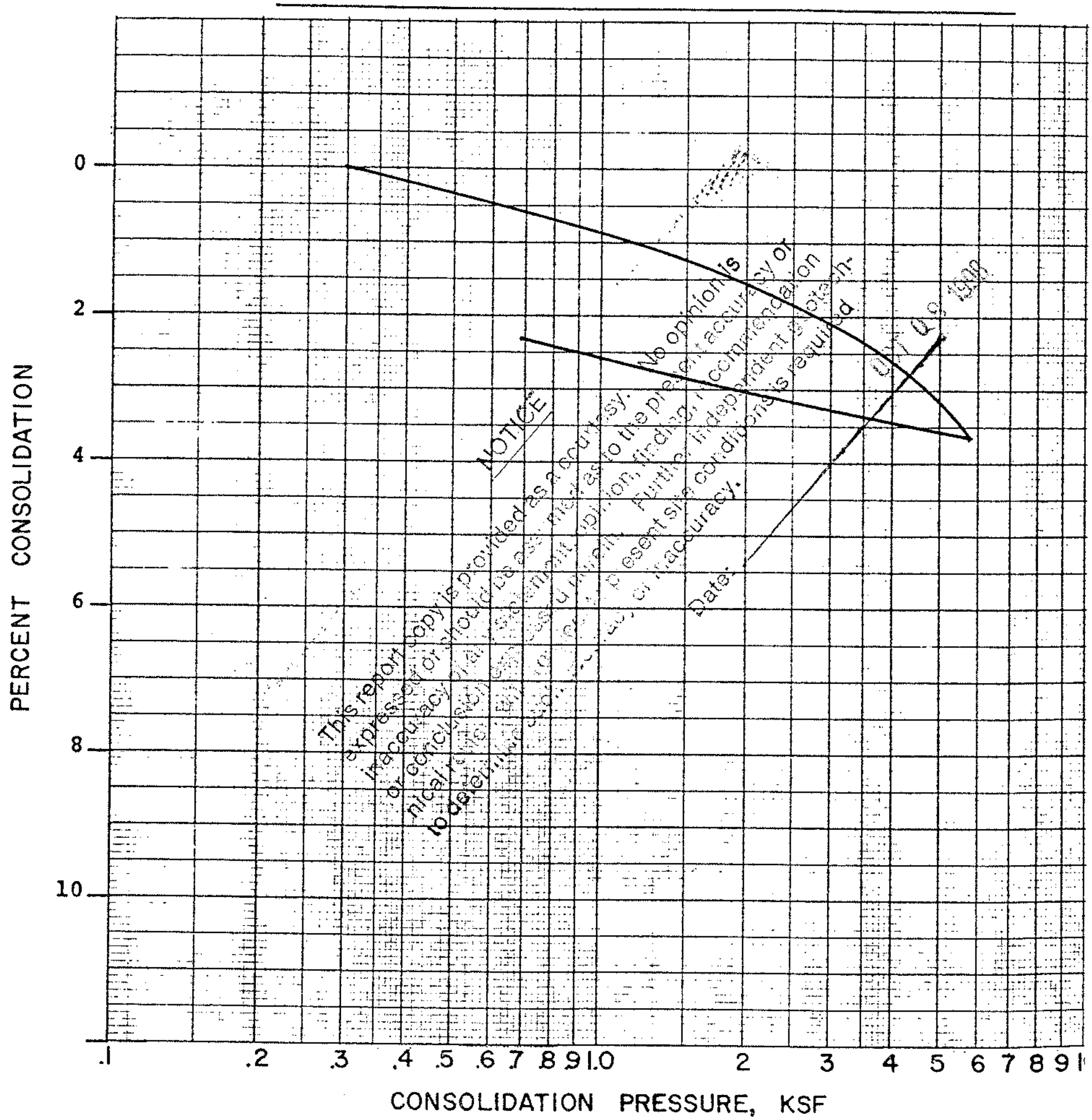


GEOLOGY and SOILS CONSULTANTS, Inc. PLATE C-6

CONSOLIDATION TEST

Project Harvard - GSC 614

Sample 2 @ 35.0' Water added at 3 ksf



SLIDE STABILITY CALCULATIONS

SEGMENT	$\gamma = 110$ W (K)	N W Cos θ	D W Sin θ	$c = 260$ cL	N tan ϕ	R
1	$(\frac{9+6}{2})(125) = 940$ 103	$\theta = 17.5°$ 98	31.0	30.	45.7	75.7
2	$(10+5)/2\,(130) = 975$ 107	$\theta = 14°$ 104	25.9	33.8	48.6	82.4
3	$(10+12)/2\,(120) = 1320$ 145	$\theta = 12.5°$ 142	31.4	31.2	66.4	97.6
4	$(12+7)/2\,(120) = 1140$ 125	$\theta = 10°$ 123	21.6	31.2	57.5	887
5	$\frac{1}{2}(7)(105) = 370$ 40.7	$\theta = 11°$ 40.	-7.8	-28.6	-18.7	-47.3
			102.1			297.1

$$F.S. = \frac{297.1}{102.1} = 2.9$$

Geology & Soils Consultants Inc.

BY G.C.M DATE 24 JAN SUBJECT HARVARD SCHOOL

CHKD ____ DATE ____

GSC 614

PLATE D

January 29, 1973
GSC 614

REFERENCE LIST

1. Converse Foundation Engineers, Foundation Report, dated April 10, 1961. Job #61-169-A

2. LeRoy Crandall and Associates and Hood & Schmidt, Foundation Investigation, report dated August 26, 1966, Job #A-66131; City of Los Angeles Approval Letter, Board File No. 670179

3. LeRoy Crandall and Associates and Hood & Schmidt, Supplemental Geologic Report and Supplemental Foundation Report, dated July 26, 1968. Job #A-66131-C; City of Los Angeles Approval Letter, Board File No. 670179

4. LeRoy Crandall and Associates, Foundation Investigation and Subsurface Drainage System, dated November 17, 1967. Job #B67150; City of Los Angeles Approval Letter, dated December 19, 1967.

ENGINEERING GEOLOGY ■ SOILS & FOUNDATION ENGINEERING
12526 VENTURA BLVD., STUDIO CITY, CALIFORNIA 91604 (213)980-0825 (L.A.)877-2757

Appendix III

Figures

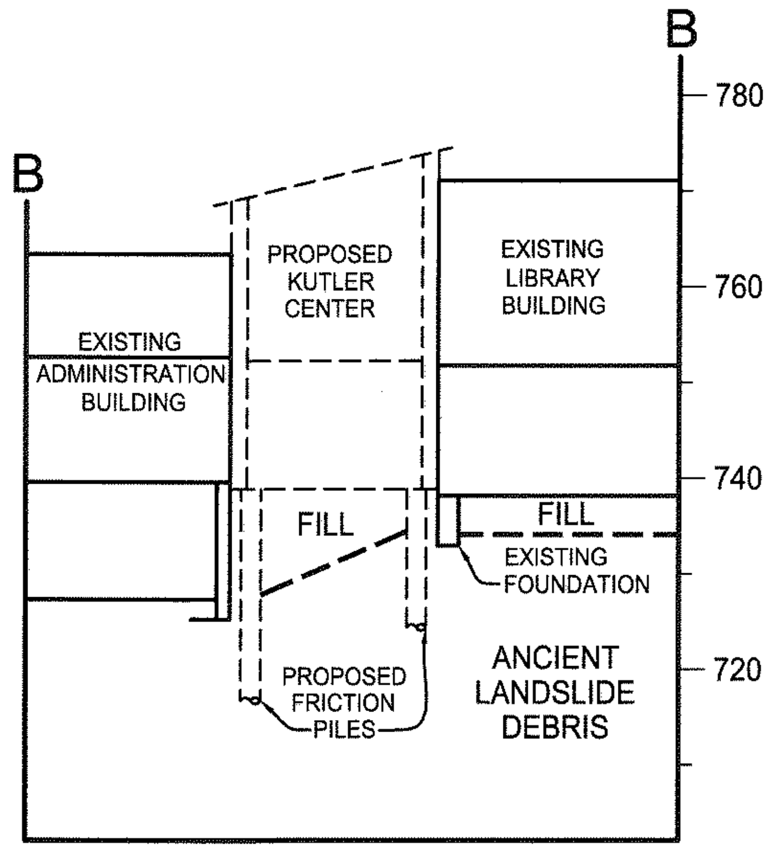
BYER
GEOTECHNICAL
INC.
1461 E. CHEVY CHASE DR., SUITE 200
GLENDALE, CA 91206
818.549.9959 TEL
818.543.3747 FAX

SECTION B

BG: 21256 HARVARD - WESTLAKE SCHOOL

CONSULTANT: JET	SCALE: 1" = 20'

