Appendix E.1 Preliminary Geotechnical Report

GPI 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-

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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

JEH:sph

FEB - 6 2013

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

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

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Project No. 2270.I

July 27, 2010

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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		**Portland Cement Concrete**	
Auto Parking Stalls	4	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.

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Exp 9-30-14

7/13/12

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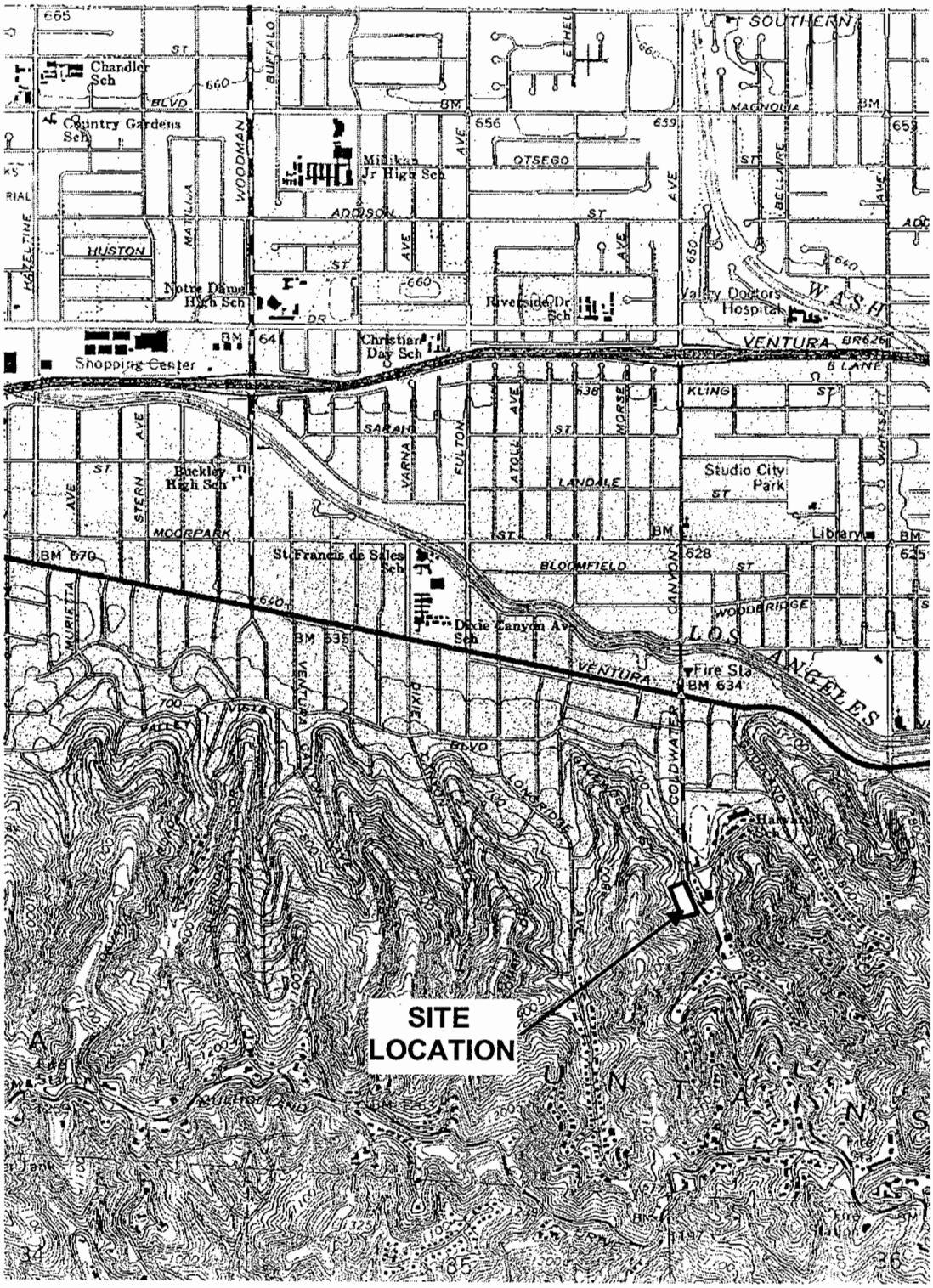


N

0 2000 4000 FEET

BASE MAP REPRODUCED FROM VAN NUYS QUADRANGLE FROM USGS MAPS

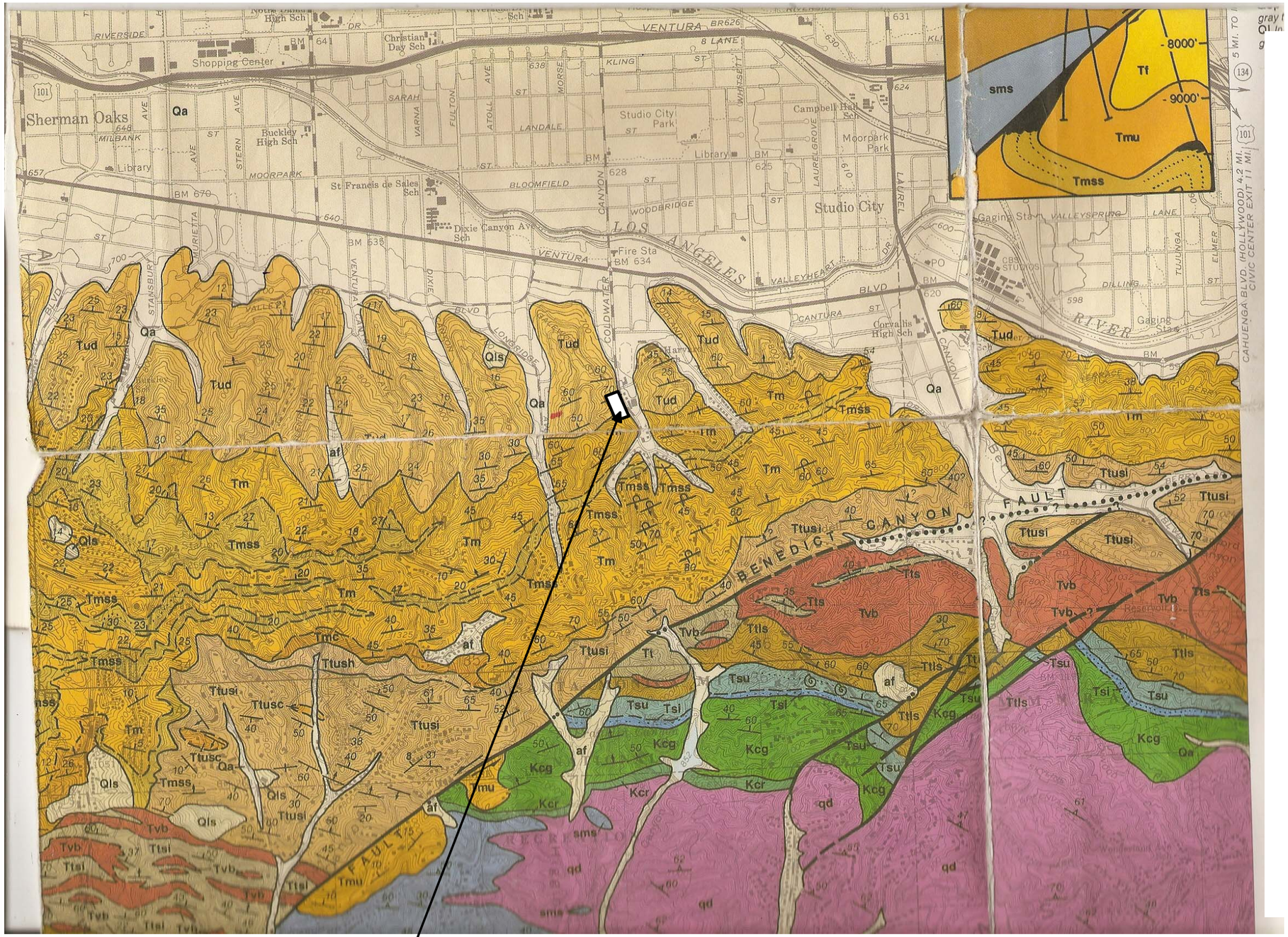
GPI GEOTECHNICAL PROFESSIONALS, INC.

