Geotechnical Report

APPENDIX G

GEOTECHNICAL
PROFESSIONALS INC.

GEOTECHNICAL INVESTIGATION
PROPOSED KIA DEALERSHIP DEVELOPMENT
24460 CALABASAS ROAD
CALABASAS, CALIFORNIA

Prepared for:
Hello Auto Group
c/o Integrity Design & Construction Services
P.O. Box 1171
Solana Beach, California 92075

Prepared by:
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Project No. 3162.I

December 16, 2022

GEOTECHNICAL
PROFESSIONALS INC.

December 16, 2022

Hello Auto Group
c/o Integrity Design & Construction Services
PO Box 1171
Solana Beach, CA 92075

Attention: Jody Stout (jstout@integritydcs.com)

Subject: Report of Geotechnical Investigation
Proposed Kia Dealership Development
24460 Calabasas Road, Calabasas, California
GPI Project No. 3162.I

Dear Jody:

Transmitted herewith is an electronic copy of our geotechnical investigation report for the subject project. The report presents our evaluation of the foundation conditions at the site and recommendations for design and construction. This report will need to be submitted by you or your representative to the City for review and approval.

We appreciate the opportunity of offering our services on this project and look forward to seeing the project through its successful completion. Feel free to call us if you have questions regarding our report or need further assistance.

Very truly yours,
Geotechnical Professionals Inc.

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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 Kia Dealership in Calabasas, California. The geographical location of the site is shown on the Site Location Map, Figure 1.

1.2 PROPOSED DEVELOPMENT

The proposed development will consist of a new dealership development located at the south side of Calabasas Road, west of Parkway Calabasas. Based on our review of the provided conceptual plans by AHT Architects, the proposed development consists of a 2-story dealership building with rooftop parking, including showroom, offices, service department, and parking. The building footprint is approximately 18,375 square feet and will be founded at about the existing grade (Elevation 1080 feet) on the north end and extending to a depth of about 18 feet below grade on the south end.

Based on limited information provided, the maximum column and wall loads for the proposed structure are anticipated to be on the order of 400 kips and 12 kips per lineal foot, respectively.

The recommendations given in this report are based upon preliminary structural and grading information. We should be notified if the actual loads and/or grade changes are known to either confirm or modify our recommendations.

1.3 PURPOSE OF INVESTIGATION

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

1.4 PRIOR NEARBY GEOTECHNICAL INVESTIGATIONS

We performed a geotechnical investigation for a previously planned automobile dealership at the site and presented the results in a report dated July 6, 2016 (GPI, 2016). Our prior investigation included borings (B-1 to B-9), test pits (TP-1 to TP-6), and geologic mapping across the site. Two of the borings (B-1 and B-2) were drilled near the footprint of the currently planned dealership building with the remainder of the explorations performed to the south of the proposed building. We have incorporated our prior data and findings into this report.

Additionally, we requested the available on-site and nearby prior geotechnical investigations from the City of Calabasas. The City did not have records of prior on-site geotechnical studies but did provide copies of the prior geotechnical reports for nearby sites, which we reviewed as part of our investigation. These reports are included in the list of references herein.

2.0 SCOPE OF WORK

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

The field exploration for this report consisted of two exploratory borings (B-101 and B-102). We have also included the field explorations from our previous investigation onsite which included nine exploratory borings and six test pits. The locations of the subsurface explorations are presented on the Site Plan, Figure 2.

The borings were drilled using predominately hollow-stem auger borings to depths ranging between 20 to 51 feet below existing grades. Two of the borings were drilled as part of our previous investigation using bucket auger equipment to depths of about 18 to 49 feet below existing grades. The larger diameter bucket auger borings were downhole logged by our Certified Engineering Geologist. The test pits were excavated with a backhoe to depths of 5 to 16 feet below grade to expose the subsurface soils for observation and sampling. A description of field procedures and logs of borings and test pits are presented in Appendix A.

Two infiltration tests were performed as part of our previous investigation in 2016. Test wells were installed adjacent to Borings B-1 and B-2. The wells were installed to depths of about 10 feet below the existing grade. Infiltration testing was performed in each well, with details of the testing presented herein, and test results presented in Tables 1.1 and 1.2, Borehole Infiltration Test Results.

Laboratory soil tests were performed on selected representative soil and bedrock samples as an aid in soil classification and to evaluate the engineering properties of the materials. The geotechnical laboratory testing program included determinations of moisture/density, shear strength (direct shear), compressibility (consolidation), expansion potential, compaction, gradation, R-value, and corrosion potential. Laboratory testing procedures and results are summarized in Appendix B.

R-value testing was performed by GeoLogic Associates, Inc. under subcontract to GPI. The test results are presented in Appendix B. Soil corrosivity testing was performed by HDR under subcontract to GPI. The results are provided in Appendix B of this report.

A geologic evaluation of the site was performed by our consulting Certified Engineering Geologist. The evaluation included downhole logging of two of our exploratory borings, observation during excavation of the test pits, surface mapping, and review of available published and unpublished data. The geologic-seismic evaluation is incorporated herein.

Engineering evaluations were performed to provide earthwork criteria, foundation, retaining structure, and slab design parameters, and assessments of seismic hazards. The results of our evaluations are presented in the remainder of this report.

3.0 SITE CONDITIONS

3.1 SURFACE CONDITIONS

The site is irregularly shaped and bounded by Calabasas Road and a separate lot to the north, an existing automobile dealership to the east, an ascending slope up to a street (Park Granada) to the south, and undeveloped land to the west. The site was formerly occupied by a commercial nursery, and several minor structures related to those operations are still present on the site. Also present on the site are both paved and unpaved access roads and numerous trees, including medium to very large oak trees.

The ground surface elevations within the area planned for development ranges from 1079 feet at the site entrance off Calabasas Road up to about Elevation 1110 feet at the southern edge of the planned development. The slope continues to ascend to the street above the planned development (Park Granada), to an elevation of about 1370 feet. The site topography is shown on the Site Plan, Figure 2, Site Geologic Plan, Figure 3, and Figure 4, Subsurface Cross Section.

The pavement observed during our fieldwork appeared to be in poor to fair condition. The existing pavement sections at our boring locations consisted of 2 to 5 inches of asphalt concrete over about 0 to 5 inches of aggregate base. The patio slab outside the former sales building at Boring B-1 consisted of 3 inches of portland cement concrete.

3.2 SUBSURFACE CONDITIONS

Our field explorations disclosed a subsurface profile generally consisting of undocumented fills overlying natural materials. Detailed descriptions of the materials encountered are shown on the Logs of Borings and Test Pits, Appendix A. The geologic conditions are also discussed in a following section.

Fill soils were encountered in some of our borings and test pits to depths of 1 to 10½ feet. The deeper fill soils were encountered in the lower elevations near Boring B-5 and Test Pit TP-2. The fill soils consisted predominantly of silty and sandy clays with varying amounts of shale fragments and caliche. The fill was not consistent with properly compacted fill in that densities were variable and moisture contents varied from slightly moist to very moist. Documentation regarding the placement and compaction of the fills was not available. The fill soils have a medium expansion potential and are anticipated to shrink and swell with changes in moisture content.

The natural materials encountered consisted of colluvium/alluvium over bedrock. The colluvium/alluvium extended to a depth of about 20 feet at the northern portion of the proposed building pad and about 6 to 10 feet on the southern end. The colluvium/alluvium consisted predominantly of sandy clay, silty clay, clayey silt, silt, and silty sand. These soils were generally stiff to very stiff and medium dense with relatively low densities and slightly porous in the upper 10 feet. Below 10 feet these materials were generally very stiff to hard. The moisture content of the soils ranged from slightly moist to wet. The underlying bedrock consisted of siltstone and sandstone that was found to be hard and dense to very dense (soil consistencies). The sandstone was found to generally be massive, poorly to moderately cemented, and contain infrequent layers of conglomerate. The siltstone was found to generally be massive to moderately bedded. Further details regarding the bedrock materials encountered are presented in the following Site Geologic Conditions section of this report.

The natural soils and siltstone have a medium expansion potential and will shrink and swell with changes in moisture content. Select corrosivity testing indicates that the on-site soils are moderately corrosive to concrete (sulfate content) and reinforcing steel (chloride content). These results are consistent with the findings at nearby sites.

3.3 SITE GEOLOGIC CONDITIONS

The project site consists of an irregular-shaped parcel in moderate relief hillside terrain in the Calabasas area of Los Angeles County. The property is on the south side of an unnamed canyon, where the 101 Freeway and Calabasas Road have been constructed. The property consists of relatively small ridgelines and canyons that descend from a higher ridgeline to the south to the main drainage where the freeway is located.

Regional geologic maps (Dibblee, T.W. Jr., 1992, Geologic Map of the Calabasas Quadrangle) indicate that the site area is underlain by Tertiary age Sedimentary rocks typical of this portion of the Santa Monica Mountains. Specifically, the site is mapped as being underlain by bedrock of the Upper Topanga formation, generally dipping at moderate inclinations to the northwest and northeast near the crest and east limb of a northerly plunging anticline, as indicated on the attached Regional Geologic Map, Figure 3.

Our field investigation, which consisted of a total of eleven borings, six backhoe pits, and geologic mapping of available bedrock exposures in road cuts on the site, generally confirmed the bedrock geology of the site. Soil and geologic conditions determined by our field investigation, as well as the locations of exploratory excavations, are shown on Figure 3, and Subsurface Cross Section, Figures 4. Five of the borings were drilled utilizing a hollow stem auger in the canyon bottoms to determine the thickness and engineering characteristics of the surficial deposits overlying the bedrock. The surficial deposits were mapped as colluvium/alluvium undifferentiated and consisted primarily of silty and sandy clays. These deposits were typically 5 to 10 feet thick in the higher portions of the site and reached a maximum thickness of approximately 20 feet in the area near Calabasas Road in Boring B-1 and B101. Three backhoe test pits were also excavated in the canyon areas to further delineate the occurrence and depth of the surficial deposits.

Two of the borings were drilled utilizing a 24-inch diameter bucket or spiral auger to facilitate downhole logging to determine geologic structure in bedrock areas. Geologic structure determined by downhole logging was augmented by logging of three backhoe excavations and geologic mapping of bedrock exposures. As shown on the attached Site Geologic Map, bedding in the bedrock generally strikes nearly east-west and dips at moderate inclinations to the north, northeast, and northwest. In Boring B-7, the dip of bedding was observed to be steeper than in other areas, possibly influenced by east-west trending faults observed in Borings B-4 and B-9. As observed in the surface outcrops, borings and backhoe excavations, the Upper Topanga formation consisted of generally fine-grained, massive to vaguely bedded sandstone with infrequent conglomerate beds, and massive to thinly bedded siltstone.

The native soil and bedrock are overlain in limited areas with generally thin fill deposits placed during the previous use of the site as a nursery. The fills should be considered as uncompacted.

3.4 GROUNDWATER AND CAVING

Groundwater seepage was observed at depths of 34 feet in Boring B-6, 31 feet in Boring B-8, and 46 feet in Boring B-9, corresponding to elevations of about 1103 to 1108 feet. Groundwater was not encountered in the remainder of our field explorations, including the four borings performed within the currently proposed building area. The seepage was observed in the sandstone and appears to be perched on underlying siltstone or more cemented sandstone layers. The possibility exists that some minor seepage of groundwater may be encountered in the excavation and walls below grade planned for the southern portion of the proposed building that will extend into the bedrock. The local groundwater source is likely from irrigation practices above the site and from infiltration of rain runoff. The depth to groundwater and groundwater seepage can be expected to vary annually.