OCTOBER 31, 2011



SECTION B

BYER
GEOTECHNICAL
INC.
1461 E. CHEVY CHASE DR., SUITE 200
GLENDALE, CA 91206
818.549.9959 TEL
818.543.3747 FAX

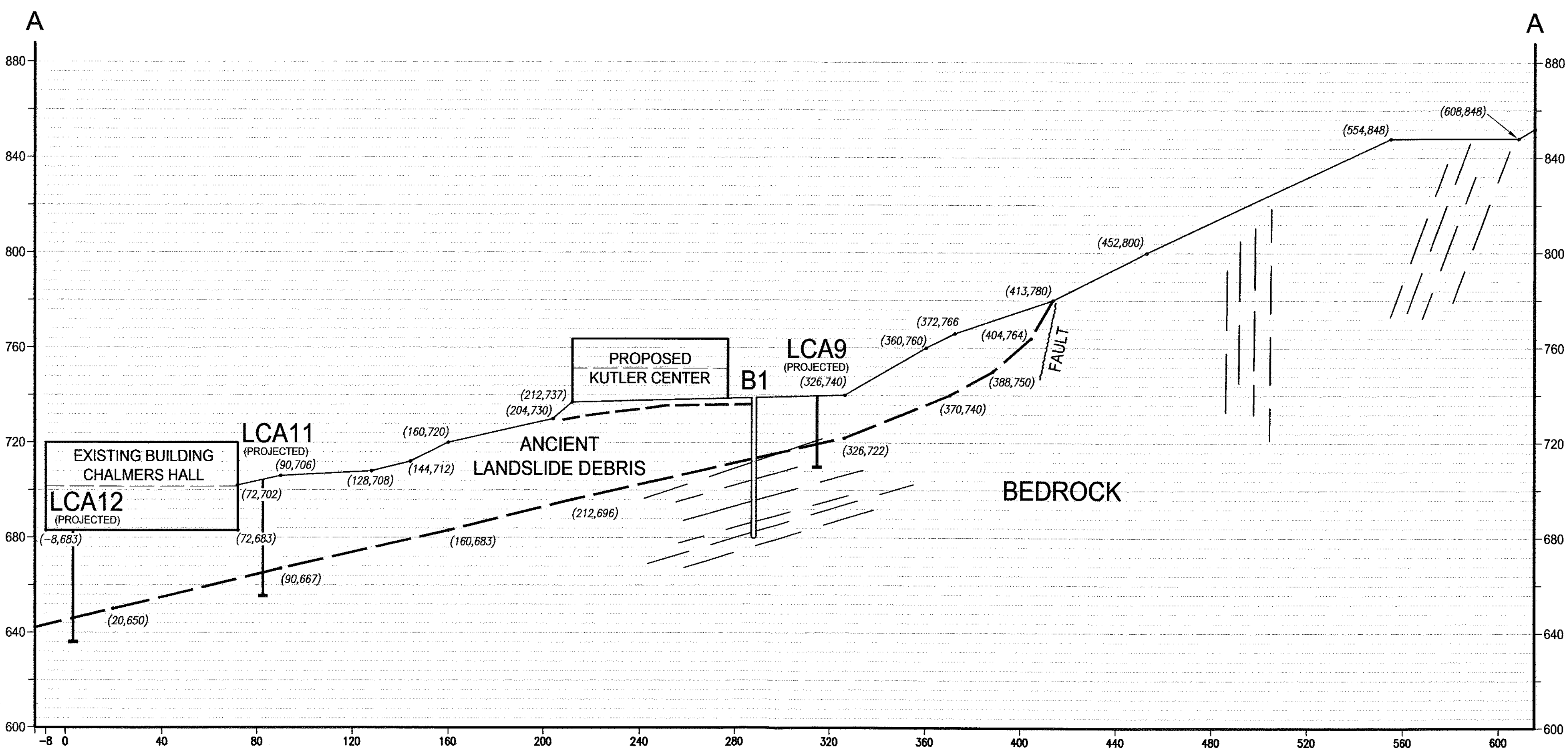
REVISED SECTION A

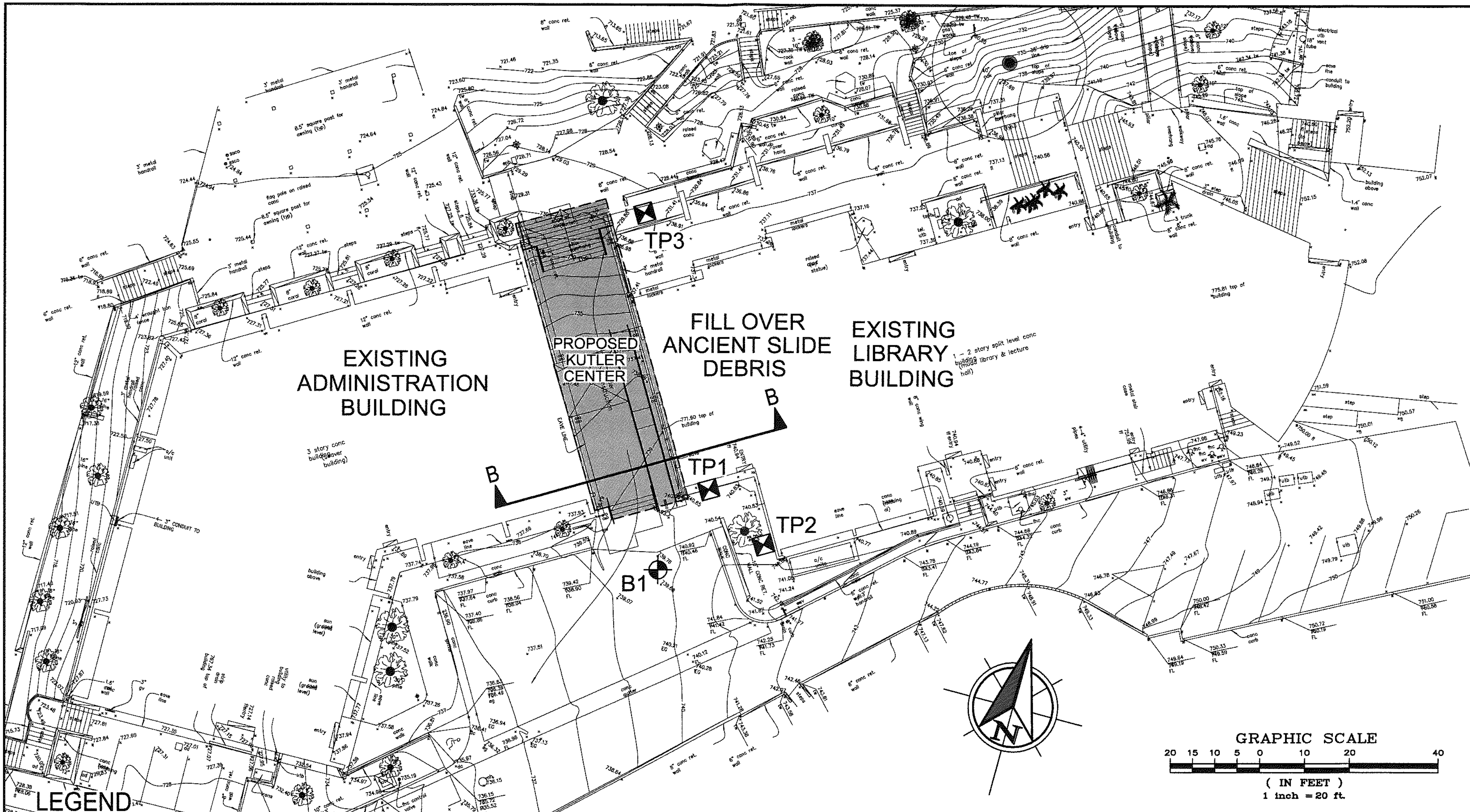
BG: 21256 HARVARD-WESTLAKE SCHOOL

CONSULTANT: JET

SCALE: 1" = 40'

OCTOBER 31, 2011





LEGEND

TP3 LOCATION AND NUMBER OF HAND-DUG TEST PIT

B1 LOCATION AND NUMBER OF BORING

B—B LINE OF CROSS SECTION

OCTOBER 31, 2011

REFERENCE: SITE PLAN, SHEET A1.10, PREPARED BY TOBIAS ARCHITECTURE, AND DATED 9/15/2011.

BYER GEOTECHNICAL INC.
1461 E. CHEVY CHASE DR., SUITE 200
GLENDALE, CA 91206
818.549.9959 TEL
818.543.3747 FAX

GEOLOGIC MAP 2

BG: 21256 HARVARD - WESTLAKE SCHOOL

CONSULTANT: JET

SCALE: 1" = 20'

BYER GEOTECHNICAL, INC.

January 10, 2012
BG 21256

Harvard-Westlake School
700 North Faring Road
Los Angeles, California 90077

Attention: Mr. Jim DeMatte

Subject

Summary of Friction Pile Excavations
Proposed Mudd Library Renovation
Arb. 1, Portion of Lot 1111, Tract 1000
3700 North Coldwater Canyon Avenue
North Hollywood, California

References: Reports by Byer Geotechnical, Inc.:

Geologic and Soils Engineering Exploration, Proposed Brendon Kutler Center and Mudd Library Renovation, Arb. 1, Portion of Lot 1111, Tract 1000, 3700 North Coldwater Canyon Avenue, North Hollywood, California, dated December 30, 2010; and

Addendum Geologic and Soils Engineering Exploration, Response to City of Los Angeles Correction Letter, Proposed Brendon Kutler Center and Mudd Library Renovation, Arb. 1, Portion of Lot 1111, Tract 1000, 3700 North Coldwater Canyon Avenue, North Hollywood, California, dated October 31, 2011.

Response by the City of Los Angeles, Department of Building and Safety (LADBS):

Geology and Soils Report Approval Letter, Log # 74548-01, dated November 15, 2011.

Gentlepersons:

Byer Geotechnical has prepared this report to summarize our observations of 12 friction pile excavations. The piles were excavated per the approved plans. The 24-inch-diameter piles achieve the required embedment into ancient landslide debris and achieve the required H/3 horizontal setback from the descending slope.

The pile excavations and embedment depths were observed and approved by Peter Kilbury, engineering geologist, on December 1 and 2, 2011. The pile excavations are summarized on the enclosed Pile Table and Notices of Field Observation. The City of Los Angeles may require copies of this report in order to close the permit file.

Byer Geotechnical appreciates the opportunity to continue as your geotechnical consultant. Any questions regarding this or the referenced reports should be directed to the undersigned.

Respectfully submitted,
BYER GEOTECHNICAL, INC.