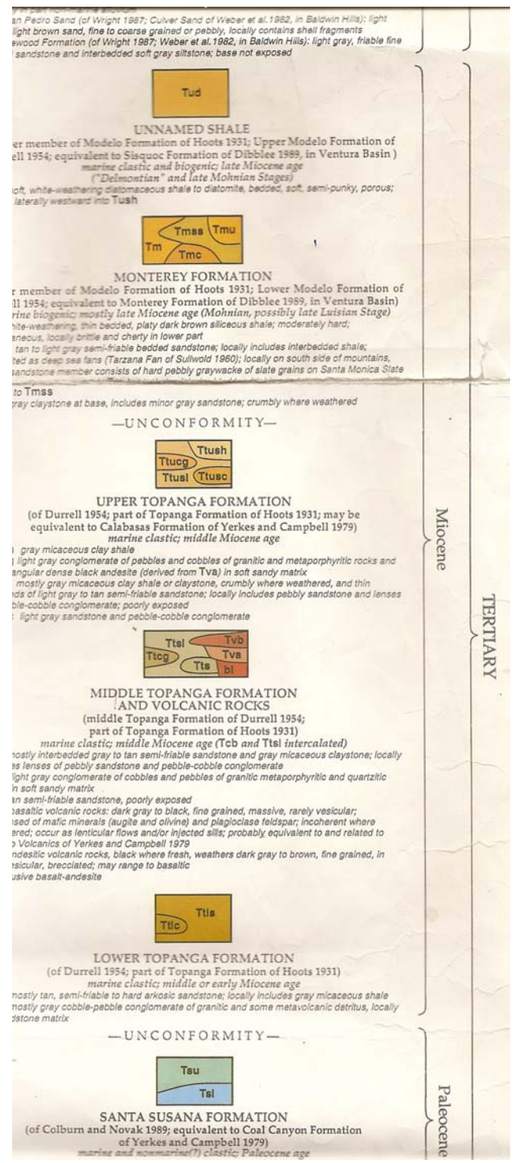
HARVARD WESTLAKE

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

SITE LOCATION

FIGURE 1





SITE LOCATION

0 1500 3000 FEET

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

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)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
				0	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	760
18.4	61	3	D	5	**SANDY SILT (ML)** brown, very moist, very stiff	
					@ 6' and 10'-6", thick gravel beds of shale fragments, irregular	755
19.3	65	3	D	10		
					Monterey Formation:	
					SILTSTONE gray to light brown, very moist, hard, highly weathered, fractured, diatomaceous shale	750
					No continuous or coherent bedding	
25.7	82	9	D	15	@ 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	745
					J: N10E, 44SE	
					Gypsum filled joints at 6"-12" spacing	
37.7	72	11	D	20	@ 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	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:	
					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:	
32.7	77	10	D	25	**SILTSTONE** gray to light brown, very moist, hard, high weathered with soil pockets, no continuous bedding	
					@ 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	

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)
				40	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.	
					@ 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
				50	@ 49 feet, B71E, 72NW	
					@ 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	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.	
					Fill:	
					SILT (ML) yellow brown and white, diatomaceous silt, with shale debris	
					Monterey Formation:	760
46.2	64	8	D	5	**SILTSTONE** gray to light brown, very moist, hard, moderately fractured, diatomaceous shale	
					@ 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		
					@ 33 feet, B: N70E, 73NW	730
44.4	71	11	D	35		
					@ 36 feet, J: N8W, 58NE	
					B: N70E, 71NW	
						725

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)
				0	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.	
					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
48.3	62	4	D	10	@ 8 feet, B: N61E, 69NW @ 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	710
43.0	74	25/6"	D	50	@ 49 feet, B: N70E, 68NW	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)
				0	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.	
					Fill: **SILT (ML)** brown, dry, soft, horizontal contact with soil, with 3/4" crushed gravel	760
16.1	74	4	D	5	Residual Soil/Colluvium: **CLAYEY SILT (ML)/SILTY CLAY (CL)** dark brown, moist, firm, porous, with 10%-20% shale fragments, with roots to 1/2" diameter	755
58.4	49	12	D	10	Monterey Formation: **SILTSTONE** whitish and yellow brown, very moist, hard, diatomaceous shale, very few fractures @ 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)
				0	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.	740
					Monterey Formation:	
					SILTSTONE grey and yellowish white, moist to very moist, hard, diatomaceous shale, laminated to thin bedded, few fractures	
					@ 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)
				0	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.	720
					AC Pavement	
					Fill:	
					CLAYEY SILT (ML) brown, slightly moist, firm, white shale fragments	
					Alluvium/Colluvium:	
18.5	55	2	D	5	**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
					Monterey Formation:	
43.9	69	3	D	20	**SILTSTONE** grey to light brown, very moist, very stiff, highly weathered and fractured shale, no continuous bedding	700
					@ 23 feet, grey to light brown, diatomaceous shale, hard B: N80E, 45NW	
48.4	71	8	D	25	@ 25 feet, B: N85E, 44NW	695
					@ 27.5 feet, B: N75E, 72NW hard, coherent shale	
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.: 2270.1
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)
					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:	720
					CLAYEY SILT (ML) yellow brown, dry, soft, with diatomaceous, shale fragments, roots to 1" diameter, horizontal lower contact	
					Monterey Formation:	
90.7	43	6/7"	D	5	**BEDROCK** white-yellow brown, very moist, hard, diatomaceous shale, thin bedded, few tight fractures/joints	715
					@ 3 feet, B: N53E, 73NW	
					@ 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	

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:	710
					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	
				10	less fractured	705
					@ 9 feet, discontinuous shear, paper thin	
					S: N33W, 31NE	
					@ 10 feet, B: N86E, 78SE	
					Hard, few fractures	
					@ 13 feet, B: N81W, 79SW	
				15	very hard, shaly siltstone	700
					@ 15 feet, B: EW, 79S	
					hard, diatomaceous shale, very few irregular fractures	
				20	@ 19 feet, B: EW, 84S	695
					J: N10W, 88SW	
					@ 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	690
					@ 25.5 feet, B: N88E, 84SE	
					J: N7W, 78NE	
					@ 26 to 30 feet, very hard, dark grey, unoxidized shaly siltstone	
				30	Total Depth 30 feet	685
					No samples collected	
					Backfilled with cuttings	