Caving was not encountered in our 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, provided the recommendations contained herein are incorporated into the design and construction of the project.

Furthermore, in accordance with the County of Los Angeles Statement 111, it is our opinion that the project will be safe for its intended use against hazard from landslide, settlement, or slippage and the project will have no adverse effect on the stability of the site or adjoining properties.

The major geotechnical constraints related to the proposed construction are as follows:

- Based on the sloping ground surface, the finished floor of the proposed building will be at about the existing grade at the northern building limit and at about 18 feet below the existing grade at the southern building limit. Given the gradual descending slope of the bedrock surface towards the north, the southern portion of the finished building pad will be subterranean and extending into the undisturbed bedrock while the northern portion of the building will be underlain by up to about 20 feet of soil over the bedrock surface. Given the differing foundation depths and supporting materials, we recommend supporting the proposed building on a mat foundation underlain by properly compacted fill to mitigate the potential differential settlement.

- Undocumented and low-density fills are present at the site ranging from 1 to 10½ feet below existing grades, with the deepest fill encountered in the vicinity of our Borings B-2, B-102 and B-5 and Test Pit TP-2, which are located north of the planned building. These fills are not suitable for support of the proposed structures or floor slabs. We recommend that undocumented fills be removed and replaced as properly compacted fill within the building pad, where not removed by cut. The depths of removal and details regarding excavations are provided in the “Earthwork” section of this report.

- The upper natural soils immediately beneath the fill soils have variable strength and consolidation characteristics within the upper 10 feet of the ground surface. To provide uniform support for the planned building and avoid a soil/bedrock contact immediately beneath the mat foundation, the upper natural materials encountered at the completion of the fill removal or excavation for the subterranean portion of the building should be removed and replaced with properly compacted fills to provide uniform support for the mat foundation. The depths of removal and details regarding excavations are provided in the “Earthwork” section of this report.

- Foundations for retaining walls and minor structures such as screen walls or retaining walls less than 5 feet high may be supported on shallow foundations established in properly compacted fill or undisturbed natural soils. In-place undocumented fill soils should be removed in their entirety beneath the foundations. The depths of removal and details regarding excavations are provided in the “Earthwork” section of this report.

- Clays encountered at the site exhibit a medium expansion potential (expansion index of 54). To help mitigate the expansion potential, the clay soils should not be placed as compacted fill within 2 feet of pedestrian concrete hardscape subgrade. Building floor slabs will not be impacted because of the plans for a mat foundation. The soils placed to support pedestrian hardscape should consist of granular, non-expansive (E.I. of 20 or less) on-site or imported soils. Such materials were encountered within our explorations (sandstone and silty sands) but selective grading will be required to identify and stockpile these soils for use in capping these areas.

- As noted in the geologic assessment of the site, the bedrock encountered in our explorations was noted to be dipping to the northwest and northeast, which will create an adverse condition for north facing retaining walls and excavations for the southern subterranean portion (upslope) of the building. As such, the stability of excavations extending into the bedrock material is anticipated to be adversely affected by the potential for adverse bedding. We have provided recommendations herein to address the lateral earth pressures related to the adverse bedding, as well as earthwork accommodations to maintain stable excavations. We recommend that our Geologist be on-site during the excavation to confirm the actual subsurface conditions encountered.

- Groundwater seepage was encountered at depths of 31 to 46 feet below the existing grades in our explorations, corresponding to Elevations 1103 to 1108 feet. Because the planned building will be located in the lowest area of the site with a partial subterranean condition established at about Elevation 1080 feet, the groundwater should be accounted for in performing the required excavations and in the long-term performance of the subterranean portions of the structure.

- Based on our field testing, the on-site soils and bedrock do not appear to be conducive to infiltration. Tested infiltration rates in wells established at depths of about 10 feet near our Borings B-1 and B-2 were found to be 0 to 0.1 inch/hour.

4.2 SEISMIC CONSIDERATIONS

4.2.1 General

The site is in a seismically active area typical 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, 2022 edition. For the 2022 CBC, a Site Class C may be used.

4.2.2 Strong Ground Motion Potential

Based on published information (geohazards.usgs.gov), the most significant active faults in the proximity of the site are the Malibu Coast and Santa Monica faults, which are located about 7½ and 9½ miles from the site, respectively.

During the life of the project, the site will likely be subject to strong ground motions due to earthquakes on nearby faults. Based on the USGS website (earthquake.usgs.gov), we computed that the site could be subjected to a peak ground acceleration (PGA_M) of 0.79g for a mean magnitude 6.8 earthquake. This acceleration has been computed using the mapped

Maximum Considered Geometric Mean peak ground acceleration from the ASCE 7-16 (for 2022 CBC) and a site coefficient (F_{PGA}) based on Site Class. The predominant earthquake magnitude was determined using a 2-percent probability of exceedance in a 50-year period, or an average return period of 2,475 years. The structural design will need to incorporate measures to mitigate the effects of strong ground motion.

The corresponding seismic design parameters from the CBC are as follows:

<u>2022 CBC:</u>

$S_S = 1.59g$	$S_{MS} = F_a * S_S = 1.91g$	$S_{DS} = 2/3 * S_{MS} = 1.27g$
$S_1 = 0.56g$	$S_{M1} = F_V * S_1 = 0.81g$	$S_{D1} = 2/3 * S_{M1} = 0.54g$

The above seismic code values should be confirmed by the Project Structural Engineer using the value above and the pertinent internet website and tables from the building code.

4.2.3 Potential for Ground Rupture

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

4.2.4 Liquefaction

The site is not located within an area mapped by the State of California as having a potential for soil liquefaction (CGS, 1997). Groundwater seepage was encountered at depths of 31 to 46 feet in three of our recent explorations. The seepage was encountered in dense sandstone. Historical information is limited as the bedrock material underlying the site is not considered to be water bearing. Excluding the site vicinity from the potential liquefaction zone was based primarily on the suggestion that the bedrock materials have likely reached a state of consolidation that would preclude liquefaction based on their geologic age, and the colluvium/alluvium soils are well above the encountered groundwater.

Based on our evaluation, we conclude that the potential for liquefaction to adversely impact the planned project is very low. The potential seismic-induced liquefaction settlement will be less than ¼-inch.

4.2.5 Seismic Ground Subsidence

Seismic ground subsidence (not related to liquefaction induced settlements), occurs when loose, granular (sandy) soils above the groundwater are densified during strong earthquake shaking. Earthquake-induced seismic subsidence during a strong earthquake is not anticipated to adversely affect the planned project because of the planned subterranean construction and stiff to very stiff cohesive soils overlying the bedrock.

4.3 SLOPE AND WALL STABILITY

The site is located within a north-south trending canyon extending up from Calabasas Road to Park Granada. The dealership development is planned in the lower portion of the site where the north-south ground surface slopes gradually up to the south at an inclination of about 8:1 (h:v), with localized slopes of up to 2:1. The area of the planned development was previously used as a commercial nursery, and much of the ground surface has been terraced and graded for access. The natural slopes at the site above the planned development are covered with moderate to heavy growth (brush and trees) and inclined at about 2¼:1 to 4:1.

The site is in an area designated as a hillside area by the County of Los Angeles (County of Los Angeles, 1990). The state (CGS, 1997) indicates that there are areas of potential earthquake-induced landslides in the vicinity of the site, but that the subject site is not within such a zone. There are no known landslides adjacent to the site, nor is the site within or in the path of known or potential landslides. The results of our site specific geotechnical and geologic investigation indicates that the on-site slopes are grossly stable.

4.3.1 Gross Stability

Based on our site reconnaissance and review of available data, there are no known landslides at the site or on immediately adjacent sites. Based on our geologic investigation, the material in the lower portions of the site consist of Upper Topanga sandstones and siltstones that were found to be predominantly massive to thinly bedded. Where measured, bedding in these deposits was generally to the northwest and northeast, which is generally adverse, but also predominantly at relatively flat inclinations (12 to 18 degrees to the horizontal). Steeper inclinations were encountered in our downhole logging of one boring (Boring B-7), but these inclinations were encountered in a siltstone layer within the upper 12 feet, which was underlain by a massive sandstone deposit. Bedding information is presented in Figure 3, with apparent bedding inclinations shown on Figure 4. Regional geologic mapping (Dibblee, 1992) indicates that the predominant bedding is to the north, but also indicates variable inclinations in the upper areas of the slopes at the site, including favorable inclinations dipping to the south. These conditions are presented on Figure 6.1 and 6.2, Regional Geologic Map and Cross Section.

Based on our findings, the natural slopes at the site are considered to be grossly stable, with the potential for slope instability to adversely affect the planned development to be low. The planned development will not require significant cuts to reach the planned finished grades except for the excavation required for the southern subterranean portion of the building, which will extend to about 20 feet in depth below the existing grades. Such excavations will expose both adverse and favorable bedding conditions. In general, we anticipate north facing cut slopes (southern building walls) to expose adverse bedding, with the predominant adverse bedding inclinations being relatively flat (12 to 18 degrees).

4.3.2 Surficial Stability

We did not observe evidence of surficial slope failure or debris flows during our geologic site reconnaissance or review of aerial photographs. We performed surficial stability analyses for the slopes that will remain above the planned development. We assumed that the in-place soils would be saturated with a failure surface extending parallel and 3 feet below the ground surface. Based on our analyses, the natural soils will have a factor of safety of at least 1.5 with respect to surficial stability. This finding corresponds to the conditions observed on-site.

4.3.3 Slope and Wall Stability Analyses

We used shear strength parameters in our slope stability analyses that were developed by testing saturated samples of the colluvium/alluvium and bedrock. We tested selected bedrock samples by re-shearing the samples multiple times to simulate along bedding strengths. We also reviewed the along-bedding shear strength values published in State documents for similar bedrock materials. The shear strength values used in our analyses are presented in the following table:

Material Type	Unit Weight (pcf)	Friction Angle (degrees)	Cohesion (psf)
Siltstone (along Bedding)	120	20	300
Siltstone (across bedding)	120	25	350
Sandstone (massive)	120	37	350
Colluvium/Alluvium	120	22	175
Compacted Fill	120	25	250

Our analyses of the planned slopes and walls/shoring included evaluating the subsurface materials using both wedge-type and circular failure surfaces. Using the soil shear strengths values above, we modeled the bedrock as having along bedding shear strengths for failure surfaces dipping out of slope between 12 and 18 degrees for north-facing slopes (south facing retaining walls). We considered east- and west-facing cuts as exposing favorable bedding and across bedding shear strengths.

For determination of lateral earth pressures, we determined the stability of the planned cut, and then determined the required point load to provide a factor of safety of 1.5. We then converted the point load into an equivalent fluid pressure to determine an appropriate lateral earth pressure to use for the design of temporary shoring and permanent retaining walls. Lateral earth pressures are presented in the Retaining Structures and Shoring section of this report.

4.4 EARTHWORK

The earthwork anticipated at the project site will consist of clearing and grubbing, excavations, subgrade preparation, and the placement and compaction of fill.

4.4.1 Clearing and Grubbing

Prior to grading, the areas to be developed should be stripped of landscaping, cleared of demolition debris, old foundations, pavements, and utilities. Deleterious material generated during the clearing operation should be removed from the site. Where appropriate, existing underground utilities should be removed in their entirety and properly capped. Should cesspools or other buried obstructions be encountered in the building areas during construction, they should be removed in their entirety. The resulting excavations should be backfilled as recommended in the "Subgrade Preparation," “Material for Fill,” and "Placement and Compaction of Fill" sections of this report. As an alternative, cesspools can be backfilled with lean sand-cement slurry. At the conclusion of the clearing operations, a representative of GPI should observe and accept the site prior to any further grading.