ENGINEERING GEOLOGIST JOHN W. BYER E. G. 883 Exp. 4/30/12 STATE OF CALIFORNIA

Peter Kilbury
E. G. 2401

PK:JWB:mh
S:\FINAL\BG\21256_Harvard-Westlake\21256_Harvard-Westlake_Pile_Report.wpd

Enc: Pile Table
Notices of Field Observation dated December 1 and 2, 2011 (2 Pages)

xc: (4) Addressee

PILE TABLE

PILE NO.	DATE	DIAMETER (Inches)	APPROXIMATE DRILL ELEVATION (Feet)	REQUIRED EMBEDMENT INTO ANCIENT LANDSLIDE DEBRIS (Feet)	ACTUAL EMBEDMEN T INTO ANCIENT LANDSLIDE DEBRIS (Feet)	TOTAL DEPTH (Feet)
1	12-1-11	24	732.0	12	14	21
2	12-2-11	24	734.0	12	13	21
3	12-2-11	24	736.0	12	13	21
4	12-2-11	24	737.0	12	13	21
5	12-2-11	24	738.5	12	14	22
6	12-2-11	24	739.0	12	13	22
7	12-1-11	24	732.0	12	16	21
8	12-1-11	24	734.0	12	16	21
9	12-2-11	24	736.0	12	16	21
10	12-2-11	24	737.0	12	17	21
11	12-2-11	24	738.5	12	20	23
12	12-2-11	24	739.0	12	19	22

BYER GEOTECHNICAL, INC.

NOTICE OF FIELD OBSERVATION

CLIENT: Hamwred Westlake DATE: 12-1-11 TIME: 10 BG#: 21256

LOCATION: 3700 Cold water Cny

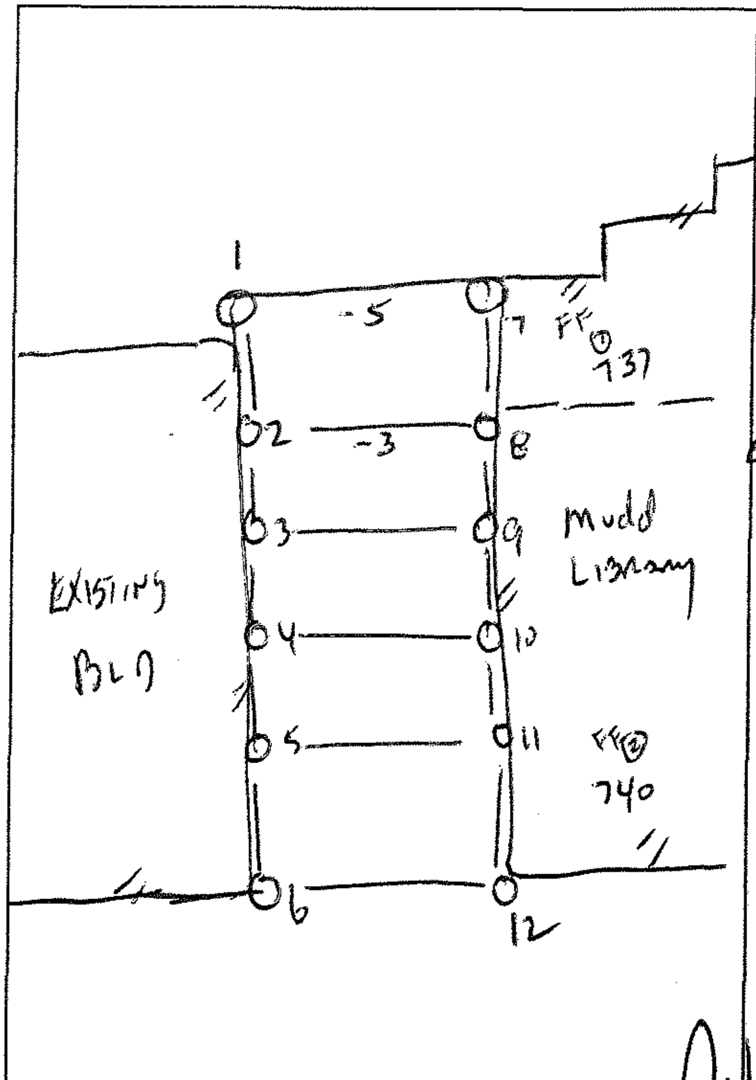
REQUESTED BY: Tony MET WITH: Walid

SPECIAL CONDITIONS: /

(WEATHER. JOB SHUTDOWN, ADVICE IGNORED, SAFETY)

WE HAVE OBSERVED THE: Piles

☐ APPROVED PER THE PLANS ☐ CALL AGENCY INSPECTOR ☐ DISAPPROVED ☑ SEE BELOW



GENERAL CONDITIONS OF APPROVAL:

Pile	Φ	Required	Achieved	TD
1	24"	12'	14	21
8	↓	↓	16	21
7			16	21

Piles in progress. 3 piles drilled achieve atleast 12' embedment into ancient slide debris as per plan.

ADDITIONAL SITE VIST(S): ☐ REQUIRED ☐ NOT REQUIRED

FOR BYER GEOTECHNICAL, INC.:

HOURS: 2 (2 HOUR MINIMUM CHARGE) NOTICE LEFT WITH: Walid

BYER GEOTECHNICAL, INC.

NOTICE OF FIELD OBSERVATION

CLIENT: Hathaway Westlake DATE: 12-2-11 TIME: 12 BG# 21256

LOCATION: 3700 Coldwater Cny

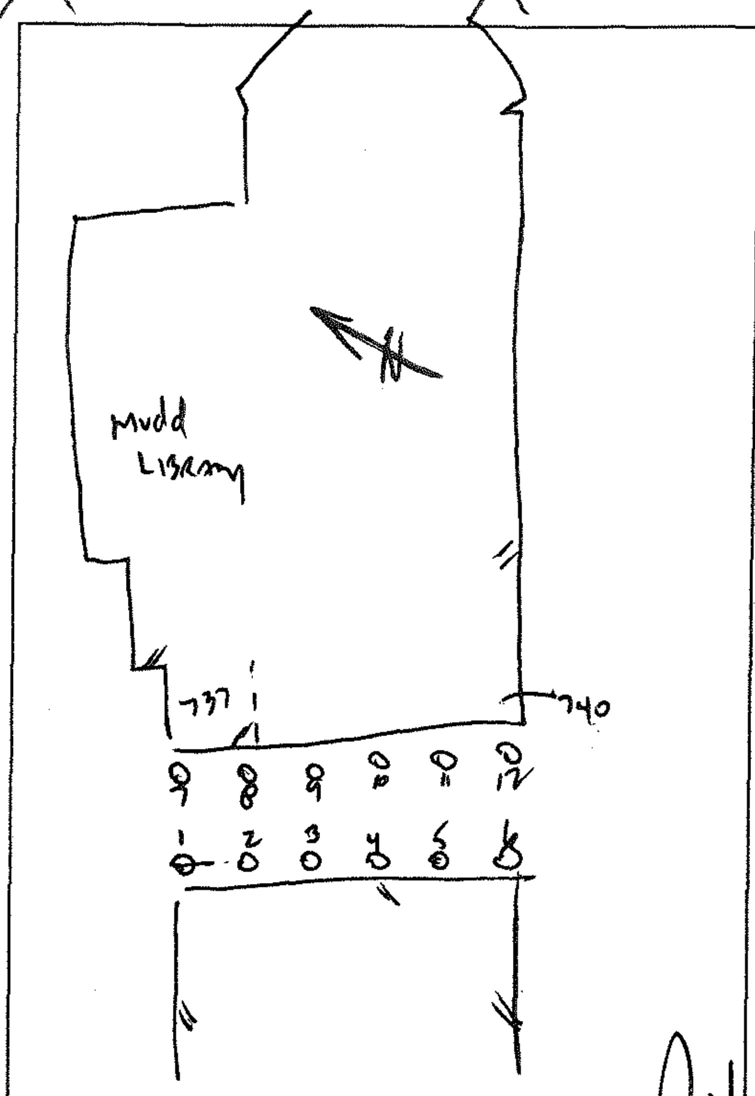
REQUESTED BY: Tony MET WITH: Tony

SPECIAL CONDITIONS:

(WEATHER, JOB SHUTDOWN, ADVICE IGNORED, SAFETY)

WE HAVE OBSERVED THE: Piles - Mudd Library Addition

☒ APPROVED PER THE PLANS ☒ CALL AGENCY INSPECTOR ~~☐ DISAPPROVED~~ ☒ SEE BELOW



GENERAL CONDITIONS OF APPROVAL:

Pile	∅	Req Embed	Actual Embed	Drill el	TD
1	24"	12'	14	732	21
2			13	734	21
3			13	736	21
4			13	737	21
5			14	738.5	22
6			13	739	22
7			16	732	21
8			16	734	21
9			16	736	21
10			17	737	21
11			20	738.5	23
12			19	739	22

Piles 1-12 are approved & are embedded into firm ancient landslide debris.

ADDITIONAL SITE VIST(S): ☒ REQUIRED ☐ NOT REQUIRED

FOR BYER GEOTECHNICAL, INC: [signature]

HOURS: 2 (2 HOUR MINIMUM CHARGE) NOTICE LEFT WITH: Tony

1461 East Chevy Chase Drive, Suite 200 • Glendale, California 91206 • tel 818.549.9959 • fax 818.543.3747 • www.byergeo.com

BYER GEOTECHNICAL, INC.

February 29, 2012
BG 21401

DWR Construction, Inc.
3051 Bostonian Drive
Los Alamitos, California 90720

Attention: Mr. Douglas W. Roberts, President

Subject

Transmittal Letter - Compaction Report
Proposed Swimming Pool and Pool House
Grading Permit # 11030 - 20000 - 05106
Arb. 1, Lot 1111, Tract 1000
3700 North Coldwater Canyon Avenue
Studio City, California

Dear Mr. Roberts:

Byer Geotechnical has completed our Compaction Report dated February 29, 2012, which describes the grading with respect to preparing the site for the proposed swimming pool and pool house. The reviewing agency for this document is the City of Los Angeles, Department of Building and Safety (LADBS). The city requires a filing fee to accompany compaction reports. Copies of the report have been distributed as follows:

(1) Addressee (E-mail and Mail)
(1) DWR Construction, Attention: Mr. Anthony Damiano (E-mail)
(1) LADBS, Attention: Mr. Yervand Chapanyan (E-mail)
(3) LADBS

Byer Geotechnical will file the report with the LADBS. Any questions concerning the report should be directed to the undersigned. Byer Geotechnical appreciates the opportunity to offer our service on this project.

Very truly yours,
BYER GEOTECHNICAL, INC.

Raffi S. Babayan
Project Engineer

BYER GEOTECHNICAL, INC.