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	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	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	735
					@ 6 feet, as above, light yellowish brown, shaly siltstone, diatomaceous in part B: N79E, 57SE J: N70W, 56NE	
				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
					@ 19 feet, B: N84E, 63SE hard diatomaceous shale, little fractures J: N80W, 45NE	
				20	@ 22 feet, B: N83E, 68SE	720
				25	@ 24.5 feet, 1/2" wide shear zone, disrupts bedding S/F: N15W, 78NE	715
					@ 26 feet, B: N87W, 78SW	
				30	@ 30 feet, hard grey shale, unfractured B: 84E, 82SE	710
					@ 32 feet, B: EW, 83S, hard (tight) @ 32 to 35 feet, patches of dark grey, unoxidized shale	
				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

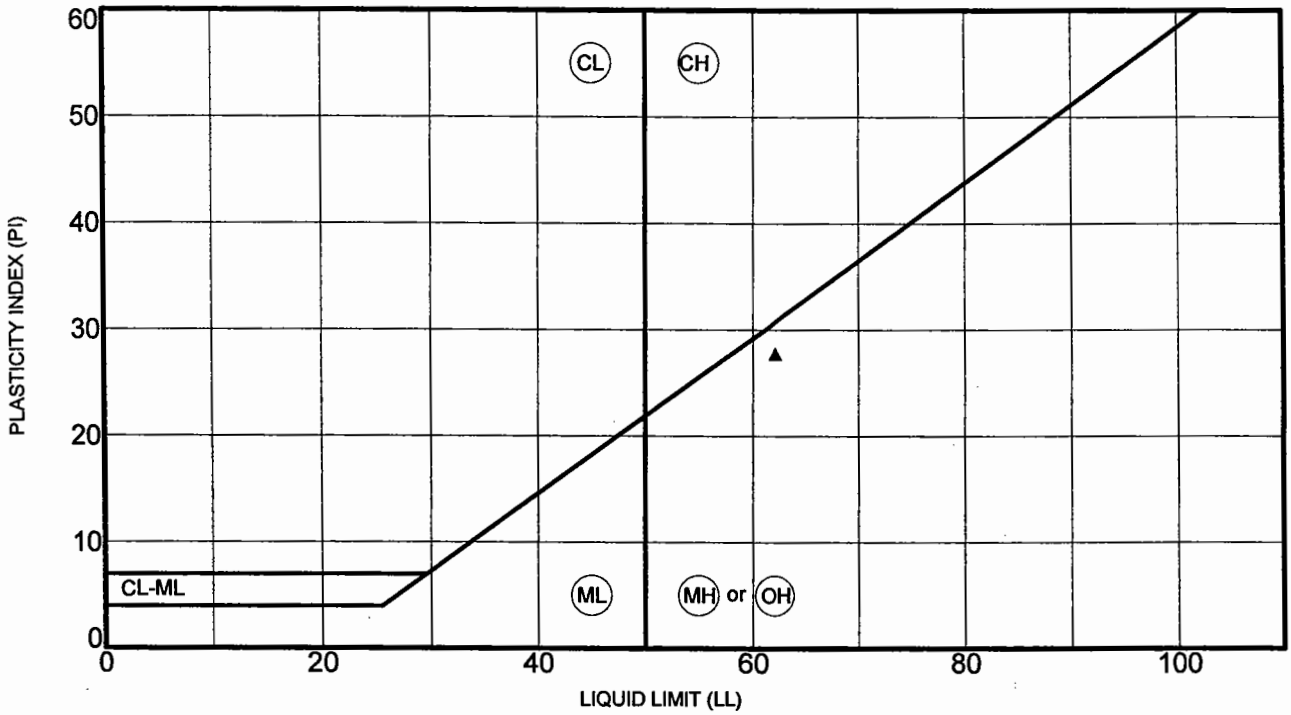
COMPACTION TEST

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

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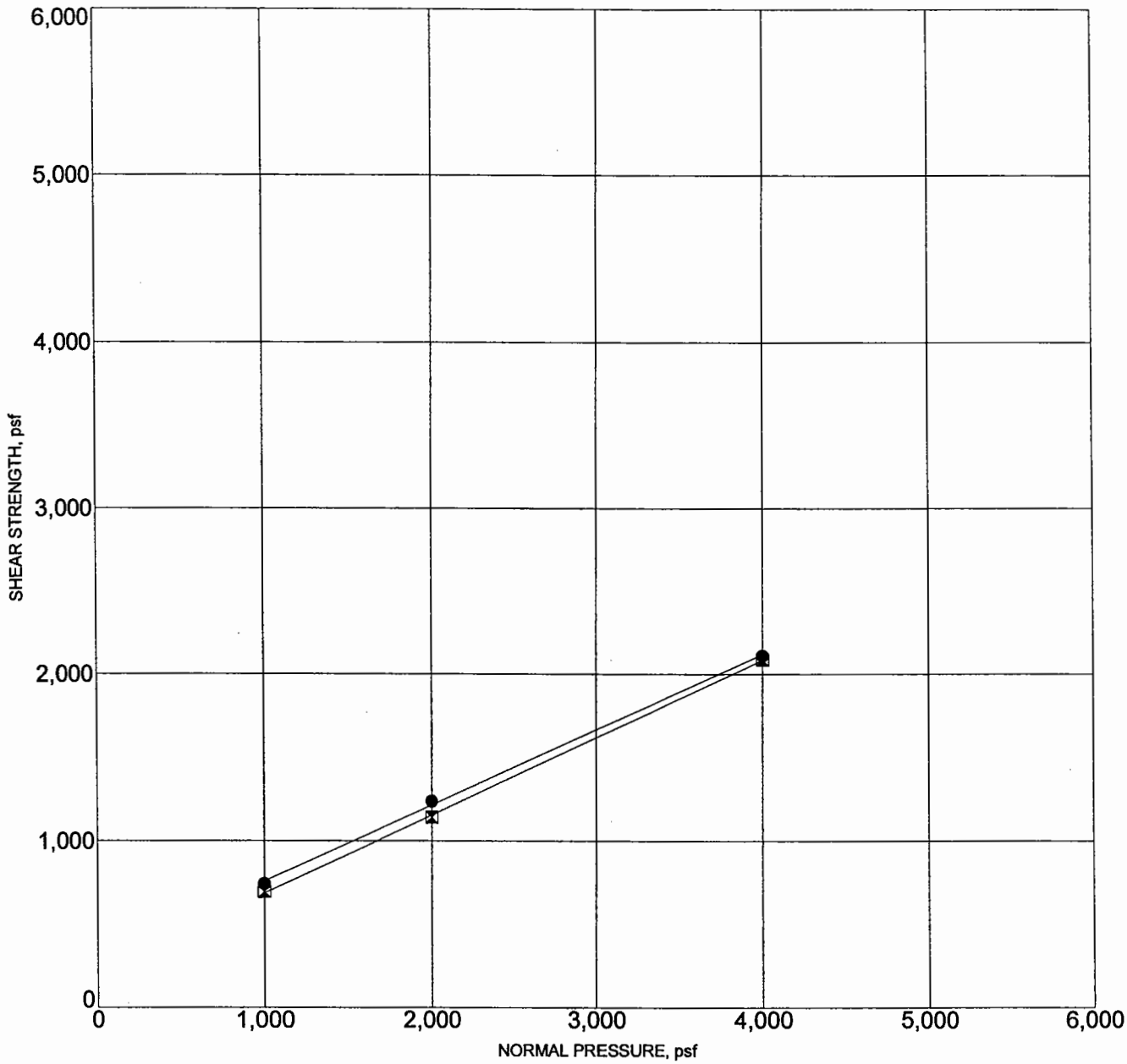
ATTERBERG LIMITS TEST RESULTS