4.4.2 Excavations

Excavation at the site will include removal of existing undocumented fill and low density/compressible natural soils in building and conventional retaining wall areas, excavation to finish subgrade, footing excavations, and trenching for utility lines.

Building Pad, Pavement, and Minor Structures

Prior to construction of the building supported on a mat foundation, existing undocumented fills and the upper natural soils in the proposed building areas should be removed and replaced as properly compacted fill when not removed by the planned cuts. Undocumented fills were encountered in our explorations in the lower portion of the site (Borings B-1, B-2, B-5, B-101, B-102, and Test Pit TP-2) to depths of up to 10½ feet below existing grades. The upper natural soils encountered in the lower portion of the site have variable strength and consolidation characteristics. As such, these soils are not considered to be suitable for direct support of the planned mat foundations and should be included in the required overexcavation within the building areas. For planning purposes, removals should extend to depths of 12 feet below the existing grade or 4 feet below the base of the mat foundation, whichever is deeper. The purpose of the removals beneath the base of the mat foundation in the subterranean portion of the building is to reduce the potential differential settlements across the soil/bedrock contact. In doing so, the required depth of excavation for the southern portion of the building will be increased beneath the mat to allow for compacted fill placement with the result being more uniform support for the mat foundation.

For planning purposes, removals for retaining walls taller than 5 feet should remove the in-place undocumented fill and extend to depths of at least 7 feet below the existing grade or 3 feet below the base of the wall footing, whichever is deeper. As an alternative, tall retaining walls footings can be established directly within the undisturbed bedrock if the entire wall footing will extend deep enough to extend at least 1 foot into the bedrock.

Removals for minor structures, such as retaining walls less than 5 feet high or screen walls, should extend at least 4 feet below the existing grade or 2 feet below the base of the foundations, whichever is deeper. In concrete pedestrian hardscape areas, removals should extend at least 2 feet below the planned finished subgrade to allow for the placement of relative non-expansive, granular soils. In pavement areas, removals should extend at least 1 foot below the existing grade or the finished subgrade, whichever is deeper.

The actual depths of removals will need to be determined during grading in the field by a representative of GPI.

Lateral Limits

The base of removals should extend laterally beyond the building line or edge of footings a minimum distance of 5 feet or the depth of overexcavation/compaction below foundations (i.e., a 1:1 projection below the bottom edge of the mat foundation or footing), whichever is greater. For the northern portion of the building where relatively deep removals are anticipated, the lateral limits of the base of the removals will be governed by the 1:1 projection below the edge of the mat. Building lines include canopies, loading docks, and other foundation supported improvements. The lateral limits of removals should be confirmed and certified by the project surveyor. GPI does not practice surveying; therefore, we cannot confirm lines, grades, or limits of earthwork.

Existing Utilities

Where not removed by the aforementioned excavations, existing utility trench backfill should be removed and replaced as properly compacted fill within the building pad. The limits of removal should be confirmed in the field. We recommend known utilities be shown on the grading plan.

Caving Potential and Cuts

Temporary construction excavations may be made vertically without shoring to a depth of 5 feet below adjacent grade. The inclination required for deeper excavations will be dependent on the direction of the cut relative to the direction of bedding of the siltstone. We recommend deeper cuts be properly shored or inclined as follows:

- Within the alluvium/colluvium soils, cuts up to 9 feet can be made at an inclination of ¾:1 (horizontal to vertical), cuts up to 14 feet sloped back to at least 1:1, and cuts up to 19 feet sloped back to 1½:1 or flatter.

- For east- or west-facing cuts in the bedrock materials, cuts up to 16 feet within the existing bedrock may be inclined at ¾:1, cuts up to 24 feet may be inclined at 1:1, and cuts up to 32 feet may be inclined at 1¼:1. Our Geologist should observe the excavations as they proceed to confirm favorable bedding conditions and the absence of potentially adverse shears in the bedrock.

- For north-facing cuts in the bedrock materials, cuts up to 10 feet within the existing bedrock may be inclined at 1:1, cuts up to 16 feet may be inclined at 1¼:1, and cuts up to 27 feet may be inclined at 1½:1 or flatter. Our Geologist should observe the excavations as they proceed to confirm the bedding conditions exposed are consistent with our initial findings. Flatter slope inclinations may be required based on the conditions exposed.

Surcharge loads should not 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 an adjacent existing building or settlement sensitive structure should be properly shored to maintain support of such adjacent elements. Excavations and shoring systems should meet the minimum requirements given in the most current State of California Occupational Safety and Health Standards.

Details regarding slope related cuts and fills are presented on Figure 7, Slope Buttress or Replacement Fill Detail, Figure 8, Benching Detail, and Figure 9, Side Hill Cut Pad Detail. Buttress fills should include a subdrain as outlined in the details.

Slot Cuts

Where space is not available for open cuts of deeper removals along property lines and adjacent to existing improvements, shoring or slot cuts will be required. Recommendations for shoring are provided in the “Retaining Structures and Shoring” section of the report. Removals that will undermine existing adjacent pavements or hardscape may utilize “ABC” slot cuts to depths not greater than 12 feet. “ABC” slot cuts adjacent to the public right-of-way where traffic loads will be present should account for a live-load traffic surcharge. Slot cut recommendations for unsurcharged and surcharged conditions are provided below:

- Unsurcharged slots up to 8 feet in height should not be wider than 8 feet.
- Unsurcharged slots up to 12 feet in height should not be wider than 6 feet.
- Surcharged slots (subject to 250 psf traffic loads) up to 8 feet in height should not be wider than 6 feet.
- Surcharged slots (subject to 250 psf traffic loads) up to 12 feet in height should not be wider than 4 feet.

The slot cuts should be backfilled immediately to finished grade prior to excavation of the adjacent four slots (two on each side of the excavated slot). Although not anticipated, the allowable width of the slots adjacent to existing structures that will surcharge the excavation can be provided when additional details of the surcharge loads are provided. A test slot should be performed prior to production slots to confirm the stability of the planned cuts.

Rippability

Based on our site-specific explorations, we anticipate that the on-site materials can generally be excavated using conventional grading equipment. We did encounter localized hard, cemented layers in some of our borings in the upper portion of the site, but these materials were outside of the proposed limits of the new building and parking areas. If plans change to include the southern portion of the overall site, these materials will likely require special excavation equipment and processes, such as heavy ripping, and should be considered high cost/low production material to excavate.

4.4.3 Subgrade Preparation

Prior to placing fills, the subgrade should be scarified to a depth of 12 inches, moisture-conditioned, and compacted to at least 90 percent (95 percent for granular soils) of the maximum dry density in accordance with ASTM D1557 and to a firm and unyielding condition.

Localized areas of very moist soils (4 to 8 percent above the optimum moisture content) were encountered within the near surface soils in our Borings B-1, B-5, B-101 and B-102. Such materials are anticipated to yield under heavy rubber-tired equipment and will likely require stabilization to support compaction efforts. For planning purposes, we anticipate about one-third of the exposed subgrade areas in the planned pavement areas in the lower site elevations will require stabilization. Such measures may consist of mechanically drying the wet soils and compacting using steel-wheel or track equipment, or the placement of 12 inches of crushed miscellaneous base prior to construction of the planned pavement section or placement of compacted fill soils. A thicker layer of aggregate base or placement of a suitable geogrid material may be required if the exposed subgrade is disturbed to the point of yielding (i.e. “pumping”). Where wet soils are encountered, subgrade processing will be evaluated in the field by GPI.

4.4.4 Material for Fill

In general, the on-site soils are suitable for use as compacted fill. However, the on-site clays should not be used as retaining wall backfill or within the upper 2 feet of concrete pedestrian hardscape areas. Additionally, drying and mixing of the on-site materials will be required to obtain near optimum moisture conditions prior to placement and compaction. Given the clayey nature of the soils and presence of siltstone and cemented sandstone, processing times will likely take longer than would with more granular soils. In addition, grading during the rainy season will likely result in more challenges with moisture conditioning the on-site soils.

Provided it is acceptable to the reviewing governmental agencies and owner, crushed, inert demolition debris, such as concrete and asphalt, may be used in fills provided it is crushed to a well graded mixture with maximum particle size of 1½ inches and blended with the on-site soils.

Imported fill material should be predominately granular (contain less than 40 percent fines-portion passing the No. 200 sieve) and non-expansive (an Expansion Index of 20 or less). The import soils should contain sufficient fines/binder to be stable in open excavations, as well as be able to support construction equipment without rutting. GPI should be provided with a sample (at least 50 pounds) and notified of the location of soils proposed for import at least 72 hours in advance of 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 GPI may be rejected if not suitable.

Both imported and existing on-site soils to be used as fill should be free of deleterious debris and particles larger than 6 inches in diameter. Within the footing depths for the building pad and retaining walls, soil particles should be 3 inches in diameter or less, precluding the placement of cobble-sized particles.

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 two sacks of cement per cubic yard and have a maximum slump of 5 inches.

If open-graded rock is used as backfill, the material should be placed in lifts and mechanically densified. Open-graded rock should be separated from the on-site soils by a suitable filter fabric (Mirafi 140N or equivalent).

4.4.5 Placement and Compaction of Fills

Fill soils should be placed in horizontal lifts, moisture-conditioned, and mechanically compacted to at least 90 percent (siltstone, silts and clays) of the maximum dry density (ASTM D1557). Fills placed at depths greater than 10 feet below finished grades should be compacted to at least 92 percent of the maximum dry density. Additionally, granular soils (sandstone, sands, silty sands, and clayey sands) should be compacted to at least 95 percent. 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, track equipment	4-6 inches
Small vibratory or static rollers	6-8 inches
Scrapers, heavy loaders, large vibratory rollers	8-12 inches

The maximum lift thickness should not be greater than 12 inches. Each lift should be thoroughly compacted and accepted prior to placement of the next lift of fill.

The moisture content of the clayey fill materials should be within 2 to 4 percent over the optimum moisture content (pumping imminent) to readily achieve the required degree of compaction and reduce the potential for moisture related swell or subsidence. For the sandy fill materials, the moisture content should be within 0 to 2 percent over the optimum moisture content. The moisture content of existing near surface soils, in general, is variable and will require both drying and moistening prior to compaction. Earthwork contractors should include moisture conditioning of the existing soils prior to recompaction in the bids.

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

4.4.6 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 20 to 25 percent and subsidence of 0.1 feet may be assumed for the existing fill and colluvium/alluvium soils. Significant shrinkage or subsidence is not anticipated for the bedrock materials. These values are estimates only and exclude losses due to removal of vegetation, debris, or existing underground structures. Actual shrinkage and subsidence will depend on the types of earthmoving equipment used and should be determined during grading.

4.4.7 Trench and Retaining Wall Backfills

Utility trench and wall backfill should be mechanically compacted in lifts. Wall backfill should consist of the on-site or imported silty sands or sands. As detailed previously, some moisture conditioning of the on-site soils should be anticipated prior to replacement as trench 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. A representative of GPI should observe and test trench and wall backfill as they are placed.

4.4.8 Observation and Testing

A representative of GPI should observe excavations, subgrade preparation and fill placement activities. Sufficient in-place field density tests should be performed during fill placement 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 additional fill.

Our geologist should observe the conditions exposed during the mass excavation. If conditions are different than expected, alternate recommendations, based on the actual conditions encountered, may be required.

4.5 FOUNDATIONS

4.5.1 General

Because the proposed building will be partially subterranean and extending into the bedrock in the southern subterranean portion while being underlain by up to about 20 feet of properly compacted fill in the northern at-grade portion, we evaluated several foundation alternatives.