COMPACTION REPORT

PROPOSED SWIMMING POOL AND POOL HOUSE

GRADING PERMIT # 11030 - 20000 - 05106

ARB. 1, LOT 1111, TRACT 1000

3700 NORTH COLDWATER CANYON AVENUE

STUDIO CITY, CALIFORNIA

FOR DWR CONSTRUCTION, INC.

BYER GEOTECHNICAL, INC., PROJECT NUMBER BG 21401

FEBRUARY 29, 2012

1461 East Chevy Chase Drive, Suite 200 • Glendale, California 91206 • tel 818.549.9959 • fax 818.543.3747 • www.byergeo.com

COMPACTION REPORT
PROPOSED SWIMMING POOL AND POOL HOUSE
GRADING PERMIT # 11030 - 20000 - 05106
ARB. 1, LOT 1111, TRACT 1000
3700 NORTH COLDWATER CANYON AVENUE
STUDIO CITY, CALIFORNIA
FOR DWR CONSTRUCTION, INC.
BYER GEOTECHNICAL, INC., PROJECT NUMBER BG 21401
FEBRUARY 29, 2012

INTRODUCTION

This report summarizes results of compaction testing and field observations performed during grading of a portion of the site. The purpose of the compaction testing was to determine that the grading specification on the plan and the requirements of the City of Los Angeles Building Code were met. The results of the compaction tests are shown on "Table I" and the test locations are plotted on the enclosed Compaction Map.

Field observations and compaction testing were coordinated with Anthony of DWR Construction, Inc.

PRIOR WORK

The following geotechnical report was prepared for the project by Byer Geotechnical, Inc.:

> *Geologic and Soils Engineering Exploration Update. Proposed Pool, Pool House, and Retaining Wall, Harvard-Westlake School, Lot 1111, Tract 1000, 3700 Coldwater Canyon Avenue, Studio City, California,* dated September 20, 2011.

The City of Los Angeles, Department of Building and Safety, reviewed the report and issued the conditional approval letter, Log # 75188, dated October 28, 2011.

SOIL CLASSIFICATION

The following soil types were used in the compacted fill:

Soil Type	Soil Description	Soil Color	Maximum Dry Density (pcf)	Optimum Moisture Content (%)	Expansion Index*
A	Clayey Silt	Dark Gray-Brown	110.00	19.0	70 - Moderate
B	Clayey Sand	Medium Brown	112.0	18.0	49 - Low

* Expansion Index as determined by Expansion Index Method (UBC Standard 29-2 or ASTM 4829-8a).

The maximum density tests were performed in accordance with ASTM D 1557-09.

PROJECT DESCRIPTION

The grading addressed in this report consisted of preparing the site for the proposed swimming pool and pool house. The approved compacted fill will be used as primary structural fill supporting the proposed swimming pool and pool house, and secondary structural fill supporting hardscape.

GRADING

Areas to receive compacted fill were cleared of vegetation and debris. Prior to placing fill, the existing fill and disturbed alluvium were removed, a minimum of three feet below the bottom of the proposed swimming pool and pool house and a minimum of three feet beyond the building footprint. The excavated soils were stockpiled for later placement as compacted fill. The bottom was observed and approved by a representative of the soils engineer. The approved bottom was scarified to a depth of six inches, moistened as required to achieve optimum moisture content, and recompacted to 90 percent of the maximum dry density.

Compaction

Fill was placed by means of an excavator, in loose lifts of about six inches, moistened as required to achieve optimum moisture content by means of a water hose, and compacted with a

Field density tests were performed in accordance with ASTM D 1556-07. Field density tests as shown on "Table I" indicate that compacted fill was placed to a minimum of 90 percent of the maximum dry density.

The maximum vertical depth of fill is nine feet, as shown on the enclosed Compaction Map.

CONCLUSIONS AND RECOMMENDATIONS

Field density tests indicate that compacted fill was placed in a satisfactory manner and is suitable for support of the proposed swimming pool, pool house, and hardscape. The grading was performed according to the approved plan prepared by Arch Pac Incorporated.

FOUNDATION DESIGN

General Conditions

The following foundation recommendations are minimum requirements. The structural engineer may require footings that are deeper or wider, depending on the final loads.

Spread Footings

Continuous and/or pad footings may be used to support the proposed swimming pool and pool house, provided they are founded in approved compacted fill. Continuous footings should be a minimum of 12 inches in width. Pad footings should be a minimum of 24-inches square. The following chart contains the recommended design parameters:

Bearing Material	Minimum Embedment Depth of Footing (Inches)	Vertical Bearing (psf)	Coefficient of Friction	Passive Earth Pressure (pcf)	Maximum Earth Pressure (psf)
Approved Compacted Fill	12	2,000	0.30	300	4,000

Increases in the bearing value are allowable at a rate of 400 pounds-per-square-foot for each additional foot of footing width or depth to a maximum of 4,000 pounds-per-square-foot. For bearing calculations, the weight of the concrete in the footing may be neglected.

The bearing value shown above is for the total of dead and frequently applied live loads and may be increased by one-third for short duration loading, which includes the effects of wind or seismic forces. When combining passive and friction for lateral resistance, the passive component should be reduced by one-third.

All continuous footings should be reinforced with four #4 steel bars: two placed near the top, and two near the bottom of the footings. Footings should be cleaned of all loose soil, moistened, free of shrinkage cracks, and approved by the geologist prior to placing forms, steel, or concrete.

Foundation Settlement

Settlement of the foundation system is expected to occur on initial application of loading. A settlement of one-fourth to one-half of an inch may be anticipated. Differential settlement should not exceed one-fourth of an inch.

FLOOR SLABS

Floor slabs should be cast over approved compacted fill and reinforced with a minimum of #4 bars on 16-inch centers, each way. Prior to the placement of concrete slabs, the expansive soils encountered on the subject property shall be pre-moistened until the moisture content reaches at least 120 percent of the optimum moisture content to a depth of 12 inches. The pre-moistened subgrade soils should be tested 12 inches below grade, and verified to be 120 percent of optimum moisture content. Following the verification of moisture content, the polyethylene plastic, and sand shall be placed within one day.

EXTERIOR CONCRETE DECKS AND BLEACHERS

Decking and bleachers should be cast over approved compacted fill and should be reinforced with a minimum of #3 bars placed 18 inches on center, each way. Decking that caps a retaining wall should be provided with a flexible joint to allow for the normal one to two percent deflection of the retaining wall. Decking that does not cap a retaining wall should not be tied to the wall. The space between the wall and the deck will require periodic caulking to prevent moisture intrusion into the retaining wall backfill. The subgrade should be moistened prior to placing concrete.

DRAINAGE

Control of site drainage is important for the performance of the project. Pad and roof drainage should be collected and transferred to the street or an approved location in non-erosive drainage devices. Drainage should not be allowed to pond on the pad or against any foundation or retaining wall. Planters located within retaining wall backfill should be sealed to prevent moisture intrusion into the backfill. Drainage control devices require periodic cleaning, testing, and maintenance to remain effective.

ADDITIONAL GRADING

Fill that may be placed beyond the limits shown on the enclosed Compaction Map should be compacted with suitable equipment and observed by our representative. Byer Geotechnical, Inc., cannot be responsible for earth materials placed beyond the limits shown by test elevations on the enclosed Compaction Map. Fill placed below slabs, parkways, sidewalks, patios, driveways, parking lots, around footings, as retaining wall backfill, building wall backfill, garden wall backfill, and in utility trenches should be compacted. It is the responsibility of the contractor to place fill in accordance with the approved plans and specifications.

Byer Geotechnical appreciates the opportunity to provide our service on this project. Any questions concerning the data or interpretation of this report should be directed to the undersigned.

Respectfully submitted,
BYER GEOTECHNICAL, INC.

REGISTERED PROFESSIONAL ENGINEER
RAFFI S. BABAYAN
No. 72168
Exp. JUNE 30, 2012
CIVIL
STATE OF CALIFORNIA

Raffi S. Babayan
P. E. 72168

CK:RSB:mh
S:\FINAL\BG\21401_DWR_Construction\21401_DWR_Construction_Compaction_Report.wpd

Enc: Table I - Field Density Tests
Certificate of Compliance
Grading Permit # 11030 - 20000 - 05106
Compaction Map

xc: (1) Addressee (E-mail and Mail)
(4) DWR Construction, Attention: Mr. Anthony Damiano (E-mail)
(1) LADBS, Attention: Mr. Yervand Chapanyan (E-mail)

TABLE I

FIELD DENSITY TESTS

Test #	Date	Tech.	Location	Elevation (feet)	Moisture Content (%)	Dry Unit Weight (pcf)	Soil Type	Maximum Density (pcf)	Relative Compaction (%)
1	12-1-11	RD	Pool House	678.5	17.7	102.8	A	110.0	93
2	12-2-11	RD	Pool House	680.5	18.5	102.2	A	110.0	93
3	1-3-12	DJ	Swimming Pool	681.0	23.5	105.0	B	112.0	94
4	1-10-12	RD	Swimming Pool	681.0	20.2	101.8	A	110.0	93
5	1-10-12	RD	Swimming Pool	676.0	21.8	99.3	A	110.0	90
6	1-11-12	RD	Swimming Pool	683.0	19.8	103.1	A	110.0	94
7	1-11-12	RD	Swimming Pool	676.0	20.3	102.8	A	110.0	93
8	1-12-12	RD	Swimming Pool	678.0	19.6	104.0	A	110.0	95
9	1-12-12	RD	Swimming Pool	681.0	19.0	104.5	A	110.0	95
10	1-19-12	CK	Swimming Pool	680.0	23.5	103.6	A	110.0	94
11	1-19-12	CK	Swimming Pool	680.0	22.8	101.3	A	110.0	92
12	1-20-12	CK	Swimming Pool	682.5	24.2	104.1	A	110.0	95
13	1-20-12	CK	Swimming Pool	682.5	21.7	101.7	A	110.0	92
14	1-20-12	CK	Swimming Pool	679.0	23.4	102.5	A	110.0	93
15	1-20-12	CK	Swimming Pool	680.5	21.6	102.1	A	110.0	93
16	2-1-12	PK	Swimming Pool	676.0	20.5	104.0	A	110.0	95
17	2-1-12	PK	Swimming Pool	677.0	19.1	102.0	A	110.0	93
18	2-2-12	CK	Swimming Pool	679.0	20.3	103.5	A	110.0	94
19	2-6-12	DJ	Swimming Pool	682.5	18.3	100.1	A	110.0	91
20	2-23-12	DJ	Swimming Pool	680.0	20.9	103.7	B	112.0	93
21	2-24-12	CK	Swimming Pool	681.0	19.6	103.6	A	110.0	94
22	2-24-12	CK	Swimming Pool	683.0	18.5	105.2	A	110.0	96