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

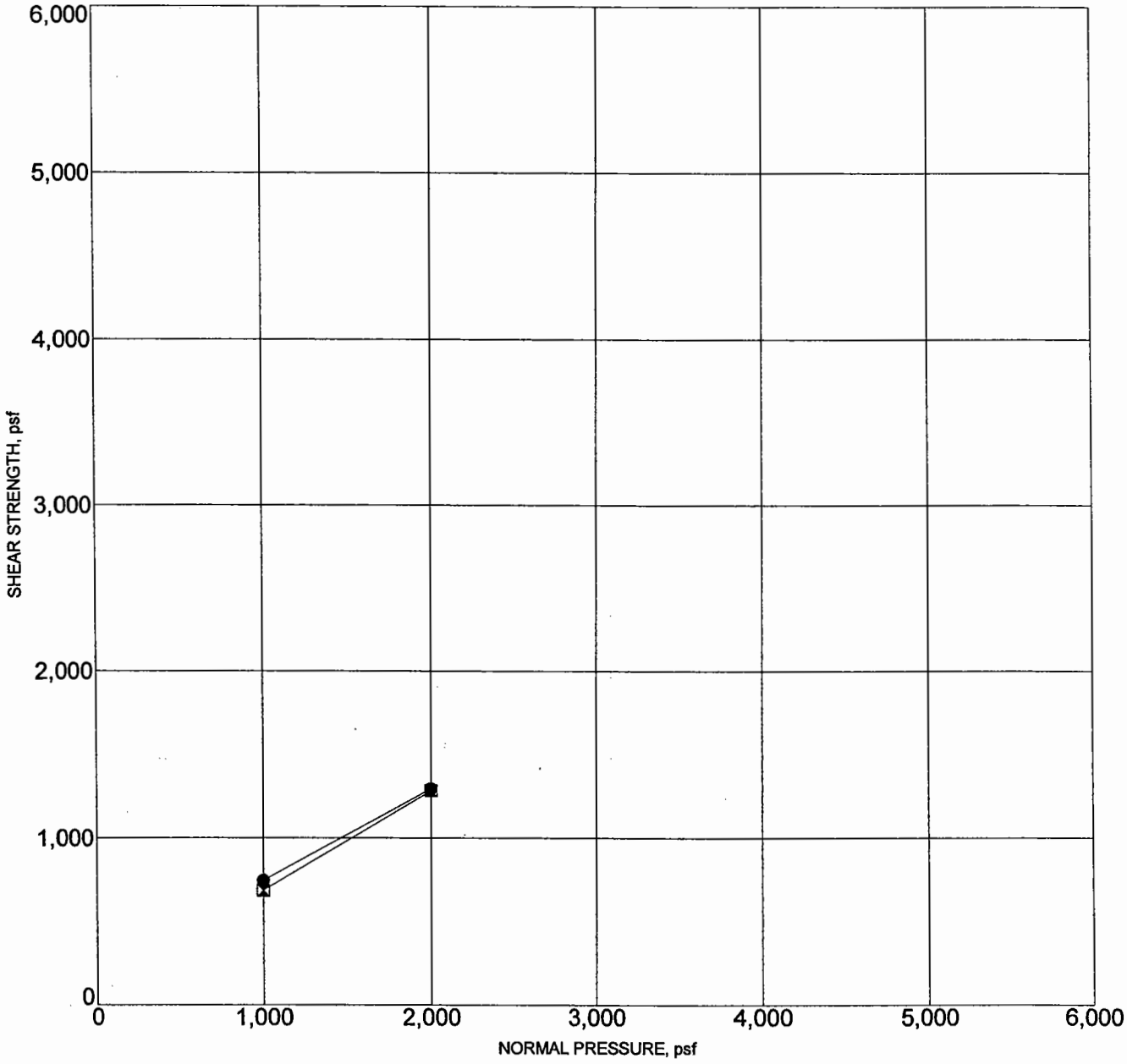
Sample Location		Classification	DD,pcf	MC,%
B-2	10-15	SANDY SILT (MH)	81	29.0

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

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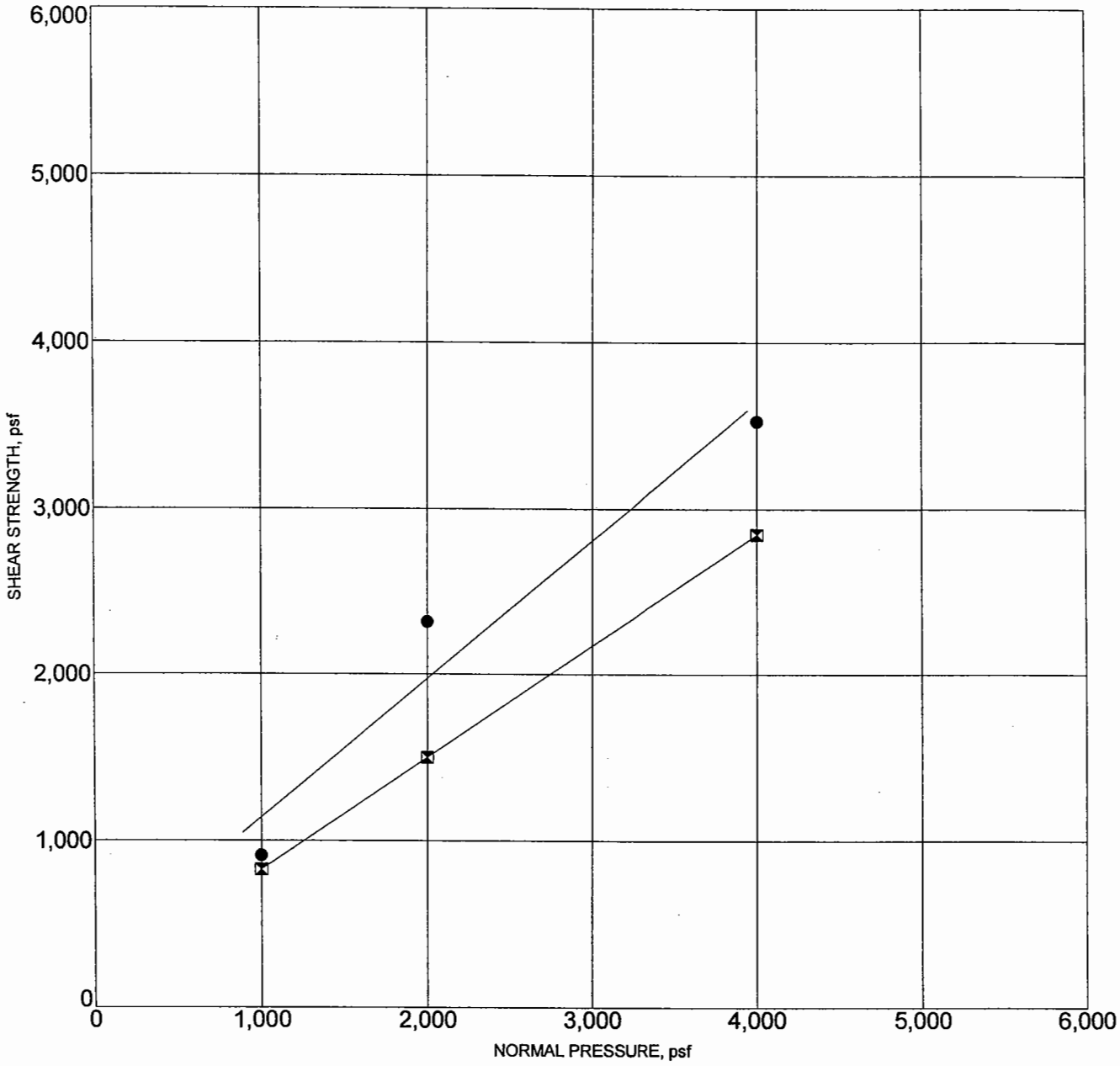
DIRECT SHEAR TEST RESULTS