Included in these alternatives was the use of deep foundations or ground improvement methods such as rammed aggregate piers for the northern at-grade portion, or performing remedial grading and supporting the building on a more uniform mat foundation across both the northern and southern portions. As a result of our evaluation, we recommend performing the remedial earthwork (overexcavation and recompaction of the undocumented fill and a portion of the natural soils and bedrock) outlined previously in the Earthwork section of the report and supporting the building on a mat foundation.

Retaining walls and minor structures 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.

New footings should be located beyond a 1:1 plane drawn upward from the base of new or existing retaining walls, or the wall should be designed for the additional surcharge load.

4.5.2 Mat Foundation - Building

Allowable Bearing Capacity and Elastic Modulus

The allowable bearing pressure for a mat foundation is generally not the governing geotechnical design issues as compared to the anticipated settlement. At this preliminary stage, estimate static mat foundation pressures for the proposed dealership building are not yet available. GPI should be provided with a detailed plot of the anticipated mat bearing pressures for our review when those plans are available.

For elastic design of the mat foundation, a preliminary modulus of subgrade reaction (k-value) of 120 pounds per cubic inch (pounds per square inch per inch of deflection. This value is for a 1-foot by 1-foot square loaded area and should be adjusted for the area of the mat foundation using appropriate elastic theory. Using generally accepted methods and our site-specific consolidation test results, we recommend using a value of approximately 30 pci for the adjusted k-value in designing the mat foundation. As previously discussed, we should be provided with the anticipated mat pressures when they are developed so that we can review and confirm the recommendations provided as well as provide an estimate for the anticipated maximum settlements for the mat foundations.

The allowable soil bearing pressure will be significantly greater than the average bearing pressures required for the mat foundation discussed above. For preliminary design purposes, an average allowable bearing pressure of 2,500 pounds per square foot (psf) may be used. At localized areas of the mat, such as columns and point-of-load applications along exterior walls, a static allowable bearing pressure of up to 4,000 psf may be used. These allowable bearing pressures are for dead-plus-live loads and may be increased one-third for short-term, transient, wind and seismic loading.

Settlement

Based on the load information assumed for the building (column loads on the order of 400 kips and wall loads on the order of 12 kips per lineal foot corresponding to mat pressures on the order of 2,500 psf), the total static foundation settlement for the mat foundation is estimated to range from ¼ to 1 inch with maximum differential settlements expected to be on the order of ¾-inch.

4.5.3 Shallow Footings – Retaining Walls and Minor Structures

Allowable Bearing Capacity and Footing Depths

Based on the shear strength and elastic settlement characteristics of the recompacted on-site soils (new fills), a static allowable bearing pressure of up to 3,000 pounds per square foot (psf) may be used for retaining wall and minor structure footings supported on properly compacted fill. Based on the shear strength and elastic settlement characteristics of the undisturbed on-site bedrock, a static allowable bearing pressure of up to 6,000 pounds per square foot (psf) may be used for continuous footings that extend at least 1 foot into the undisturbed bedrock (wall footings should be supported entirely in properly compacted fill if it cannot be supported entirely in the undisturbed bedrock).

These bearing pressures are for dead load-plus-live loads, and may be increased one-third for short-term, transient, wind and seismic loading. The actual bearing pressure used may be based on economics and structural loads and will 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.

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

Footings on Properly Compacted Fill

Static Bearing Pressure (psf)	Minimum Footing Width (inches)	Minimum Footing* Embedment (inches)
3,000	36	24
2,500	24	24
2,000	24	18
1,500	18	18

* Refers to minimum depth below lowest adjacent grade.

Minimum footing widths and depths of 18 inches should be used even if the actual bearing pressure is less than 1,500 psf for footings supported in properly compacted fill.

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

Footings on Undisturbed Bedrock

Static Bearing Pressure (psf)	Minimum Footing Width (inches)	Minimum Footing* Embedment (inches)
6,000	48	24
5,000	36	24
4,000	24	24
2,500	18	18

* Refers to minimum depth below lowest adjacent grade.

Minimum footing widths and depths of 18 inches should be used even if the actual bearing pressure is less than 2,500 psf for footings supported in the undisturbed bedrock.

Settlement

Based on the allowable bearing capacities previously presented for shallow footings supporting retaining walls and minor structures, the total static foundation settlement is anticipated to be within tolerable limits. We can provide more detailed settlement estimates for footings when further details on the loads are provided for walls and minor structures.

4.5.4 Lateral Load Resistance

Soil resistance to lateral loads will be provided by a combination of frictional resistance between the bottom of the mat foundation or 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 of foundations for structures supported in properly compacted fills, such as the building mat foundation. A coefficient of friction of 0.40 may be used for design of retaining wall footings supported on the undisturbed bedrock. In addition, an allowable lateral bearing pressure equal to an equivalent fluid weight of 250 pounds per cubic foot may be used for the mat foundation or footings are poured tight against compacted fill soils. An allowable lateral bearing pressure equal to an equivalent fluid weight of 400 pounds per cubic foot may be used for retaining wall footings that are poured tight against the undisturbed bedrock. These values (friction and lateral bearing) may be used in combination without reduction.

Footings adjacent to descending slopes should be deepened to allow for a lateral distance of at least one-half of the slope height, but not less than 15 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.

4.5.5 Foundation Concrete

Laboratory testing by HDR (Appendix B) on a selected sample indicates that the near surface soils exhibit a soluble sulfate content between 65 and 1,750 mg/kg. For the 2022 CBC, foundation concrete should conform to the requirements outlined in ACI 318, Section 4.3 for Category S2 levels of soluble sulfate exposure from the on-site soils. Chloride levels in the sample of the upper soils tested were found to be between 11 and 438 mg/kg, and we recommend a Category C1 be used for design.

4.5.6 Footing Excavation Observation

Prior to placement of concrete and steel, a representative of GPI should observe and approve the subgrade for the mat foundation and footing excavations.

4.6 RETAINING STRUCTURES AND SHORING

At the time of this report, plans for temporary shoring and permanent retaining walls were not finalized. However, the southern, eastern, and western building walls will require retaining walls, and if deeper removals are chosen over deep foundations, temporary shoring will likely be required along the north, east, and western portions of the site due to the relatively close proximity to the property lines. Retaining structures may include subterranean building walls and stand-alone retaining walls. The following recommendations are provided for temporary shoring and conventional retaining walls. We recommend that conventionally backfilled walls be backfilled with sandy (granular) soils. Sufficient granular soils may not be readily available on-site to backfill retaining walls and cap the floor slab and pedestrian hardscape subgrade. Because of the preliminary timing of this report and complexity of the site and project configuration, we should be provided with the design plans for shoring and retaining systems prior to finalizing to confirm suitable geotechnical design parameters have been used.

4.6.1 Lateral Earth Pressures

We recommend cantilevered shoring or retaining walls be designed using a triangular lateral earth pressure distribution. For braced or tied-back shoring or retaining walls, we recommend a trapezoidal lateral earth pressure distribution be used in design, as shown on Figure 10, Trapezoidal Earth Pressure Distribution. In addition to the static lateral earth pressure, retaining walls should be designed to resist short term seismic lateral earth pressures. We recommend seismic earth pressures be taken as an inverse triangular distribution.

The magnitude of lateral earth pressures will depend on the direction of the shoring or retaining wall, as well as the condition of backfill (level or sloping). The following earth pressures are recommended for the design of shoring and retaining walls, and assume fully drained conditions:

Direction Facing*	Bedding Condition	Backfill Condition	Cantilever (pcf)	Braced/Tie-Back (psf)	Seismic (psf)
North	Adverse	Level	58	40	15H
North	Adverse	2:1 (h:v)	87	61	25H
West	Favorable	Level	32	22	15H
West	Favorable	2:1 (h:v)	48	33	25H
East	Favorable	Level	32	22	15H
East	Favorable	2:1 (h:v)	48	33	25H
South	Favorable	Level	32	22	15H
South	Favorable	2:1 (h:v)	48	33	25H

* A north-facing wall is located at the south side of the building.

The coefficient H is the height of wall in feet. When the plans for the retaining walls and shoring are further developed, we can evaluate the specific configurations to further refine the above values.

In addition to the recommended earth pressure, the upper 10 feet of the shoring adjacent to roads 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 shoring due to normal automobile traffic. If traffic is kept at least 10 feet from the shoring, the traffic surcharge may be neglected.

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.

4.6.2 Conventional Retaining Walls

The recommended lateral earth 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 ¾ inch 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.

For retaining walls taller than 15 feet, the drain at the base of the wall should be supplemented by chimney drains extending up the back side of the wall. The chimney drain can consist of a prefabricated product, such as Miradrain, or a layer of gravel immediately behind the wall at least 12 inches wide and separated from the backfill soil with a suitable filter fabric. The chimney drain should be terminated about 2 feet from the finished grade, with the upper backfill consisting of the on-site soils.

The Structural Engineer should specify the use of select, granular wall backfill on the plans for conventional retaining walls. The select fill should extend behind the wall a distance laterally of one-third the wall height or to the back of the retaining wall footing, whichever is less. Wall footings should be designed as discussed in the "Foundations" section.

4.6.3 Soldier Pile Shoring and Tie-Backs

For the deep cuts planned at the south, east, and west sides of the planned dealership building (and the north side if deep foundations are not used), there may not be sufficient space for a sloped embankment. Potential methods for retaining the cut would be to install temporary shoring in front of a conventional wall, or use permanent tied-back soldier pile walls. The temporary shoring or soldier pile wall would consist of steel soldier piles placed in drilled holes, backfilled with concrete, and tied-back with earth anchors.

For temporary shoring and permanent soldier pile walls, the lateral earth pressures previously provided in Section 4.7.1 may be used for design.

<u>Soldier Piles</u>

For design of soldier piles spaced at least two diameters on centers for a permanent soldier pile wall, the allowable lateral bearing value (passive value) of the bedrock extending below the excavation may be taken to be 800 pounds per square foot at the excavated surface, up to a maximum of 8,000 psf. The allowable lateral bearing value (passive value) of the colluvium/alluvium extending below the excavation may be taken to be 500 pounds per square foot at the excavated surface, up to a maximum of 5,000 psf. To develop the full lateral value, provisions should be made to assure firm contact between the soldier piles and the undisturbed soils. The concrete placed in the soldier pile excavation below the excavated level may be a lean mix, but it should be of adequate strength to transfer the imposed loads to the surrounding soils.

Difficult drilling and refusal was locally encountered in our explorations into the bedrock materials. The shoring contractor should evaluate the potential drilling conditions when planning the installation methods. Caving was not encountered during our explorations.

The frictional resistance between the soldier piles and the retained earth may be used in resisting the downward component of the anchor load. The coefficient of friction between the soldier pile and the retained earth may be taken as 0.35. This value is based on the assumption ,that uniform full bearing will be developed between the steel soldier beam and the lean-mix concrete and between the lean mix concrete and the retained earth. In addition, provided the portion of the soldier piles below the excavated level is backfilled with structural concrete, the soldier piles below the excavated level may be used to resist downward loads. The frictional resistance between the concrete soldier piles and the soils below the excavated level may be taken as equal to 450 pounds per square foot.

For permanent tie-back walls, soldier piles should be protected from corrosion caused by contact with the on-site materials. Recommendations may be required from a corrosion engineer, such as HDR, who performed the corrosivity testing presented in Appendix B.