BYER GEOTECHNICAL, INC.
BG 21401
ENGINEER'S CERTIFICATE OF COMPLIANCE
FOR
COMPACTED EARTH FILLS

LOCATION OF THE FILL:	Arb. 1, Lot 1111, Tract 1000 3700 North Coldwater Canyon Avenue Studio City, California
PROPERTY OWNER:	Harvard Westlake School 3700 Coldwater Canyon Avenue North Hollywood, California 91604
GRADING PERMIT:	# 11030 - 20000 - 05106

DATE WORK STARTED ON PROJECT:	November 30, 2011
DATE WORK WAS COMPLETED:	February 24, 2011
DATE OF THIS CERTIFICATION:	February 29, 2012

TO THE SUPERINTENDENT OF BUILDING:

*I hereby certify that I have personally inspected and tested the placement of compacted earth fill on the above described property, and on the basis of these inspections and tests, it is my opinion that the same was placed in conformity with the requirements of the Building Code of the City of Los Angeles.

REGISTERED PROFESSIONAL ENGINEER
RAFFI S. BABAYAN
No. 72168
Exp. JUNE 30, 2012
CIVIL
STATE OF CALIFORNIA

Raffi S. Babayan
P. E. 72168

*For the purpose of this Certificate, to "have personally inspected and tested" shall include inspection and testing performed by any person responsible to the licensed engineer signing this Certificate. Where the inspection and testing of all or part of the work above is delegated, full responsibility shall be assumed by the licensed engineer whose signature is affixed thereon.

Business and Professions Code, Ch.229, Sec.3. 6735.5. "The use of the word 'certify' or 'certification' by a registered professional engineer in the practice of professional engineering or land surveying constitutes an expression of professional opinion regarding those facts or findings which are the subject of the certification, and does not constitute a warranty or guarantee, either expressed or implied."

'00 N Coldwater Canyon Ave

Permit #: **11030 - 20000 - 05106**
Plan Check #: G11VN00026
Printed: 11/07/11 12:28 PM
Event Code:

rading **GREEN - MANDATORY**
ommercial
egular Plan Check
lan Check

City of Los Angeles - Department of Building and Safety

APPLICATION FOR GRADING PERMIT AND GRADING CERTIFICATE

Last Status: Ready to Issue
Status Date: 11/07/2011

TRACT	BLOCK	LOT(s)	ARB	COUNTY MAP REF #	PARCEL ID # (PIN #)	2. ASSESSOR PARCEL #
R 1000		1111	1	M B 19-34 (SHT 34)	162B161 397	2384 - 007 - 005

PARCEL INFORMATION

rea Planning Commission - South Valley	Cmpt. Fill Grd. - FG	Energy Zone - 9
ADBS Branch Office - VN	Certified Neighborhood Council - Studio City	Fire District - VHFHSZ
ouncil District - 2	Community Plan Area - Sherman Oaks-Studio City-Tolu	Hillside Grading Area - YES
mpt. Fill Grd. - CFG-1500	Census Tract - 1439.01	Hillside Ordinance - YES
mpt. Fill Grd. - CFG-3000	District Map - 162B161	Earthquake-Induced Landslide Area - Yes

ONES(S): RE15-1-H /

4. DOCUMENTS

ZA - ZA-16047	ZA - ZA-1999-93-PAD	HCM - LA-32	CPC - CPC-8123
ZA - ZA-1992-579-PAD	ZA - ZA-5448	CPC - CPC-18760	AFF - AFF-60586
ZA - ZA-1996-882-PAD	ORD - ORD-132416	CPC - CPC-2006-2375-PAD	AFF - OB-10459-A
ZA - ZA-1997-377-PAD	HLSAREA - Yes	CPC - CPC-24600	

5. CHECKLIST ITEMS

6. PROPERTY OWNER, TENANT, APPLICANT INFORMATION

Owner(s):
Harvard Westlake School 3700 Coldwater Canyon Ave N HOLLYWOOD CA 91604

Tenant:

Applicant: (Relationship: Agent for Owner)
Jim Dematt - Harvard Westlake School 3700 N. Coldwater Canyon Ave LA, CA 91604 (818) 512-4256

7. EXISTING USE

PROPOSED USE
(70) Grading - Hillside

8. DESCRIPTION OF WORK
EXCAVATION FOR PUBLIC SW. POOL, AND WALLS BACKFILL, AND GRADING PREP.

9. # Bldgs on Site & Use: SCHOOL BLDGS

For inspection requests, call toll-free **(888) LA4BUILD** (524-2845). Outside LA County, call (213) 482-0000 or request inspections via **www.ladbs.org.** To speak to a Call Center agent, call **311** or (866) 4LACITY (452-2489). Outside LA County, call (213) 473-3231.

10. APPLICATION PROCESSING INFORMATION

BLDG. PC By: Abdul Chegeni DAS PC By:
OK for Cashier: Barry Peshek Coord. OK:
Signature: Date:

For Cashier's Use Only **W/O #: 13005106**

1. PROJECT VALUATION & FEE INFORMATION Final Fee Period

'ermit Valuation: 2,100.00 cu yd PC Valuation:

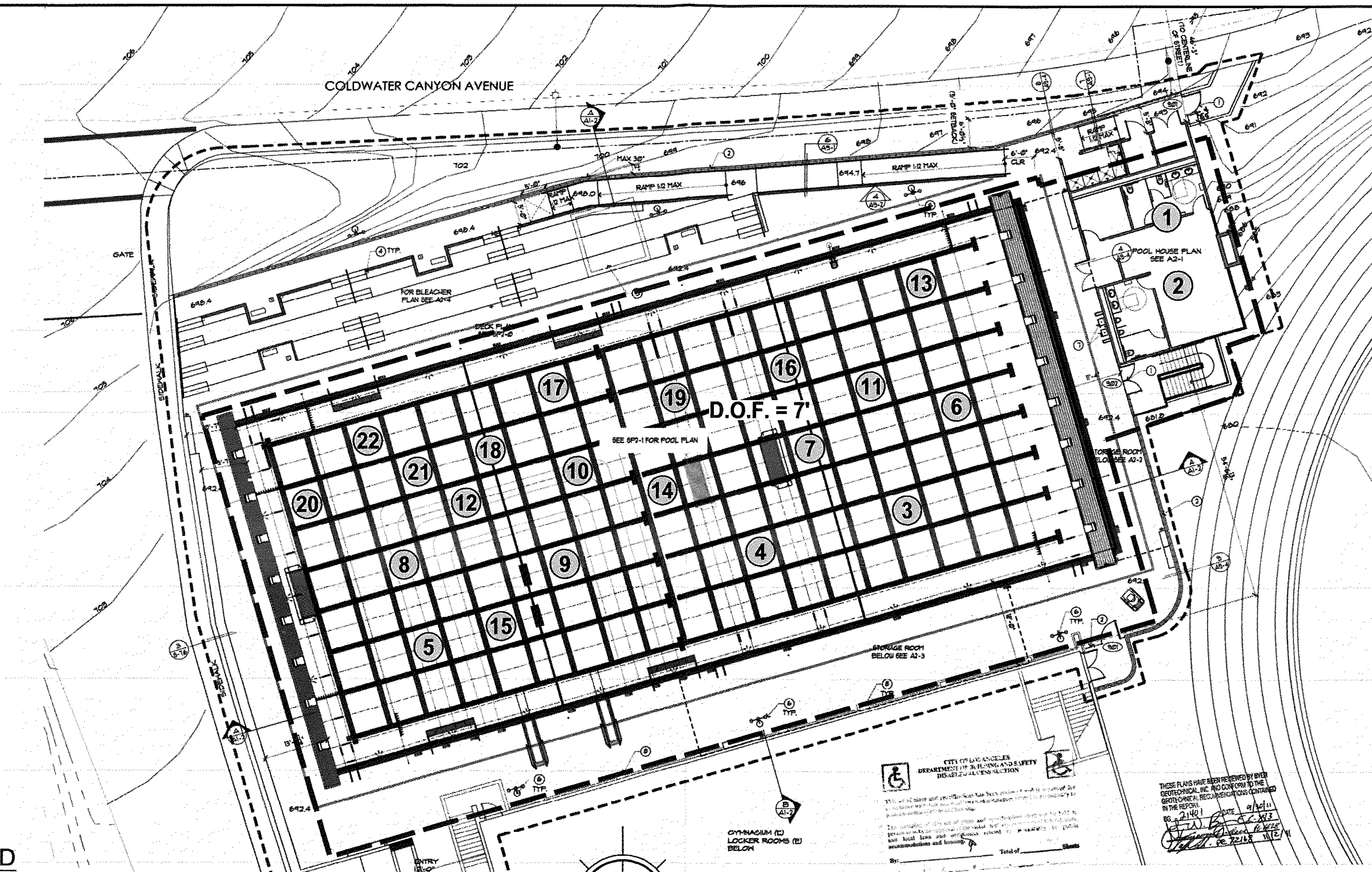
'INAL TOTAL Grading	3,090.73
'ermit Fee Subtotal Grading	1,842.50
'lan Check Subtotal Grading	0.00
)ff-hour Plan Check	753.75
'lan Maintenance	36.85
).S. Surcharge	52.66
;ys. Surcharge	157.99
'lanning Surcharge	157.99
'lanning Surcharge Misc Fee	10.00
'lanning Gen Plan Maint Surcharg	78.99
3reen Building	
'ermit Issuing Fee	0.00

11-23

ewer Cap ID: Total Bond(s) Due: $150,000

. ATTACHMENTS
lot Plan

* P 1 1 0 3 0 2 0 0 0 0 0 5 1 0 6 *



LEGEND

- LIMITS OF COMPACTED FILL
- (22) LOCATION AND NUMBER OF FIELD DENSITY TEST BY BYER GEOTECHNICAL, INC.
- SUBDRAIN w/OUTLET(S)
- D.O.F. DEPTH OF FILL (IN FEET)

REFERENCE MAP: SITE PLAN, SHEET A1-1, PREPARED BY ARCH PAC AQUATICS, DATED 11/02/2011.