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.: 2270.1

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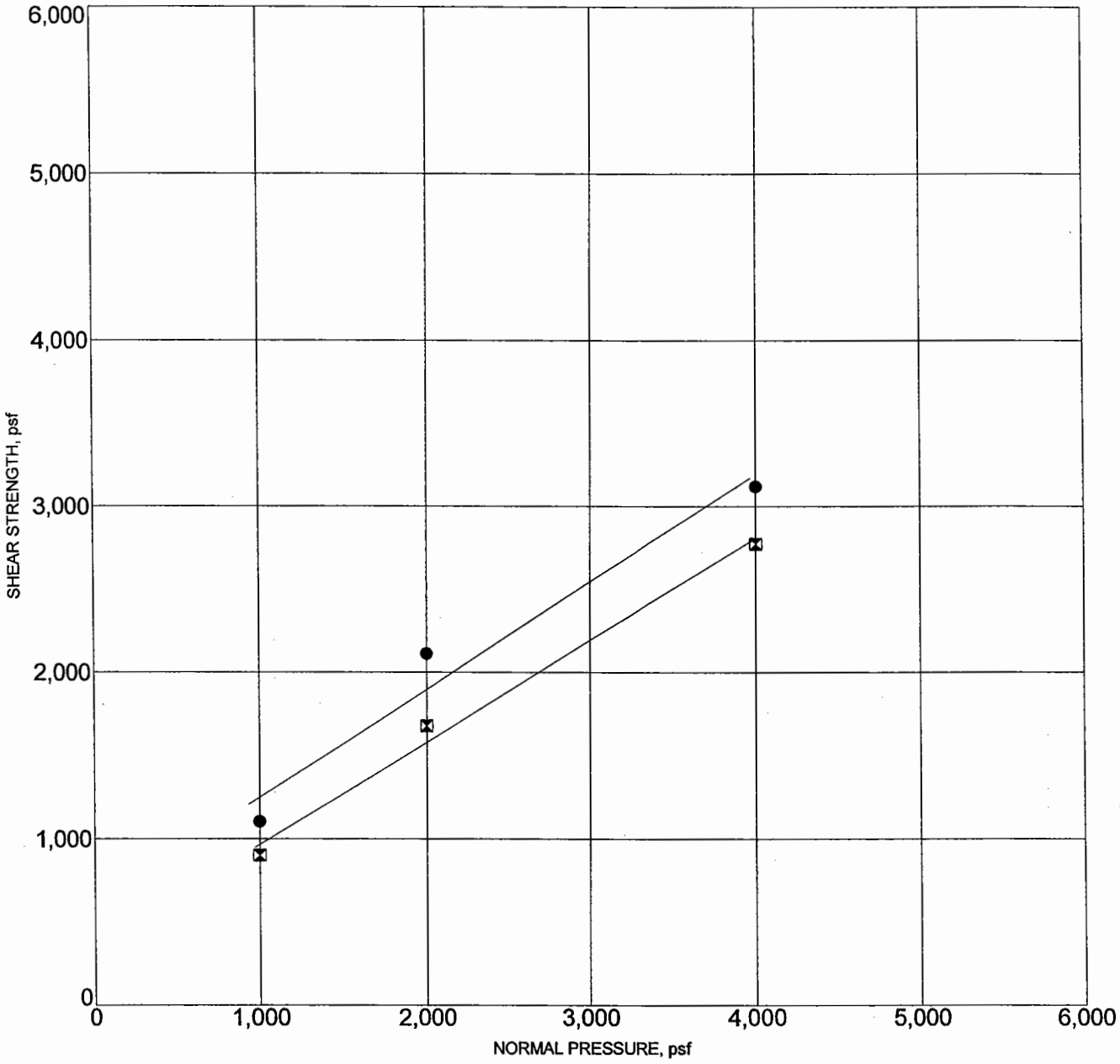
DIRECT SHEAR TEST RESULTS