Lagging

Continuous lagging will be required between the soldier piles. The lagging could consist of timber or wire mesh and shotcrete/gunite. Careful installation of timber lagging will be necessary to achieve bearing against the retained earth. In areas where caving occurs, backfill of the lagging with clean sand fill or grout will be required. The soldier piles should be designed for the full anticipated lateral pressure. However, the pressure on the lagging will be less because of arching of the soils between piles. We recommend that the lagging be designed for the recommended earth pressure but limited to a maximum value of 400 pounds per square foot.

The temporary vertical lifts for the cuts between soldier piles to install the lagging should be limited to 5 feet. Shotcrete/gunite lagging should be properly cured prior to allowing excavation for the subsequent lift.

Tie-Back Anchors

Tie-back friction anchors may be used to resist lateral loads. For design purposes, it may be assumed that the active wedge adjacent to the shoring or wall is defined by a plane drawn at 35 degrees from the vertical through the bottom of the excavation for cuts facing east, west, and south (favorable bedding). For north facing cuts (adverse bedding), it may be assumed that the active wedge adjacent to the shoring or wall is defined by a plane drawn at 72 degrees from the vertical through the bottom of the excavation. The anchors should extend at least 25 feet beyond the potential active wedge and to a greater length if necessary to develop the desired capacities.

The capacities of anchors should be determined by testing of the initial anchors as outlined in a following paragraph. For design purposes, it may be estimated that drilled friction anchors will develop an average friction value of 800 pounds per square foot. Higher friction values may be feasible depending on the layout of the anchors. If two rows of tie-backs are planned, a higher friction value may be used, and can be provided if requested. Only the frictional resistance developed beyond the active wedge would be effective in resisting lateral loads. If post-grouted tie-back anchors are used, a preliminary average friction value of 2,000 psf may be assumed for planning. A higher value may be assumed if confirmed by field load testing. If the anchors are spaced at least 6 feet on-centers, no group action reduction in the capacity of the anchors need be considered.

The anchors may be installed at angles of 15 to 40 degrees below the horizontal. Care should be taken to confirm that the new anchors do not conflict with the utility lines upslope from the development. Caving of the anchor holes should be prevented with the installation method selected. The anchors should be filled with concrete placed by pumping from the tip out, and the concrete should extend from the tip of the anchor to the active wedge. The annular space around conventional anchors within the active wedge should not be backfilled until after testing has been completed. If caving is encountered, the void may be filled with wet sand. The anchor may be filled with concrete to the surface of the shoring for post-grouted anchors that are 8 inches in diameter or less.

For permanent tie-backs, the anchors should be protected from corrosion by epoxy coating or an equivalent method. The actual method used should be developed by a corrosion protection consultant, such as HDR.

Tie-Back Anchor Testing

For temporary shoring, GPI should select at least one of the initial anchors for a 24-hour hour, 200 percent test and three additional anchors for quick 200 percent tests. For a permanent tie-back wall, each of the permanent anchors should be proof-tested to 200 percent of the design load using a quick test, with four anchors selected for 24-hour tests. The purpose of the 200 percent test is to verify the friction value assumed in design. The anchors should be tested to develop twice the assumed friction value capacity. Where satisfactory tests are not achieved on the initial anchors, the anchor diameter and/or length should be increased until satisfactory test results are obtained. When the extent of the shoring program is known, we should review the recommended test program and make modifications as necessary.

For the 200 percent tests, the 200 percent test load should be maintained for 24 hours (24-hour test) and 1 hour (quick tests). The total deflection of the anchor during the test should not exceed 12 inches. The deflection after the 200 percent test load has been applied should not exceed 0.50 inch during any 1 hour period.

For temporary anchors, the remaining anchors should be pretested to at least 150 percent of the design load. The total deflection during the test should not exceed 12 inches between the anchor and the soldier pile. The rate of creep under the 150 percent load should not exceed 0.25 inch over a one-hour period and 0.1 inch over any 15-minute period within the one hour for the anchor to be approved for the design loading.

After a satisfactory test, each production anchor should be locked-off at the design load. The locked-off load should be verified by rechecking the load in the anchor. If the locked-off load varies by more than 10 percent from the design load, the load should be reset until the target load is achieved.

Deflection

It is difficult to accurately predict the amount of deflection of the shored embankment. It should be realized, however, that some deflection will occur. If it is desired to reduce the deflection, such as immediately adjacent to existing settlement sensitive improvements, the wall should be designed for higher lateral earth pressures.

With the relatively high tie-back anchor loads anticipated, the downward component of the anchor load will impose significant axial loads on the soldier piles. The frictional resistance of the soldier pile should be confirmed versus the downward component of the anchor load to evaluate the adequacy of the soldier pile embedment.

Monitoring

We recommend performing a detailed survey of the improvements supported above the planned cut prior to and during the shoring or wall 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 tops of the soldier piles. We can discuss the scope of the monitoring with the design team and owner when the design of the shoring or wall system has been finalized.

Drainage

The permanent walls should be drained full-height using a suitable drainage composite. If shoring is used, the drainage composite should be placed between the soldier piles prior to applying the shotcrete surface to allow for 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 soldier piles.

4.7 SLOPES

Based on the available information, cut and fill slopes are planned as part of the project. Permanent slopes at the site should be constructed at an inclination of 2:1 (horizontal to vertical) or flatter. We recommend fill slopes be overbuilt by at least 3 feet during rough grading and trimmed back to a hard and unyielding surface. Slope rolling to achieve a finished compacted surface should not be performed. A keyway at least 3 feet deep and equal in width to one-half the slope height, but not less than 15 feet, should be constructed prior to filling the slope. See Figures 7 and 8 for further details.

Setbacks of structures from the top and toe of the planned slopes should be maintained as directed by the regulatory agency.

Drainage devices and erosion control measures should be installed as required by the governing agencies.

4.8 DRAINAGE

Positive surface gradients should be provided adjacent to 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 adjacent to structures. Subsurface drainage should be provided on walls below grade as discussed in a previous section.

4.9 EXTERIOR CONCRETE AND MASONRY FLATWORK

Exterior pedestrian concrete and masonry flatwork should be supported on a layer of non-expansive compacted fill if differential heave is not tolerable. The use of clayey soils within 2 feet of floor slab and concrete hardscape subgrade is not recommended. Prior to placement of concrete, the subgrade should be prepared as recommended in “Subgrade Preparation” section. The moisture content of subgrade soils should be maintained above the optimum moisture content and confirmed by a representative of GPI prior to covering. Subgrade soils allowed to dry out will require moisture conditioning, including the potential for additional processing.

4.10 PAVED AREAS

Testing on a sample of the upper soils in the planned parking area resulted in an R-value of 7. Preliminary pavement design has been based on an R-value of 5. These recommendations are based on the assumption that the pavement subgrades will consist of the existing clayey on-site soils. The following pavement sections are recommended for planning purposes only.

TRAFFIC INDEX	SECTION THICKNESS (inches)	
	ASPHALTIC CONCRETE	**AGGREGATE BASE COURSE**
4	3	7
5	3	10
6	4	12
	Portland Cement Concrete	**Aggregate Base Course**
4	6.0	4
5	6.5	4
6	7.0	4

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

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 truck traffic. Where new pavements will be constructed directly adjacent to and entering the proposed building, we recommend dowels be placed to reduce the potential for differential settlement at the transition between the building and the adjacent pavements.

The pavement base course should be compacted to at least 95 percent of maximum 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 assume 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.

4.11 STORMWATER INFILTRATION

To evaluate the infiltration characteristics of the near surface soils, we performed two field infiltration tests in accordance with methods established by the County of Los Angeles (County, 2014). Infiltration testing was performed at depths of about 10 feet below the existing ground surface at the general locations proposed for underground detention basins as provided by the project Civil Engineer.

The tests were performed in shallow borings drilled with an 8-inch hollow stem auger. The test well was constructed in the boring using a 2-inch diameter slotted well casing. The annular space between the perforated casing and the borehole was filled with No. 3 well sand.

Prior to running the tests, the soils adjacent to the well were soaked with water a minimum of five times the diameter of the well above the bottom of the boring.

The well was filled with a minimum of 18 inches of water at the initiation of the test. After confirming the initial infiltration rate, we performed the infiltration testing by taking water level measurements every 30 minutes. The infiltration rate was calculated using the lowest infiltration rate during the testing.

The adjusted infiltration rate was determined by dividing the preadjusted infiltration rate by a reduction factor that is dependent on the initial water depth and the diameter of the boring. The results of the tests indicated infiltration rates of approximately 0.0 and 0.1 inches per hour (nearly no infiltration). Detailed results of the testing are presented in Tables 1.1 and 1.2, Borehole Infiltration Test Results.

Grading at the location of the proposed underground basin is expected to be minimal and densification of the on-site soil due to grading activities is anticipated to have little effect on the permeability of the soils at the proposed depth of infiltration. The Civil Engineer should evaluate feasibility of groundwater infiltration using the infiltration rates provided and suitable factors of safety.

4.12 GEOTECHNICAL OBSERVATION AND TESTING

We recommend that a representative of GPI observe earthwork during construction to confirm that the recommendations provided in our report are applicable during construction. The earthwork activities include 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 Hello Auto 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.

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 observations and construction review will be provided by GPI during grading, excavation, and foundation construction. If field conditions during construction appear to be different than is indicated in this report, we will need to assess the impact of such conditions on our recommendations.

If construction phase geotechnical services are provided by others, the use of this report and its contents will be solely at their risk. In addition, the firm will need to accept full responsibility for geotechnical aspects of the project, including the recommendations contained herein.

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 express or implied, is included or intended in our report.

Respectfully submitted,
Geotechnical Professionals Inc.

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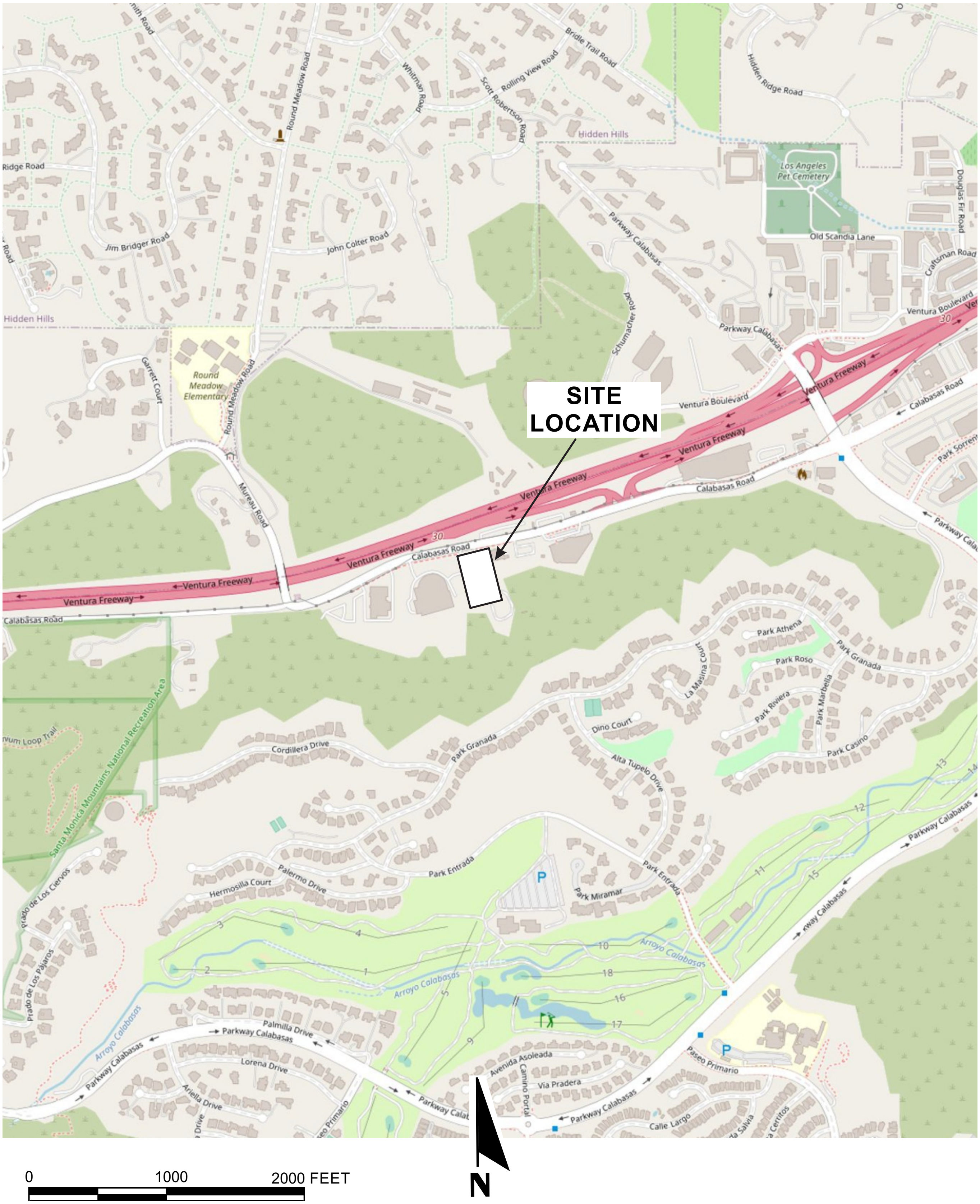


BASE PLAN REPRODUCED FROM CALTOPO © 2021

GPI GEOTECHNICAL PROFESSIONALS, INC.