FEBRUARY 29, 2012

BYER GEOTECHNICAL INC.
1461 E. CHEVY CHASE DR., SUITE 200
GLENDALE, CA 91206
818.549.9959 TEL
818.543.3747 FAX

COMPACTION MAP

BG: 21401 DWR CONSTRUCTION

ENGINEER: RSB | SCALE: 1" = 20'

LA DBS
DEPARTMENT OF BUILDING AND SAFETY

City of Los Angeles
COMPACTION REPORT APPROVAL LIST FOR PRIMARY STRUCTURAL FILL

LOG# 76604 DATE 3-8-12 COMPACTION FILE - 5

JOB ADDRESS 3700 Coldwater Canyon DISTRICT OFFICE VN

TRACT TR 1000 COUNTY REF. # MB 19-34 (SHT 34)

BLOCK PERMIT No. 11030 - 20000 - 05106

LOT 1111 ARB

FILL SOILS CLASSIFICATION, PER TABLE 1804.2 : Clayey Silt

REPORT PREPARED BY: BYBR Geotechnical DATED 2-29-12

REPORT #: BG 21401

OVERSIZED DOCUMENTS X-REF DATED

REVIEWED BY Y. Chapayan TELEPHONE (818) 374-4357

The compaction report(s) have been reviewed by the Grading Section of the Department and have been found to be acceptable provided the proposed construction complies with the conditions specified in this letter. The approval of the reports does not permit the violation of any section of the Building Code, or other local ordinance or state law.

NOTE: Numbers in parenthesis ( ) refer to Code sections in the 2008 edition of the Los Angeles City Building Code, Information Bulletin (P/BC).

INSTRUCTIONS

- All of the following listed and circled conditions shall apply: 2

CONDITIONS FOR PRIMARY STRUCTURAL FILL:

1. Compacted fill shall extend beyond the footings a minimum distance of 3 feet or a distance equal to the depth of the fill below the footings, whichever is greater. (7011.3)
2. Continuous footing bearing pressure shall not exceed a bearing value of 2000 psf at 12 inches minimum embedment into approved compacted fill with a minimum width of ____ inches.
3. Column/isolated footings bearing pressure shall not exceed a bearing value of ________ psf at ______ inches minimum embedment into approved compacted fill with a minimum width of ____ inches.
4. The soils engineer shall inspect the footing excavations to determine that they are founded in the recommended material before calling the department for footing inspection. (108.9 and 7008.2)
5. Slope erosion control, planting, and irrigation of fill slopes, and run off control are required. (91.7012 and 91.7013)
6. Interim report only.

BYER GEOTECHNICAL, INC.

September 12, 2012
BG 21401

DWR Construction, Inc.
3051 Bostonian Drive
Los Alamitos, California 90720

Attention: Mr. Douglas W. Roberts, President

Subject

Transmittal Letter - Final Compaction Report
Swimming Pool and Pool House Wall Backfills
Grading Permit # 11030 - 20000 - 05106
Arb. 1, Lot 1111, Tract 1000
3700 North Coldwater Canyon Avenue
Studio City, California

Dear Mr. Roberts:

Byer Geotechnical has completed our Final Compaction Report dated September 12, 2012, which describes the grading with respect to placing backfill behind the retaining walls of the swimming pool and pool house. The reviewing agency for this document is the City of Los Angeles, Department of Building and Safety (LADBS). The city requires a filing fee to accompany compaction reports. Copies of the report have been distributed as follows:

(1) Addressee (E-mail and Mail)
(1) DWR Construction, Attention: Mr. Anthony Damiano (E-mail)
(3) LADBS

Byer Geotechnical will file the report with the LADBS and request expedited handling. Any questions concerning the report should be directed to the undersigned. Byer Geotechnical appreciates the opportunity to offer our service on this project.

Very truly yours,
BYER GEOTECHNICAL, INC.

Raffi S. Babayan
Project Engineer

BYER GEOTECHNICAL, INC.

FINAL COMPACTION REPORT

SWIMMING POOL AND POOL HOUSE WALL BACKFILLS

GRADING PERMIT # 11030 - 20000 - 05106

ARB. 1, LOT 1111, TRACT 1000

3700 NORTH COLDWATER CANYON AVENUE

STUDIO CITY, CALIFORNIA

FOR DWR CONSTRUCTION, INC.

BYER GEOTECHNICAL, INC., PROJECT NUMBER BG 21401

SEPTEMBER 12, 2012

FINAL COMPACTION REPORT
SWIMMING POOL AND POOL HOUSE WALL BACKFILLS
GRADING PERMIT # 11030 - 20000 - 05106
ARB. 1, LOT 1111, TRACT 1000
3700 NORTH COLDWATER CANYON AVENUE
STUDIO CITY, CALIFORNIA
FOR DWR CONSTRUCTION, INC.
BYER GEOTECHNICAL, INC., PROJECT NUMBER BG 21401
SEPTEMBER 12, 2012

INTRODUCTION

This final report summarizes results of compaction testing and field observations performed during grading of a portion of the site. The purpose of the compaction testing was to determine that the grading specification on the plan and the requirements of the City of Los Angeles Building Code were met. The results of the compaction tests are shown on "Table I" and the test locations are plotted on the enclosed Compaction Map.

Field observations and compaction testing were coordinated with Anthony of DWR Construction, Inc.

PRIOR WORK

The following geotechnical reports were prepared for the project by Byer Geotechnical, Inc.:

> *Geologic and Soils Engineering Exploration Update. Proposed Pool, Pool House, and Retaining Wall, Harvard-Westlake School, Lot 1111, Tract 1000, 3700 Coldwater Canyon Avenue, Studio City, California,* dated September 20, 2011; and
>
> *Compaction Report, Proposed Swimming Pool and Pool House, Grading Permit # 11030 - 20000 - 05106, Arb. 1, Lot 1111, Tract 1000, 3700 North Coldwater Canyon Avenue, Studio City, California,* dated February 29, 2012.

The City of Los Angeles, Department of Building and Safety, reviewed the reports and issued the conditional approval letters, Log # 75188, dated October 28, 2011, and Log # 76604, dated March 8, 2012.

SOIL CLASSIFICATION

The following soil types were used in the compacted fill:

Soil Type	Soil Description	Soil Color	Maximum Dry Density (pcf)	Optimum Moisture Content (%)	Expansion Index*
A	Clayey Silt	Dark Gray-Brown	110.00	19.0	70 - Moderate
C (I)	Base	Gray	134.0	9.0	Nil
D (I)	Base	Tan	116.0	16.0	Nil

* Expansion Index as determined by Expansion Index Method (UBC Standard 29-2 or ASTM 4829-8a).

The maximum density tests were performed in accordance with ASTM D 1557-09.

PROJECT DESCRIPTION

The grading addressed in this report consisted of placing backfill behind the pool and pool house walls and around the surge tank, as shown on the enclosed Compaction Map. The approved compacted fill will be used as primary structural fill supporting the concrete bleachers and secondary structural fill supporting hardscape and decking. Grading of the swimming pool and pool house pads was previously conducted under the observation and testing of Byer Geotechnical, and was addressed in the referenced compaction report dated February 29, 2012.

GRADING

Areas to receive compacted fill were cleared of vegetation and debris. Prior to placing retaining wall backfill, the bottom was observed and approved by a representative of the soils engineer. A subdrain was placed at the base of the swimming pool walls. The subdrain consists of a four-inch-diameter perforated pipe (SDR 35/ASTM D 3034), surrounded by a minimum of one cubic foot of ¾-inch gravel. The subdrain outlets to a storm drain. The subdrain and outlet were observed and approved by a representative of the soils engineer. The locations of the subdrain and outlet are shown on the enclosed Compaction Map.

The lower three feet of the retained height behind the pool walls, measured from the base of the wall, was backfilled using a ¾-inch crushed gravel, due to the presence of several underground utility pipelines and the structural elements supporting the pool walls. Crushed gravel was densified in-place by vibration. The balance of the retained height was backfilled to the design finish grade using imported base material.

The retaining portion of the west wall of the pool house was backfilled using onsite excavated earth materials.

Compaction

Fill was placed by means of a Bobcat, in loose lifts of about six inches, moistened as required to achieve optimum moisture content by means of a water hose, and compacted with a mechanical compactor.

Field density tests were performed in accordance with ASTM D 1556-07. Field density tests as shown on "Table I" indicate that compacted fill behind the pool house was placed to a minimum of 90 percent of the maximum dry density. The compacted base material behind the swimming pool walls and surge tank was placed to a minimum of 95 percent of the maximum dry density.

The maximum vertical depth of fill is eight feet, located around the surge tank, as shown on the enclosed Compaction Map.

CONCLUSIONS AND RECOMMENDATIONS

Field density tests indicate that compacted fill was placed in a satisfactory manner and is suitable as retaining wall backfill and for support of concrete bleachers, hardscape, and decking. The grading was performed according to the approved plan prepared by Arch Pac Incorporated.

FLOOR SLABS

Floor slabs should be cast over approved compacted fill and reinforced with a minimum of #4 bars on 16-inch centers, each way. Prior to the placement of concrete slabs, the expansive soils encountered on the subject property shall be pre-moistened until the moisture content reaches at least 120 percent of the optimum moisture content to a depth of 12 inches. The pre-moistened subgrade soils should be tested 12 inches below grade, and verified to be 120 percent of optimum moisture content. Following the verification of moisture content, the polyethylene plastic, and sand shall be placed within one day.