FIGURE B-4



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

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

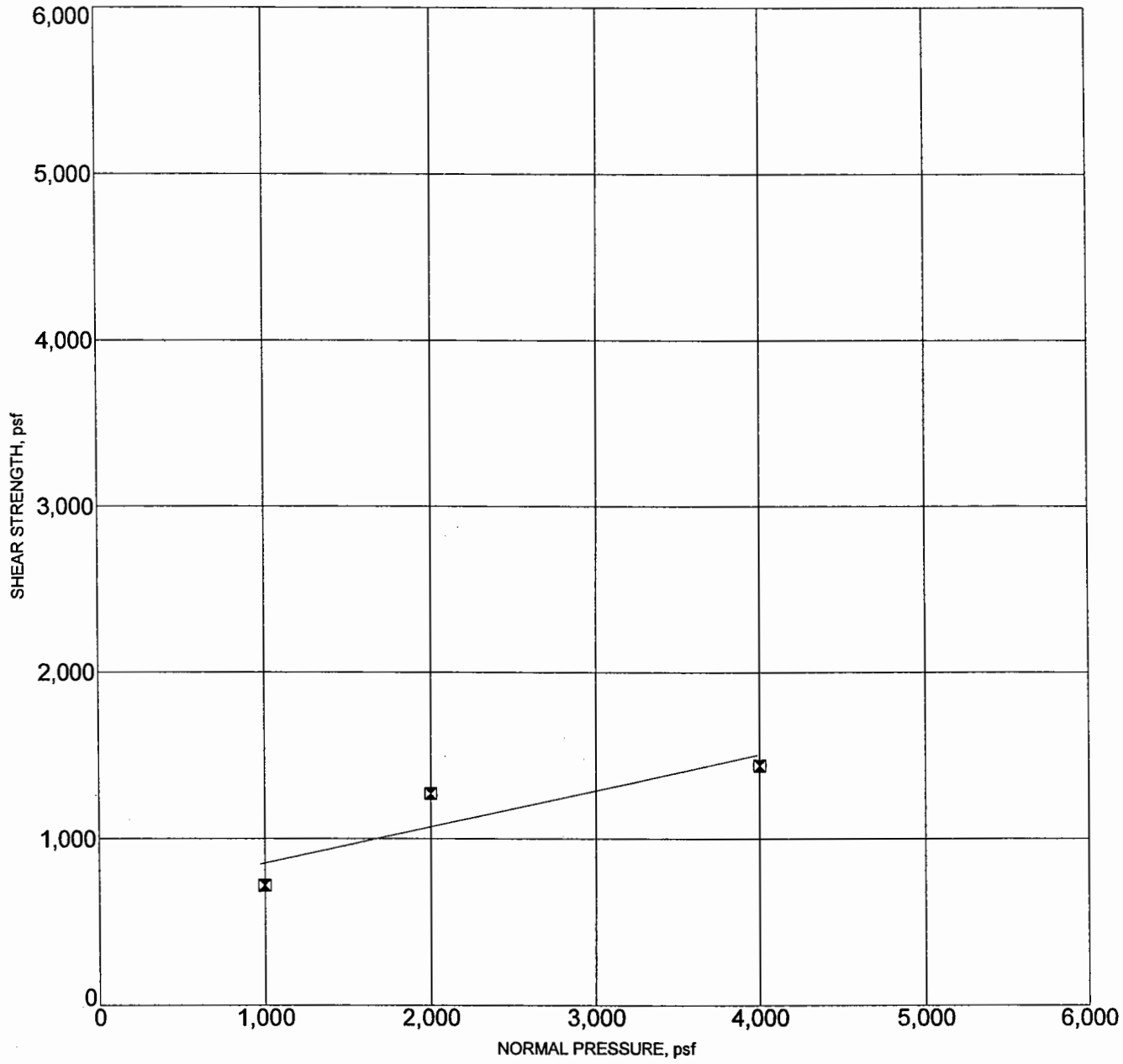
Sample Location		Classification	DD,pcf	MC,%
B-3	35.0	SILTSTONE	71	44.4

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

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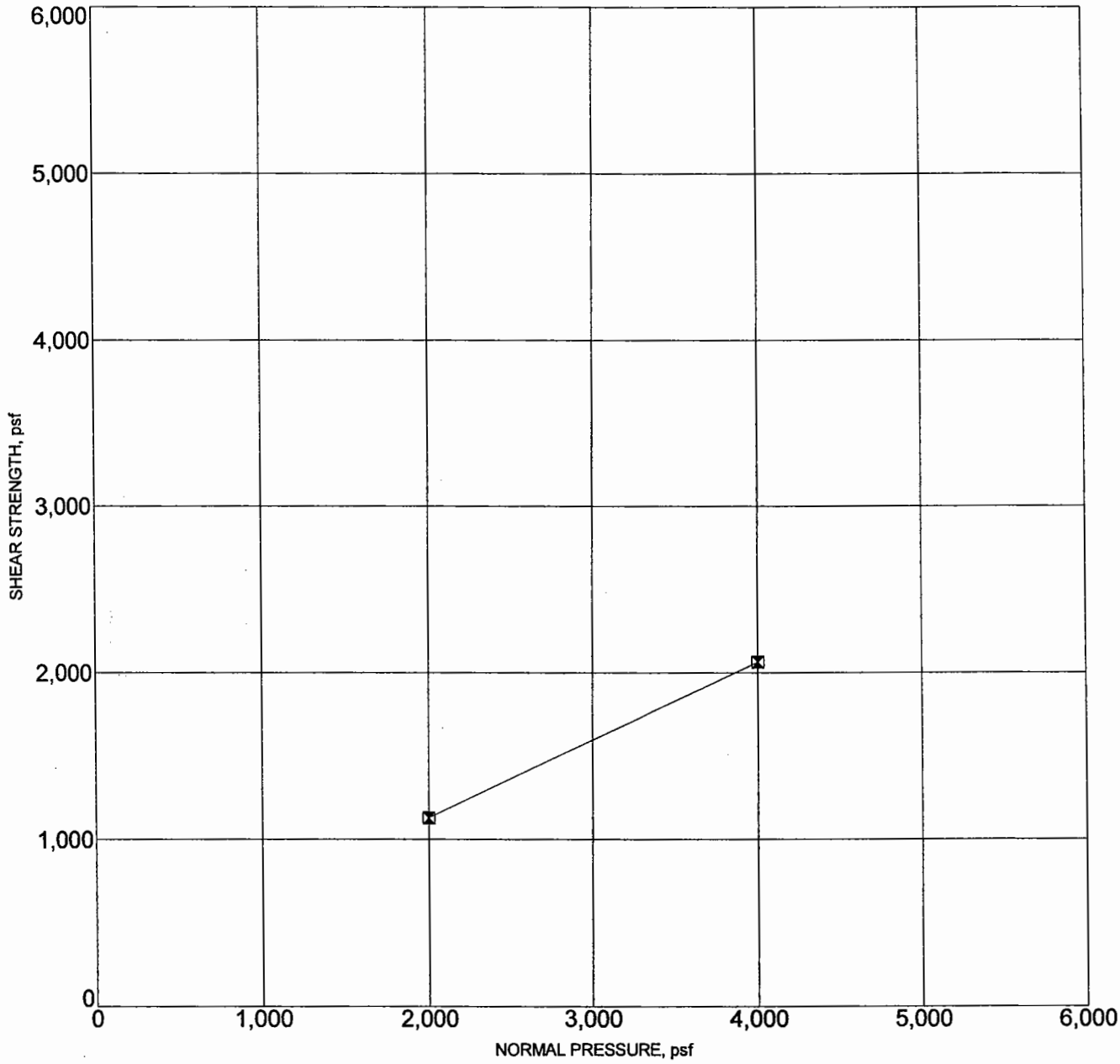
DIRECT SHEAR TEST RESULTS