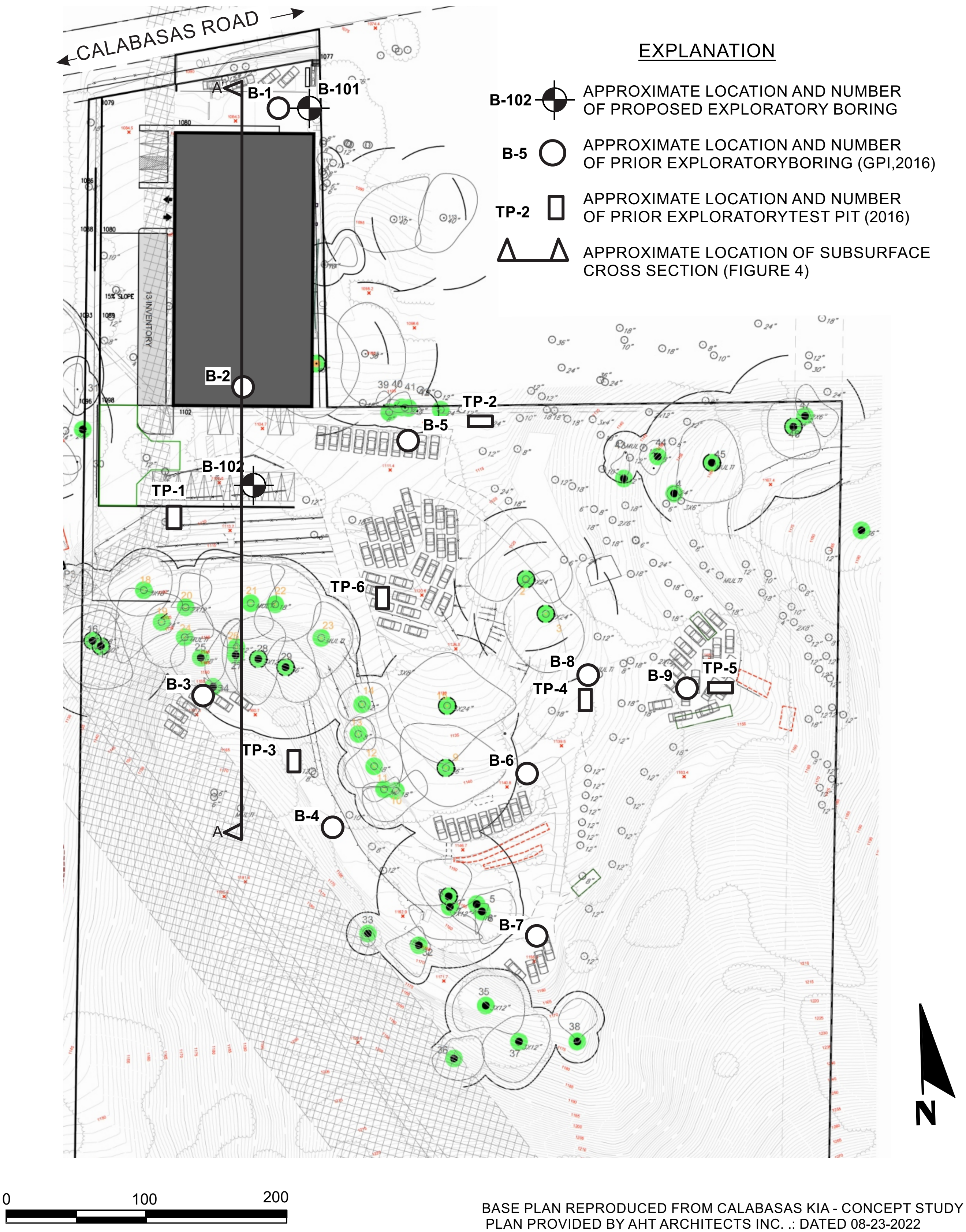
KIA CALABASAS

GPI PROJECT NO.: 3162.I

SCALE: 1" = 1000'

SITE LOCATION MAP

FIGURE 1



BASE PLAN REPRODUCED FROM CALABASAS KIA - CONCEPT STUDY PLAN PROVIDED BY AHT ARCHITECTS INC. .: DATED 08-23-2022

GEOTECHNICAL PROFESSIONALS, INC.

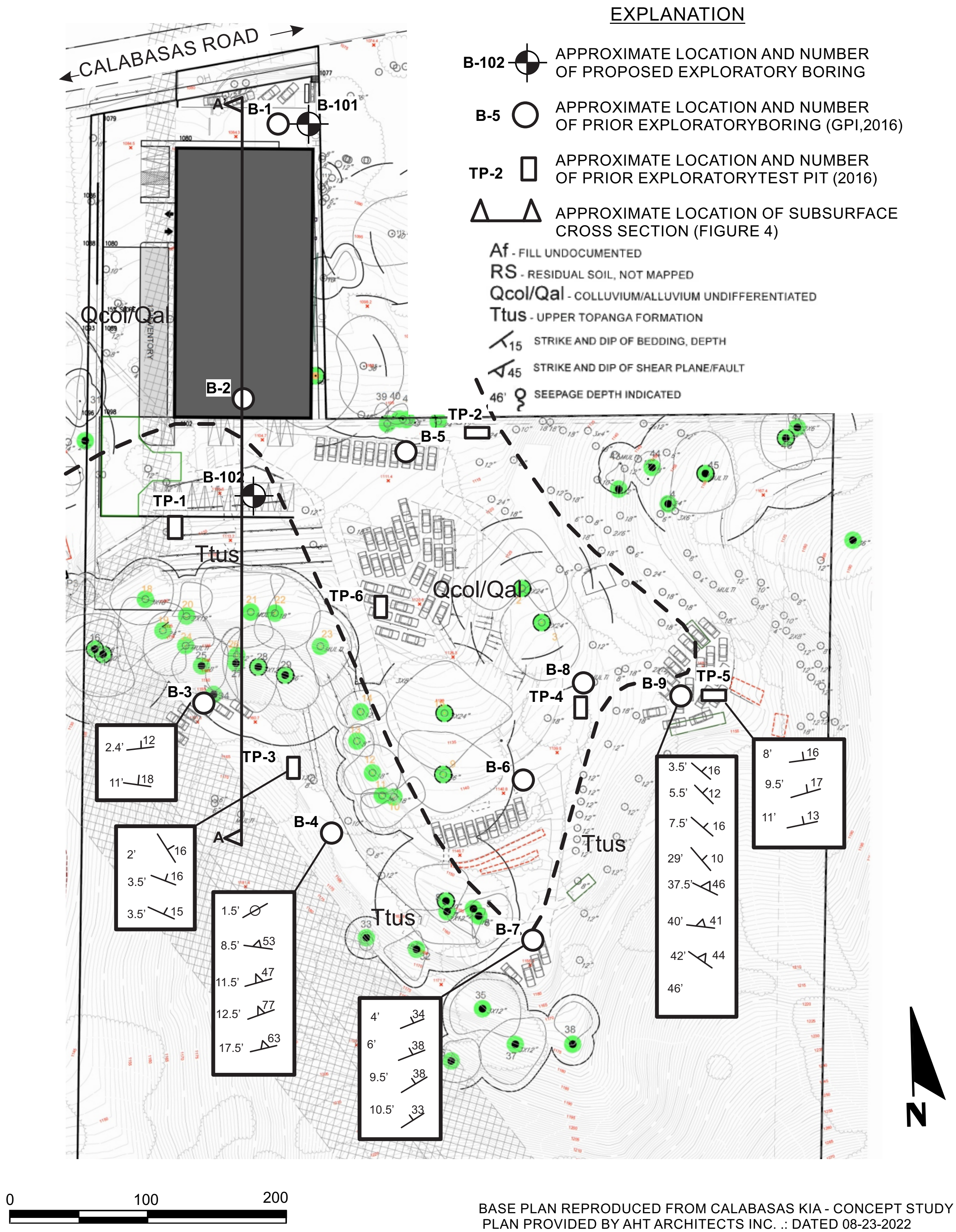
KIA CALABASAS

GPI PROJECT NO.: 3162.I

SCALE: 1" = 100'