EXTERIOR CONCRETE DECKS AND BLEACHERS

Decking and bleachers should be cast over approved compacted fill and should be reinforced with a minimum of #3 bars placed 18 inches on center, each way. Decking that caps a retaining wall should be provided with a flexible joint to allow for the normal one to two percent deflection of the retaining wall. Decking that does not cap a retaining wall should not be tied to the wall. The space between the wall and the deck will require periodic caulking to prevent moisture intrusion into the retaining wall backfill. The subgrade should be moistened prior to placing concrete.

DRAINAGE

Control of site drainage is important for the performance of the project. Pad and roof drainage should be collected and transferred to the street or an approved location in non-erosive drainage devices. Drainage should not be allowed to pond on the pad or against any foundation or retaining wall. Planters located within retaining wall backfill should be sealed to prevent moisture intrusion into the backfill. Drainage control devices require periodic cleaning, testing, and maintenance to remain effective.

ADDITIONAL GRADING

Fill that may be placed beyond the limits shown on the enclosed Compaction Map should be compacted with suitable equipment and observed by our representative. Byer Geotechnical, Inc., cannot be responsible for earth materials placed beyond the limits shown by test elevations on the enclosed Compaction Map. Fill placed below slabs, parkways, sidewalks, patios, driveways, parking lots, around footings, as retaining wall backfill, building wall backfill, garden wall backfill, and in utility trenches should be compacted. It is the responsibility of the contractor to place fill in accordance with the approved plans and specifications.

Byer Geotechnical appreciates the opportunity to provide our service on this project. Any questions concerning the data or interpretation of this report should be directed to the undersigned.

Respectfully submitted,
BYER GEOTECHNICAL, INC.

REGISTERED PROFESSIONAL ENGINEER
RAFFI S. BABAYAN
No. 72168
Exp. JUNE 30, 2014
CIVIL
STATE OF CALIFORNIA

Raffi S. Babayan
P. E. 72168

RSB:mh
S:\FINAL\BG\21401_DWR_Construction\21401_DWR_Construction_Compaction_Report_9.12.12.wpd

Enc: Table I - Field Density Tests
Certificate of Compliance
Grading Permit # 11030 - 20000 - 05106
Compaction Map

xc: (1) Addressee (E-mail and Mail)
(1) DWR Construction, Attention: Mr. Anthony Damiano (E-mail)
(3) LADBS (BG to Submit)

TABLE I

FIELD DENSITY TESTS

Test #	Date	Tech.	Location	Elevation (feet)	Moisture Content (%)	Dry Unit Weight (pcf)	Soil Type	Maximum Density (pcf)	Relative Compaction (%)
23	5-1-12	CK	Pool (Base)	683.0	8.7	129.3	C	134.0	96
24	5-2-12	CK	Pool (Base)	683.0	8.3	129.7	C	134.0	97
25	5-2-12	CK	Pool (Base)	683.0	9.4	128.2	C	134.0	96
26	6-28-12	JET	Pool House Wall Backfill	687.0	20.1	101.0	A	110.0	92
27	6-28-12	JET	Pool House Wall Backfill	690.0	21.0	100.3	A	110.0	91
28	7-12-12	CK	Pool Wall Backfill	689.0	15.6	112.3	D	116.0	97
29	7-13-12	CK	Pool Wall Backfill	691.0	14.8	111.2	D	116.0	96
30	7-13-12	CK	Pool Wall Backfill	691.4	15.1	110.9	D	116.0	96
31	7-13-12	CK	Pool Wall Backfill	690.0	13.8	111.6	D	116.0	96
32	7-16-12	CK	Pool Wall Backfill	690.0	15.3	111.1	D	116.0	96
33	7-17-12	CK	Pool Wall Backfill	682.4	13.6	110.3	D	116.0	95
34	7-17-12	CK	Pool Wall Backfill	684.4	14.8	112.7	D	116.0	97
35	7-17-12	CK	Pool Wall Backfill	687.4	14.3	111.6	D	116.0	96
36	7-17-12	CK	Pool Wall Backfill	686.4	13.9	110.0	D	116.0	95
37	7-18-12	CK	Pool Wall Backfill	689.0	14.2	110.4	D	116.0	95
38	7-19-12	CK	Pool Wall Backfill	691.0	13.7	112.1	D	116.0	97
39	7-19-12	CK	Pool Wall Backfill	693.0	13.3	111.7	D	116.0	96
40	7-23-12	CK	Pool Wall Backfill	694.0	13.2	111.7	D	116.0	96

Tests 1 - 22 can be found in the Compaction Report dated February 29, 2012.

BYER GEOTECHNICAL, INC.
BG 21401
ENGINEER'S CERTIFICATE OF COMPLIANCE
FOR
COMPACTED EARTH FILLS

LOCATION OF THE FILL:	Arb. 1, Lot 1111, Tract 1000 3700 North Coldwater Canyon Avenue Studio City, California
PROPERTY OWNER:	Harvard Westlake School 3700 Coldwater Canyon Avenue North Hollywood, California 91604
GRADING PERMIT:	# 11030 - 20000 - 05106

DATE WORK STARTED ON PROJECT:	May 1, 2012
DATE WORK WAS COMPLETED:	July 23, 2012
DATE OF THIS CERTIFICATION:	September 12, 2012

TO THE SUPERINTENDENT OF BUILDING:

*I hereby certify that I have personally inspected and tested the placement of compacted earth fill on the above described property, and on the basis of these inspections and tests, it is my opinion that the same was placed in conformity with the requirements of the Building Code of the City of Los Angeles.

REGISTERED PROFESSIONAL ENGINEER
RAFFI S. BABAYAN
No. 72168
Exp. JUNE 30, 2014
CIVIL
STATE OF CALIFORNIA

Raffi S. Babayan
P. E. 72168

*For the purpose of this Certificate, to "have personally inspected and tested" shall include inspection and testing performed by any person responsible to the licensed engineer signing this Certificate. Where the inspection and testing of all or part of the work above is delegated, full responsibility shall be assumed by the licensed engineer whose signature is affixed thereon.

Business and Professions Code, Ch.229, Sec.3. 6735.5. "The use of the word 'certify' or 'certification' by a registered professional engineer in the practice of professional engineering or land surveying constitutes an expression of professional opinion regarding those facts or findings which are the subject of the certification, and does not constitute a warranty or guarantee, either expressed or implied."

'00 N Coldwater Canyon Ave

Permit #: **11030 - 20000 - 05106**
~~Plan~~ Check #: G11VN00026
Event Code:
Printed: 11/07/11 12:28 PM

rading **GREEN - MANDATORY**
ommercial
egular Plan Check
lan Check

City of Los Angeles - Department of Building and Safety

APPLICATION FOR GRADING PERMIT AND GRADING CERTIFICATE

Last Status: Ready to Issue
Status Date: 11/07/2011

TRACT	BLOCK	LOT(s)	ARB	COUNTY MAP REF #	PARCEL ID # (PIN #)	2. ASSESSOR PARCEL #
R 1000		1111	1	M B 19-34 (SHT 34)	162B161 397	2384 - 007 - 005

PARCEL INFORMATION

rea Planning Commission - South Valley
ADBS Branch Office - VN
ouncil District - 2
mpt. Fill Grd. - CFG-1500
mpt. Fill Grd. - CFG-3000
Cmpt. Fill Grd. - FG
Certified Neighborhood Council - Studio City
Community Plan Area - Sherman Oaks-Studio City-Tolu
Census Tract - 1439.01
District Map - 162B161
Energy Zone - 9
Fire District - VHFHSZ
Hillside Grading Area - YES
Hillside Ordinance - YES
Earthquake-Induced Landslide Area - Yes

ONES(S): RE15-1-H /

4. DOCUMENTS

ZA - ZA-16047	ZA - ZA-1999-93-PAD	HCM - LA-32	CPC - CPC-8123
ZA - ZA-1992-579-PAD	ZA - ZA-5448	CPC - CPC-18760	AFF - AFF-60586
ZA - ZA-1996-882-PAD	ORD - ORD-132416	CPC - CPC-2006-2375-PAD	AFF - OB-10459-A
ZA - ZA-1997-377-PAD	HLSAREA - Yes	CPC - CPC-24600	

5. CHECKLIST ITEMS

6. PROPERTY OWNER, TENANT, APPLICANT INFORMATION

Owner(s):
Harvard Westlake School 3700 Coldwater Canyon Ave N HOLLYWOOD CA 91604

Tenant:

Applicant: (Relationship: Agent for Owner)
Jim Dematt - Harvard Westlake School 3700 N. Coldwater Canyon Ave LA, CA 91604 (818) 512-4256

7. EXISTING USE

PROPOSED USE
(70) Grading - Hillside

8. DESCRIPTION OF WORK
EXCAVATION FOR PUBLIC SW. POOL, AND WALLS BACKFILL, AND GRADING PREP.

9. # Bldgs on Site & Use: SCHOOL BLDGS

10. APPLICATION PROCESSING INFORMATION

BLDG. PC By: Abdul Chegeni
DAS PC By:
OK for Cashier: Barry Peshek
Coord. OK:
Signature:
Date:

For inspection requests, call toll-free **(888) LA4BUILD** (524-2845). Outside LA County, call (213) 482-0000 or request inspections via **www.ladbs.org**. To speak to a Call Center agent, call **311** or (866) 4LACITY (452-2489). Outside LA County, call (213) 473-3231.