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

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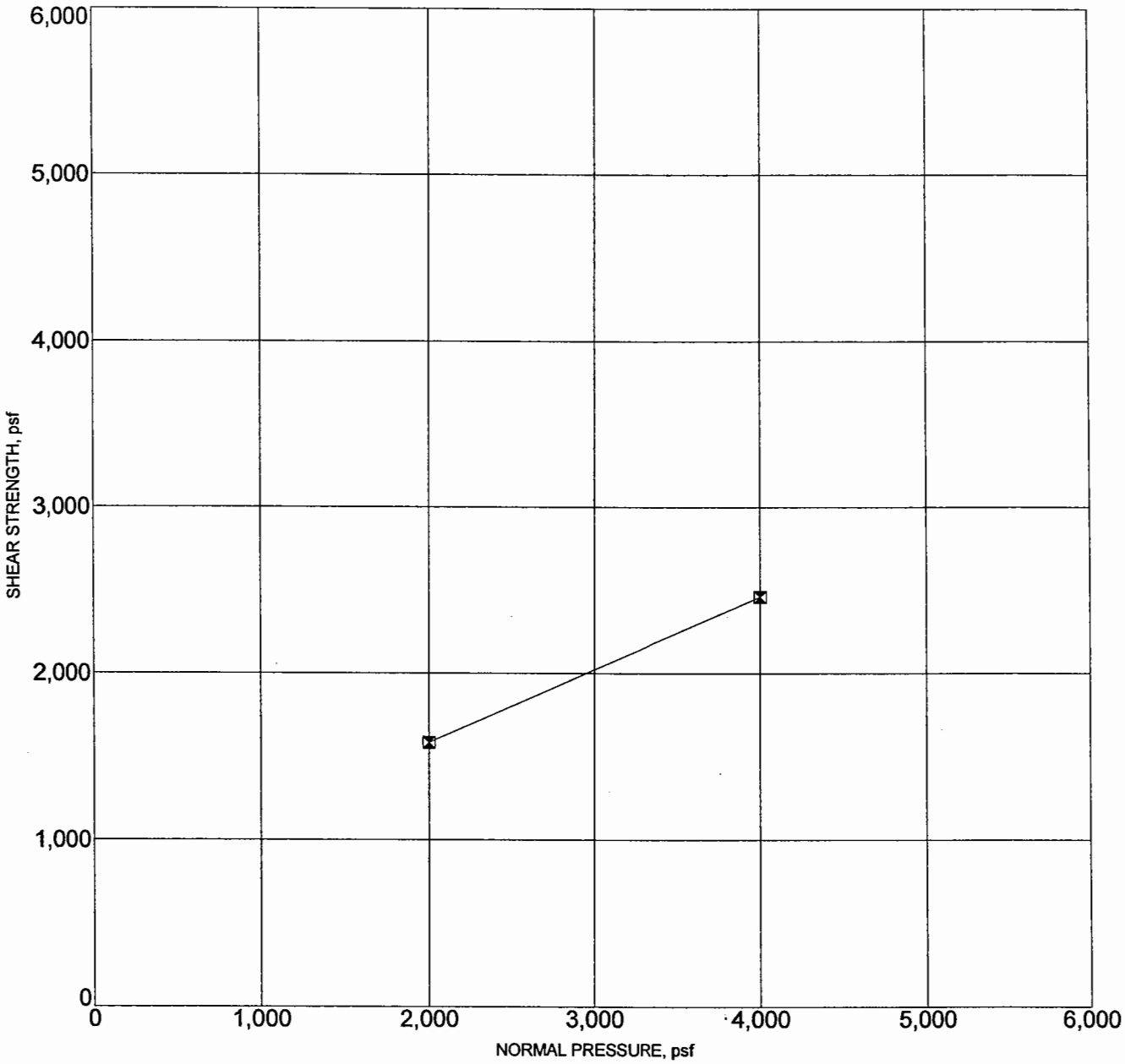
DIRECT SHEAR TEST RESULTS

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

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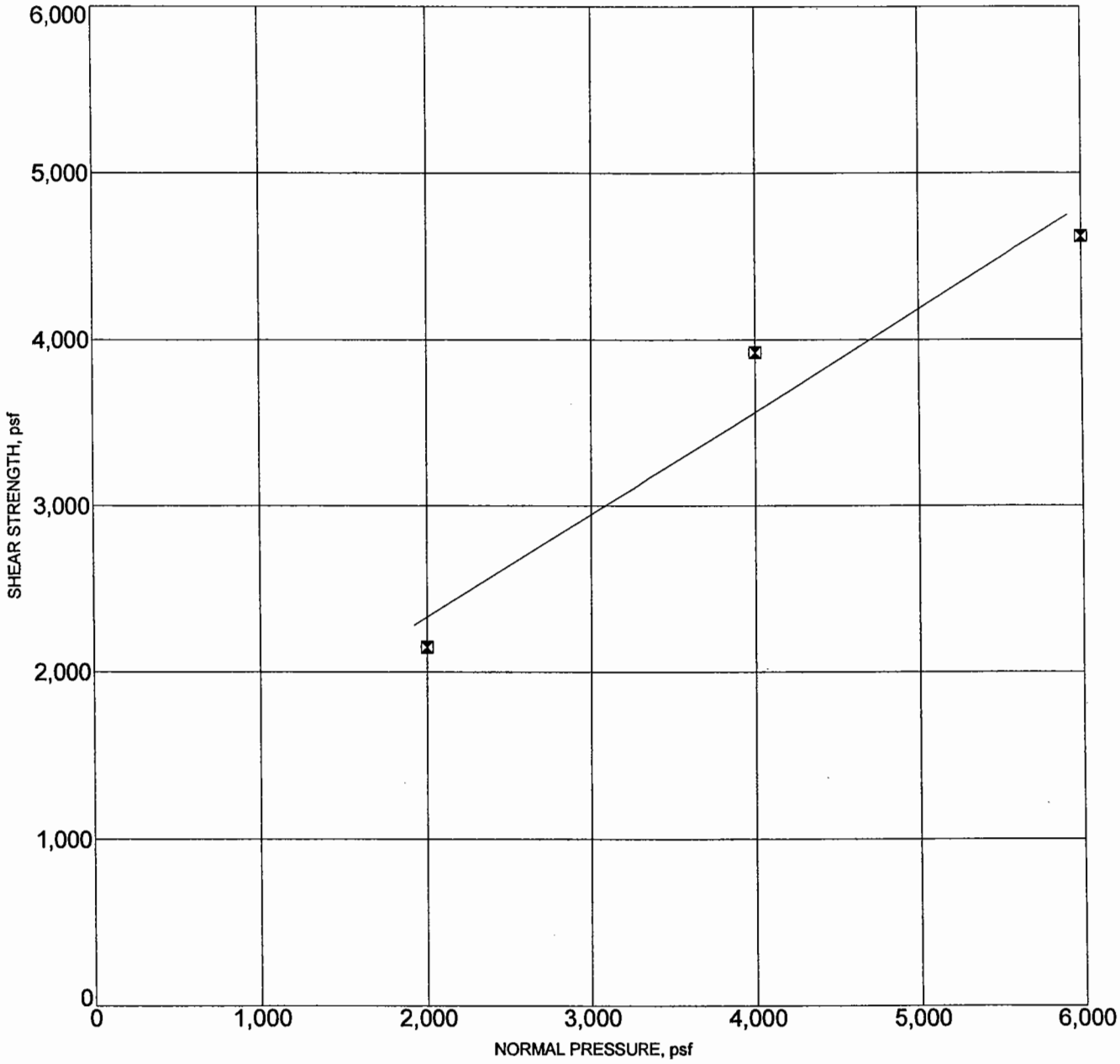
DIRECT SHEAR TEST RESULTS