SITE PLAN

FIGURE 2



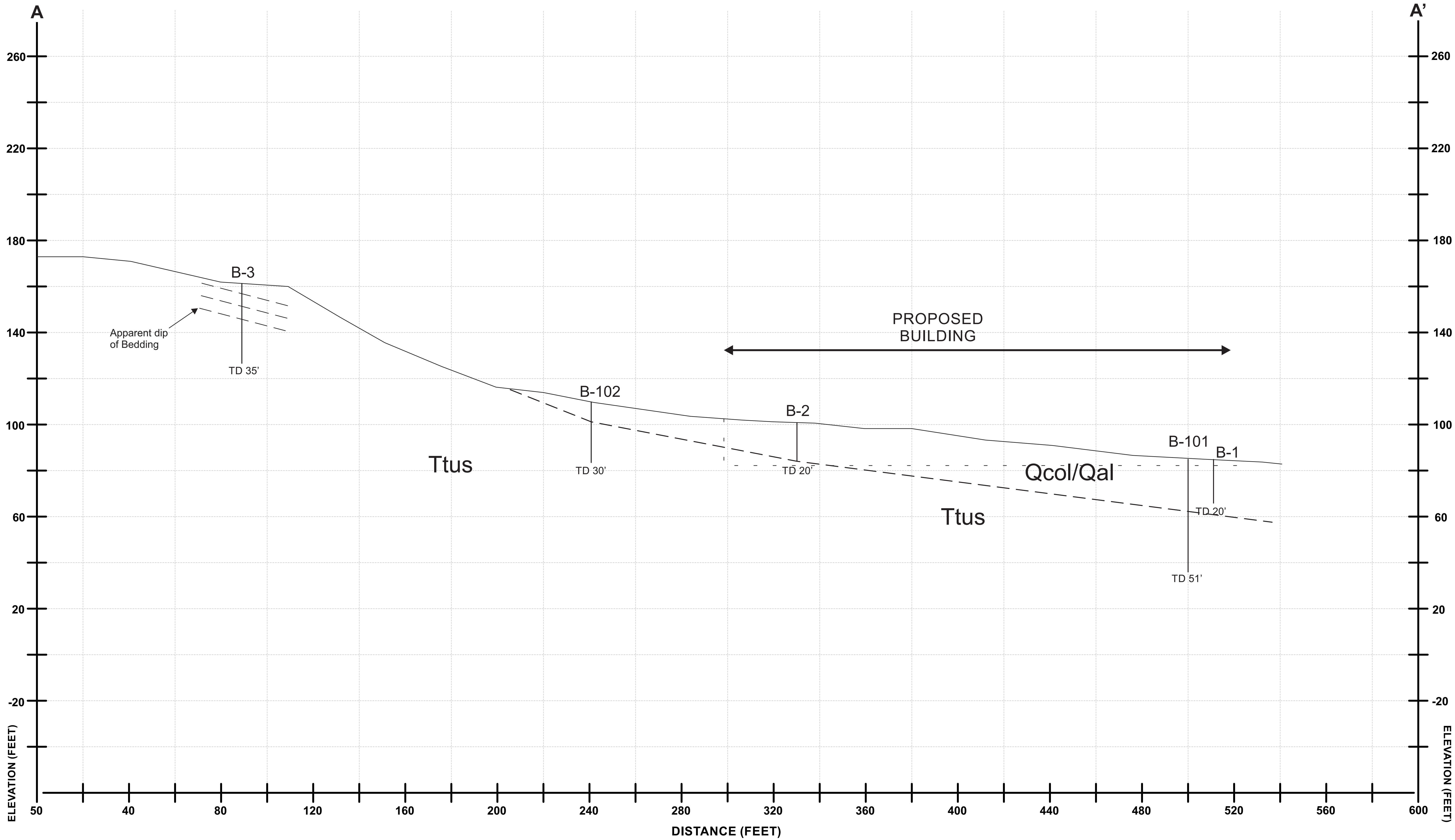
GEOTECHNICAL PROFESSIONALS, INC.

KIA CALABASAS

GPI PROJECT NO.: 3162.I | SCALE: 1" = 100'

GEOLOGIC SITE PLAN

FIGURE 3



Note: This section is based upon information obtained at borings and CPTs obtained during geotechnical investigation. The section is based upon limited geotechnical data and localized variations should be anticipated. This section is intended for descriptive purposes only.

GPI GEOTECHNICAL PROFESSIONALS, INC.

KIA CALABASAS

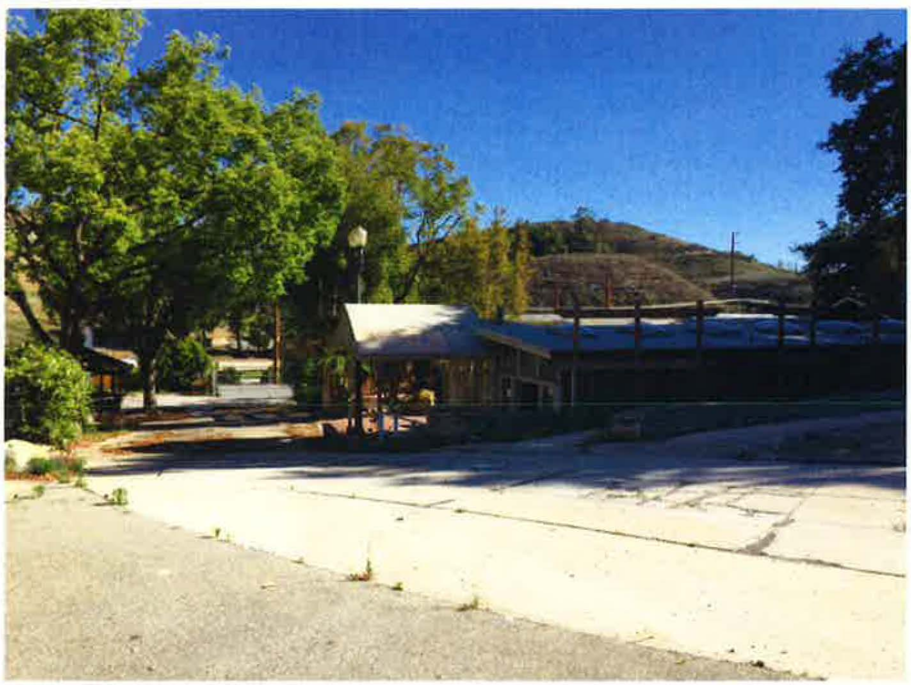
GPI PROJECT NO.: 3162.I

SCALE AS SHOWN

SUBSURFACE CROSS SECTION
A-A'

FIGURE 4

AT BORING B-5 LOOKING SOUTH TOWARDS BORING B-8



NEAR BORING B-2 LOOKING NORTH TOWARDS CALABASAS ROAD

SITE PHOTOGRAPHS TAKEN APRIL 17, 2016

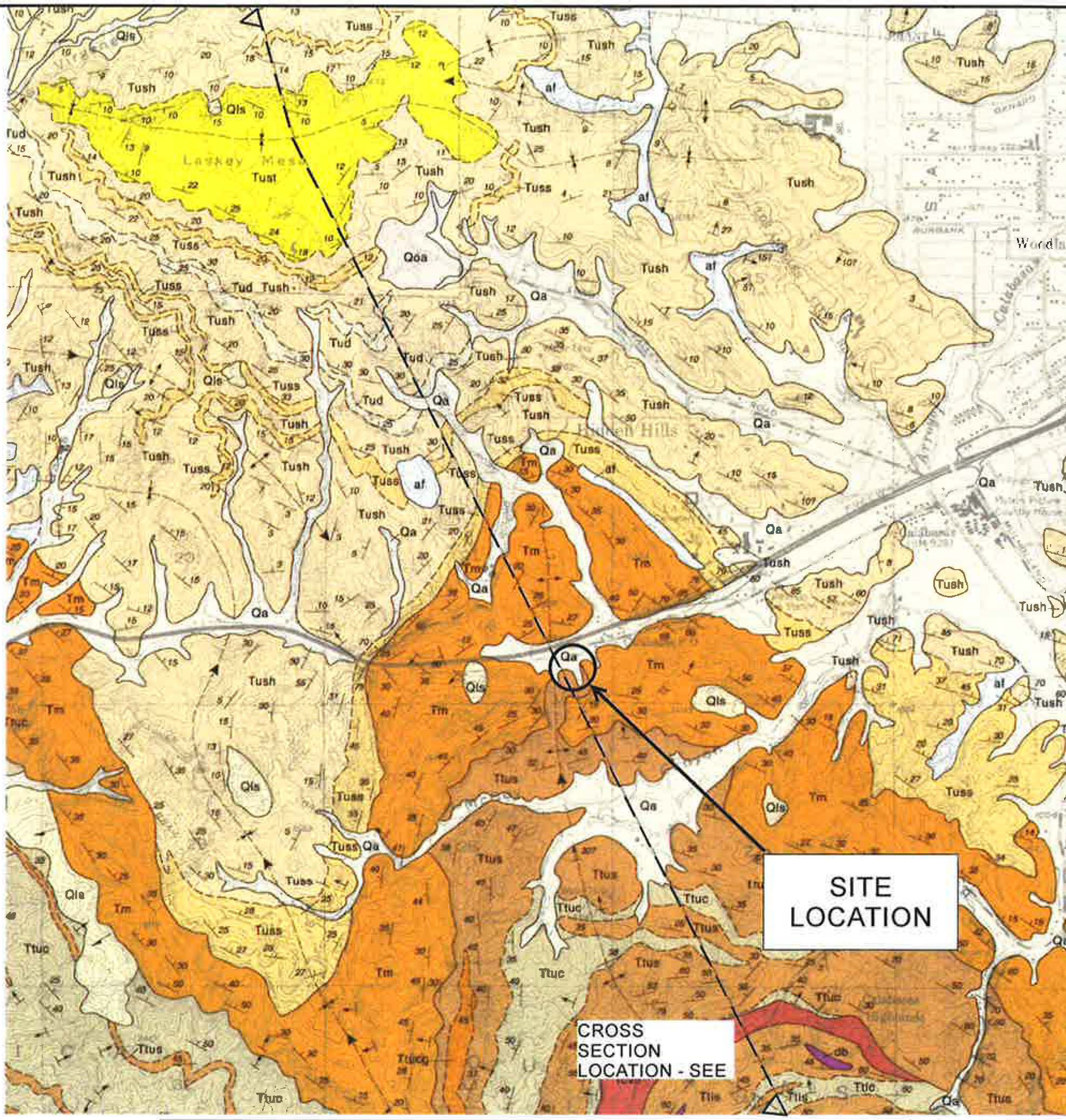
GPI GEOTECHNICAL PROFESSIONALS, INC.	SITE PHOTOGRAPHS
KIA CALABASAS	
GPI PROJECT NO. 3162.1 — NTS	FIGURE 5.1

NEAR BORING B-4 LOOKING WEST TOWARDS BORING B-3

AT BORING B-9 LOOKING WEST TOWARDS BORING B-6

SITE PHOTOGRAPHS TAKEN APRIL 17, 2016

GPI GEOTECHNICAL PROFESSIONALS, INC.		SITE PHOTOGRAPHS
KIA CALABASAS		
GPI PROJECT NO. 3162.1	NTS	FIGURE 5.2



LEGEND

MONTEREY FORMATION

(Lower part Modelo Formatin of Hoots 1931; Soper 1938; Durrell 1954; A.E.G. maps 1982; Modelo Formation of Yerkes and Campbell 1979; Weber 1984; Modelo-Monterey and lower Monterey Formation of Truex and Hall 1969, Truex 1976; ; equivalent to Monterey Formation of Dibblee 1989, in Ventura basin)

Marine, biogenic and clastic; middle and late Miocene age (late (?) Luisian and Mohnian Stages)

Tm *Gray-brown, white weathering siliceous shale, thin bedded, moderately hard with platy fracture; includes soft fissile diatomaceous shale, hard, brittle, cherty shale, and few layers of hard, yellow-weathering calcareous concretions or lenses*

Tmss *Light gray to tan, semi-friable bedded sandstone*

Tmcg *Gray cobble conglomerate of mostly granitic detritus in sandstone matrix*

SURFICIAL SEDIMENTS

af *Artificial cut and fill*

Qa *Alluvium: gravel, sand and clay of valley areas, includes gravel of stream channels gravel and sand of alluvial fans, and slope wash; undissected to slightly dissected*

UPPER TOPANGA FORMATION

(Of Durrell 1954; Topanga Formation of Soper 1938; Truex and Hall 1969; Truex 1976; Weber 1984; Calabasas Formation of Yerkes and Campbell 1979, 1980)

Marine clastic; middle Miocene age (Luisian Stage)

Ttuc *Gray claystone, bedded; crumbly with ellipsoidal fracture*

Ttus *Light gray sandstone, semi friable, thick bedded*

Ttucg *Gray conglomerate of cobbles of granitic rocks, sandstone, and volcanic rocks in sandstone matrix*

N

0 3000 6000 FEET

BASE MAP REPRODUCED FROM THE GEOLOGIC MAP OF THE CALABASAS QUADRANGLE PREPARED BY THE DIBBLEE GEOLOGICAL FOUNDATION: DATED 1992

GEOTECHNICAL PROFESSIONALS, INC.