For Cashier's Use Only **W/O #: 13005106**

1. PROJECT VALUATION & FEE INFORMATION Final Fee Period

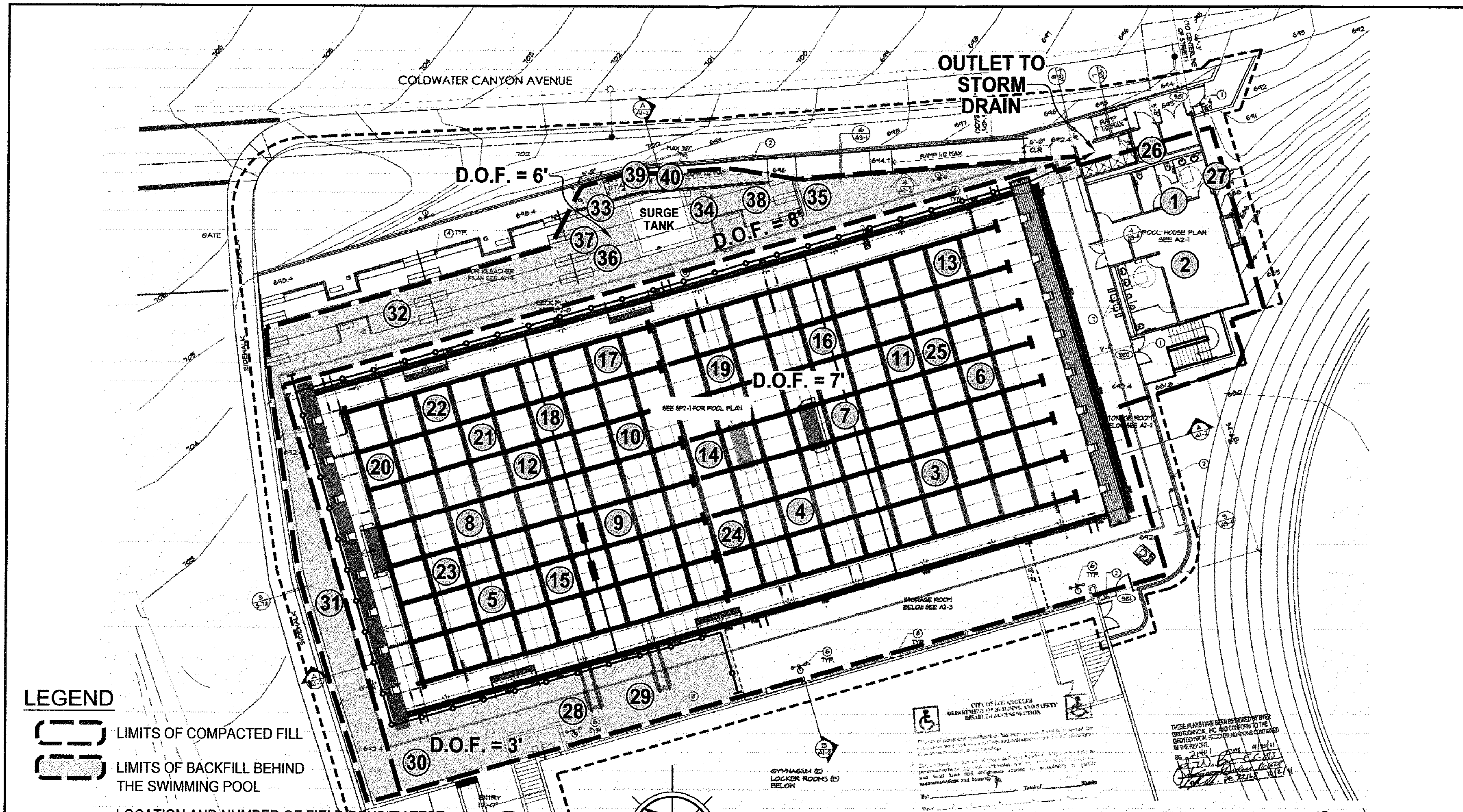
'ermit Valuation: 2,100.00 cu yd
PC Valuation:

'INAL TOTAL Grading	3,090.73
'ermit Fee Subtotal Grading	1,842.50
'lan Check Subtotal Grading	0.00
)ff-hour Plan Check	753.75
'lan Maintenance	36.85
).S. Surcharge	52.66
ys. Surcharge	157.99
'lanning Surcharge	157.99
'lanning Surcharge Misc Fee	10.00
'lanning Gen Plan Maint Surcharg	78.99
ireen Building	
'ermit Issuing Fee	0.00

ewer Cap ID:
Total Bond(s) Due: $150,000

. ATTACHMENTS
lot Plan

P11030200000005106



LEGEND

LIMITS OF COMPACTED FILL

LIMITS OF BACKFILL BEHIND THE SWIMMING POOL

1 - 22 LOCATION AND NUMBER OF FIELD DENSITY TEST (BYER GEOTECHNICAL, INC., 2/29/2012)

23 - 40 LOCATION AND NUMBER OF FIELD DENSITY TEST (THIS REPORT)

SUBDRAIN w/OUTLET(S)

D.O.F. DEPTH OF FILL (IN FEET)

REFERENCE MAP: SITE PLAN, SHEET A1-1, PREPARED BY ARCH PAC AQUATICS, DATED 11/02/2011.

SEPTEMBER 12, 2012

BYER GEOTECHNICAL INC.
1461 E. CHEVY CHASE DR., SUITE 200
GLENDALE, CA 91206
818.549.9959 TEL
818.543.3747 FAX

COMPACTION MAP			
BG: 21401 DWR CONSTRUCTION			
ENGINEER:	RSB	SCALE:	1" = 20'

CITY OF LOS ANGELES

CALIFORNIA

ANTONIO R. VILLARAIGOSA
MAYOR

DEPARTMENT OF
BUILDING AND SAFETY
201 NORTH FIGUEROA STREET
LOS ANGELES, CA 90012

ROBERT R. "BUD" OVROM
GENERAL MANAGER

RAYMOND S. CHAN,C.E., S.E.
EXECUTIVE OFFICER

9-20-12

LOG #: 78400

Harvard Westlake School
3700 Coldwater Canyon Av
Studio City, CA 91604

TRACT: 1000
BLOCK:
LOT: 1111

PERMIT No. 11030-20000-05106
DISTRICT MAP: 162B161
COUNTY REF.

LOCATION: 3700 Coldwater Canyon Av

SUBJECT: **SECONDARY STRUCTURAL FILL FOR HARDSCAPE**

LOTS HAVING COMPACTED FILL: same

Soils Compaction Report No. BG 21401, dated 9-12-12, prepared by Byer Geotechnical, Inc.

Approval is granted for compacted fill constructed on the above lot as described in the compaction report. Approval is limited to the area shown in the report and by the following conditions:

1. This fill may be used for the support of floor slabs and pavement. However, the fill is not approved for the support of structural footings.

2. Planting and irrigation of cut and fill slopes in hillside areas is required per Code Section 91.7012 of the Los Angeles City Building Code.

For compacted fill to be classified as structural fill, the soil testing laboratory responsible for controlling the placement of the fill must first, certify its placement and secondly, provide the allowable vertical and lateral bearing values which the fill can safely support. Where such values exceed those permitted in Table 18.1.A of the Los Angeles City Building Code, test data and calculations, including settlement calculations, shall be submitted for review.

GRADING INSPECTOR
Yervand Chapanyan
cc: Chapanyan-VN Office
Byer Geotechnical, Inc.

NOTE: Grading oversized document is not attached. (Document Type 92)

GEOTECHNICAL
PROFESSIONALS INC.

February 5, 2013
(Revised February 6, 2013)

Innovative Design Group
17848 Sky Park Circle, Suite D
Irvine, California 92614

Attention: Mr. Steve Kuhn

Subject: Update Letter
Geotechnical Investigation
Proposed Parking Structure
Harvard-Westlake School
3700 Coldwater Canyon Avenue
Los Angeles, California
GPI Project No. 2270.C

Dear Mr. Kuhn:

This letter updates our preliminary geotechnical investigation report (Reference 1) dated July 27, 2010 for the parking structure planned at Harvard-Westlake School in Los Angeles, California. Reference 1 addressed a 4 level (3 suspended decks) parking structure that encroaches into an ascending slope adjacent to Coldwater Canyon Avenue.

We understand that the design level grading and structural plans have yet to be completed for the design of the parking structure and surrounding slopes. After completion of the geometry of the final design, we recommend a grading plan review be performed to include our final geotechnical design recommendations.

The recommendations contained in our geotechnical investigation report (Reference 1) remain applicable for new parking structure proposed for the site except as follows:

- We assume the seismic design of the proposed development will be in accordance with the California Building Code, 2010 edition. For the 2010 CBC, a Soil Class C may be used.

- Based on the USGS website (Reference 2), we computed that the site could be subject to a peak ground acceleration of 0.40g. This acceleration has been computed using 40 percent of the short period design spectral acceleration, S_{DS}, for the project.

- A seismic increment of 12H in pounds per square foot (where H is equal to the height of the wall) may be added to the static lateral earth pressures to estimate seismic loading. The seismic earth pressure was estimated using the Mononobe-

5736 Corporate Avenue • Cypress, CA 90630 • (714) 220-2211, FAX (714) 220-2122

Okabe method and a pseudo-static coefficient of 0.15g, which is approximately one-third of the design acceleration.

We trust this information satisfies the requirements of the design team and the City of Los Angeles to update our previous report for this project.

Please do not hesitate to call if you have any questions on the contents of this letter.

Respectfully submitted,
Geotechnical Professionals Inc.

James E. Harris, G.E.
Principal

REGISTERED PROFESSIONAL ENGINEER
JAMES E. HARRIS, IV
No. GE 2100
Exp. 12-31-13
GEOTECHNICAL
STATE OF CALIFORNIA

FEB - 6 2013

JEH:sph

Enclosures: References

Distribution: (1) Addressee
(4) Mr. Michael Nytzen, Paul Hastings LLP

REFERENCES

1. Geotechnical Professionals, Inc., "Preliminary Geotechnical Investigation, Proposed Parking Structure, Harvard-Westlake School, 3700 Coldwater Canyon Avenue, Los Angeles, California," GPI Project No. 2270.I, dated July 27, 2010.

2. United States Geological Survey, "Seismic Design Values for Buildings, Seismic Hazard Mapping, Research and Monitoring, Website Address: http://earthquake.usgs.gov/research/hazmaps/design/index.php