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

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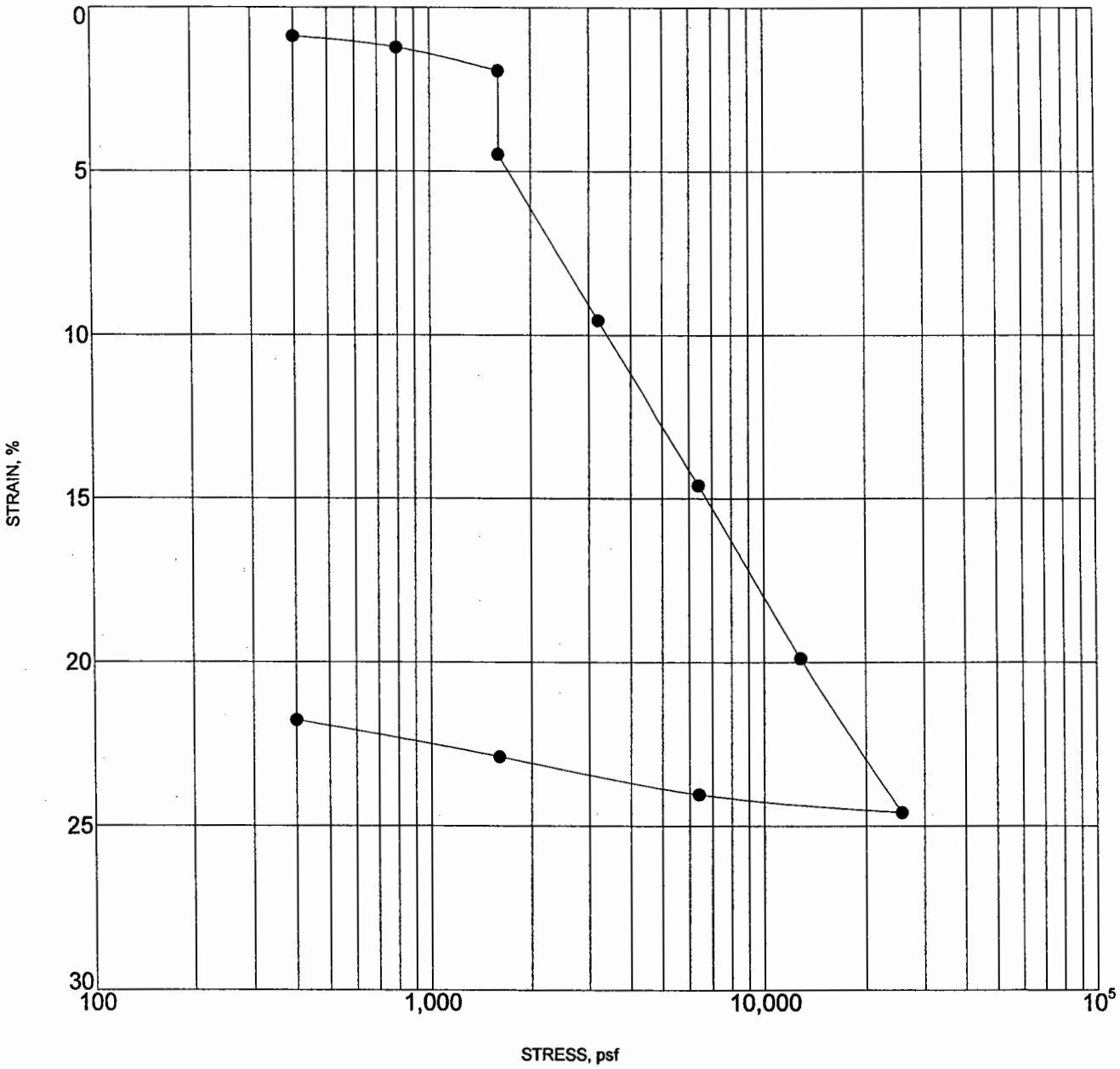
DIRECT SHEAR TEST RESULTS

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

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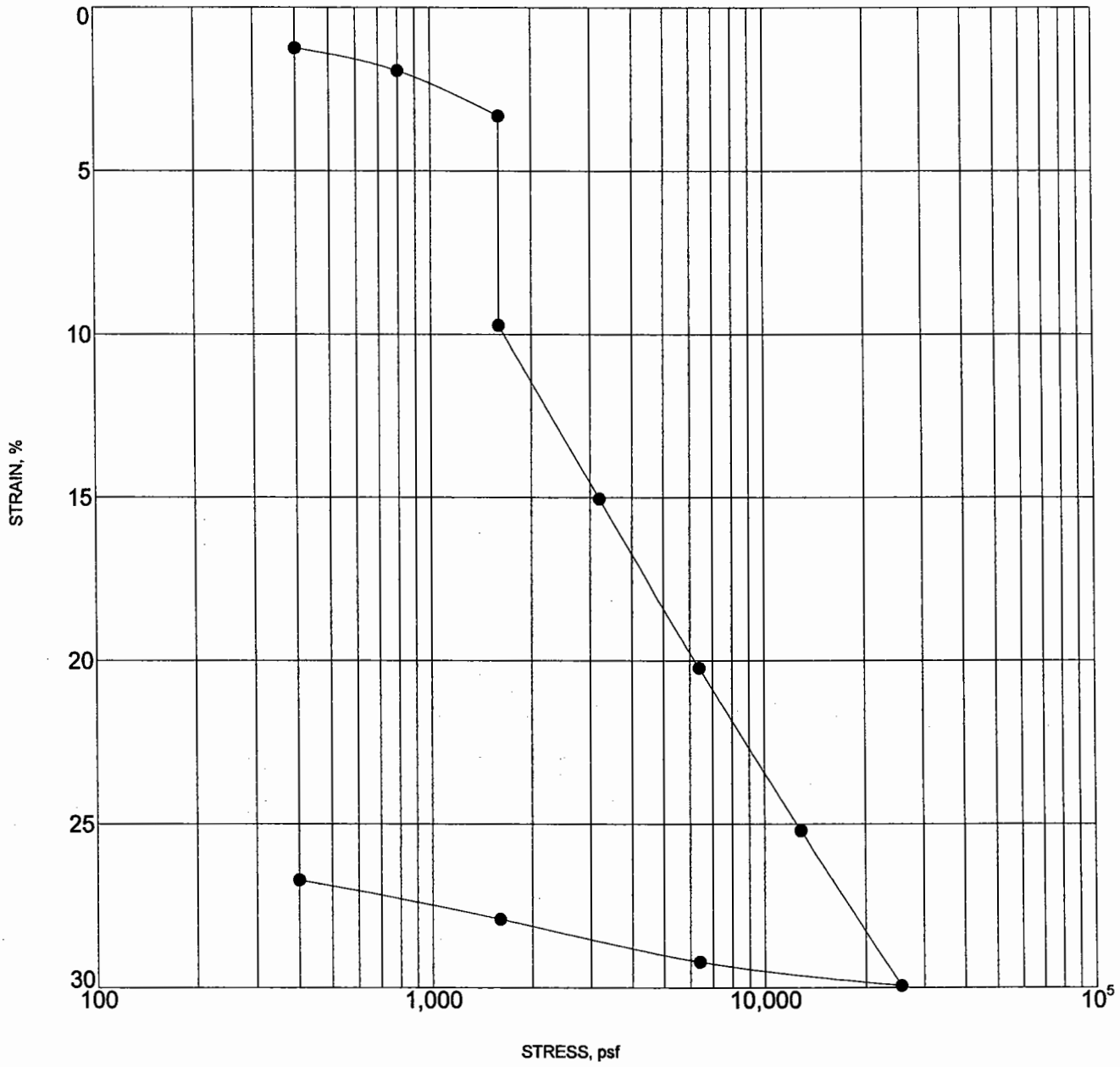
CONSOLIDATION TEST

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

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CONSOLIDATION TEST

FIGURE B-11

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