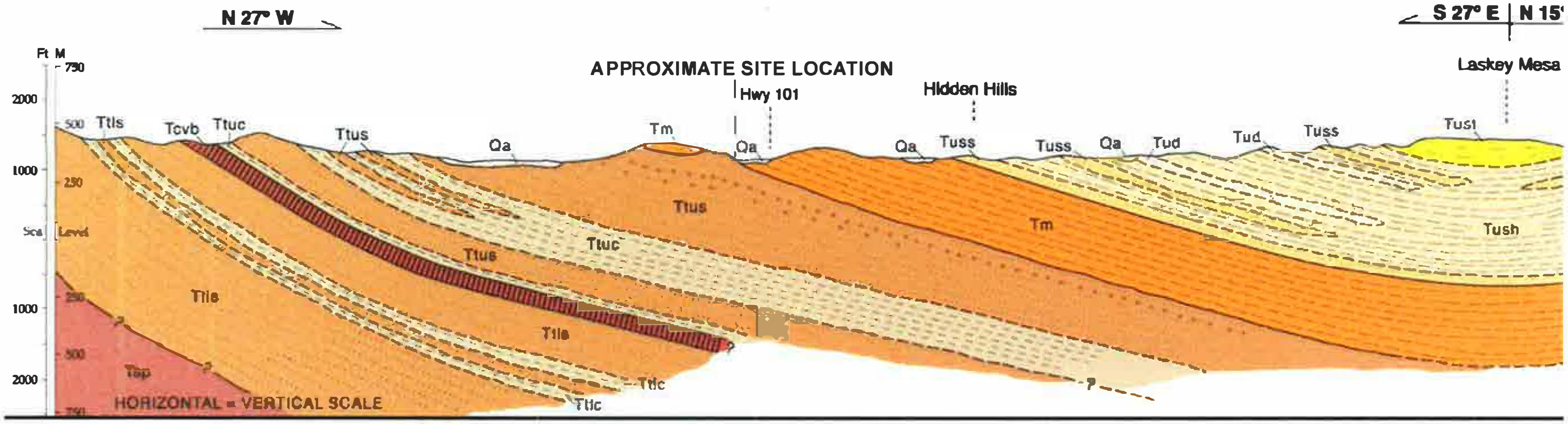
KIA CALABASAS

GPI PROJECT NO.:3162.I

SCALE: 1" = 3000'

REGIONAL GEOLOGIC MAP

FIGURE 6.1



SEE FIGURE 4.1 FOR CROSS SECTION LOCATION

LEGEND

MONTEREY FORMATION

(Lower part Modelo Formation of Hoots 1931; Soper 1938; Durrell 1954; A.L.C. maps 1982; Modelo Formation of Yerkes and Campbell 1979; Weber 1984; Modelo-Monterey and lower Monterey Formation of Truex and Hall 1969, Truex 1976; ; equivalent to Monterey Formation of Dibblee 1989, in Ventura basin)

Marine, biogenic and clastic; middle and late Miocene age (late (?) Luisian and Mohnian Stages)

Tm Gray-brown, white weathering siliceous shale, thin bedded, moderately hard with platy fracture; includes soft fissile diatomaceous shale, hard, brittle, cherty shale, and few layers of hard, yellow-weathering calcareous concretions or lenses

Tmss Light gray to tan, semi-friable bedded sandstone

Tmcg Gray cobble conglomerate of mostly granitic detritus in sandstone matrix

SURFICIAL SEDIMENTS

af Artificial cut and fill

Qa Alluvium: gravel, sand and clay of valley areas, includes gravel of stream channels gravel and sand of alluvial fans, and slope wash; undissected to slightly dissected

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(Of Durrell 1954; Topanga Formation of Soper 1938; Truex and Hall 1969; Truex 1976; Weber 1984; Calabasas Formation of Yerkes and Campbell 1979, 1980)

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Ttus Light gray sandstone, semi-friable, thick bedded

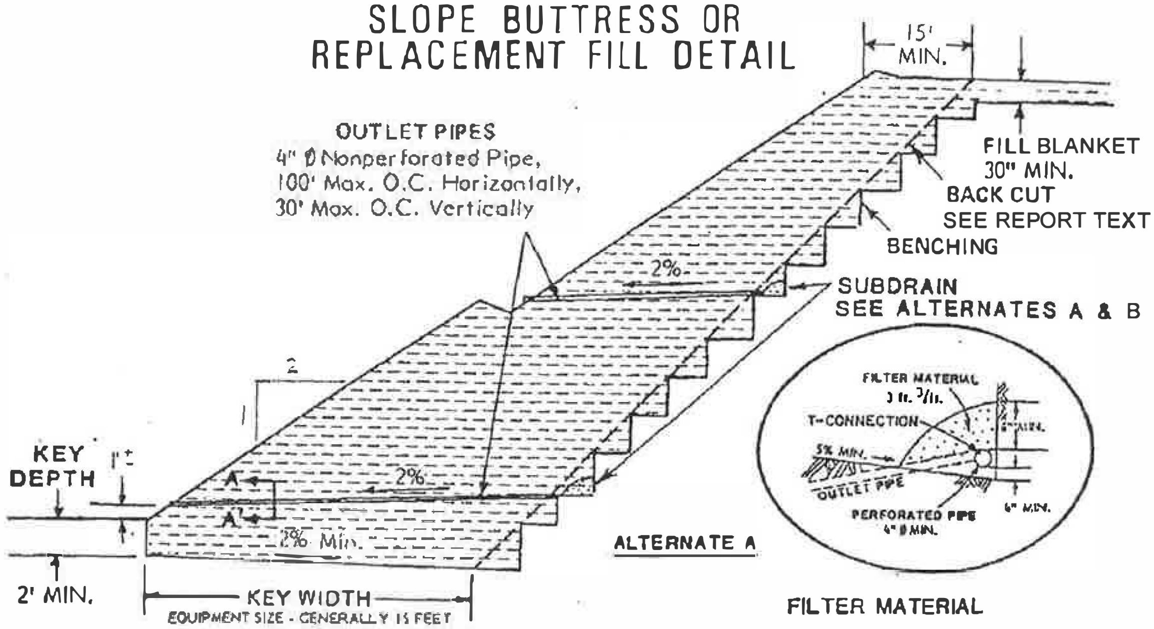
Ttucg Gray conglomerate of cobbles of granitic rocks, sandstone, and volcanic rocks in sandstone matrix

BASE MAP REPRODUCED FROM THE GEOLOGIC MAP OF THE CALABASAS QUADRANGLE
PREPARED BY THE DIBBLEE GEOLOGICAL FOUNDATION: DATED 1992

GEOTECHNICAL PROFESSIONALS, INC.

KIA CALABASAS	**REGIONAL GEOLOGIC MAP CROSS SECTION**
GPI PROJECT NO.: 3162.I SCALE: 1" = 2500'	FIGURE 6.2

SLOPE BUTTRESS OR REPLACEMENT FILL DETAIL

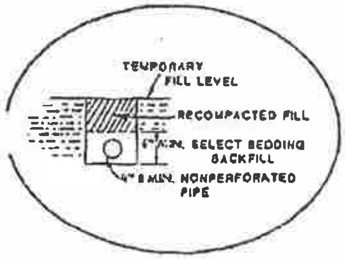


ALTERNATE A

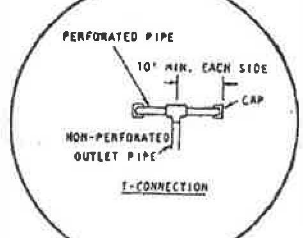
FILTER MATERIAL

Filter material shall be Class 2 permeable material per State of California Standard Specifications, or approved alternate. Class 2 grading as follows:

SIEVE SIZE	PERCENT PASSING
1"	100
3/4"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3



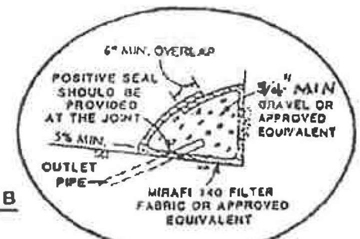
DETAIL A-A'



T-CONNECTION

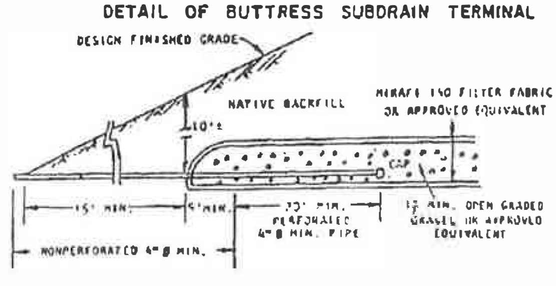
NOTES:

- Fill blanket, back cut, key width and key depth are subject to field change, per report/plans.
- Key heel subdrain, blanket drain, or vertical drain may be required at the discretion of the geotechnical consultant.
- SUBDRAIN INSTALLATION - Subdrain pipe shall be installed with perforations down or, at locations designated by the geotechnical consultant, shall be nonperforated pipe.
- SUBDRAIN TYPE - Subdrain type shall be ASTM C508 Asbestos Cement Pipe (ACP) or ASTM D2751, SDR 23.5 or ASTM D1527, Schedule 40 Acrylonitrile Butadiene Styrene (ABS) or ASTM D3034 SDR 23.5 or ASTM D1785, Schedule 40 Polyvinyl Chloride Plastic (PVC) pipe or approved equivalent.



ALTERNATE B

DETAIL OF BUTTRESS SUBDRAIN TERMINAL



GEOTECHNICAL PROFESSIONALS, INC.

KIA CALABASAS

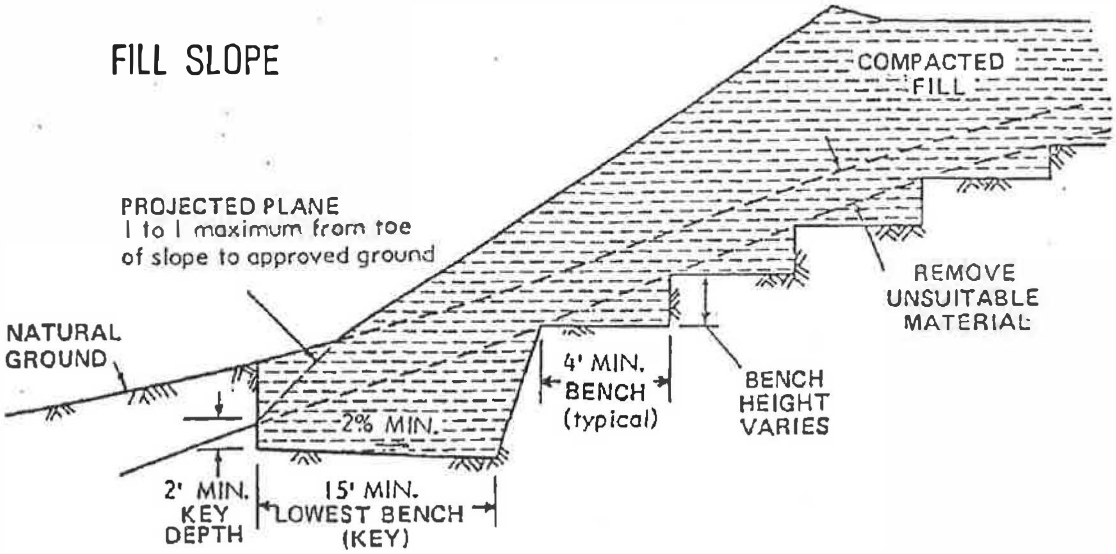
GPI PROJECT NO.: 3162.I	NO SCALE

SLOPE BUTTRESS OR REPLACEMENT FILL DETAIL

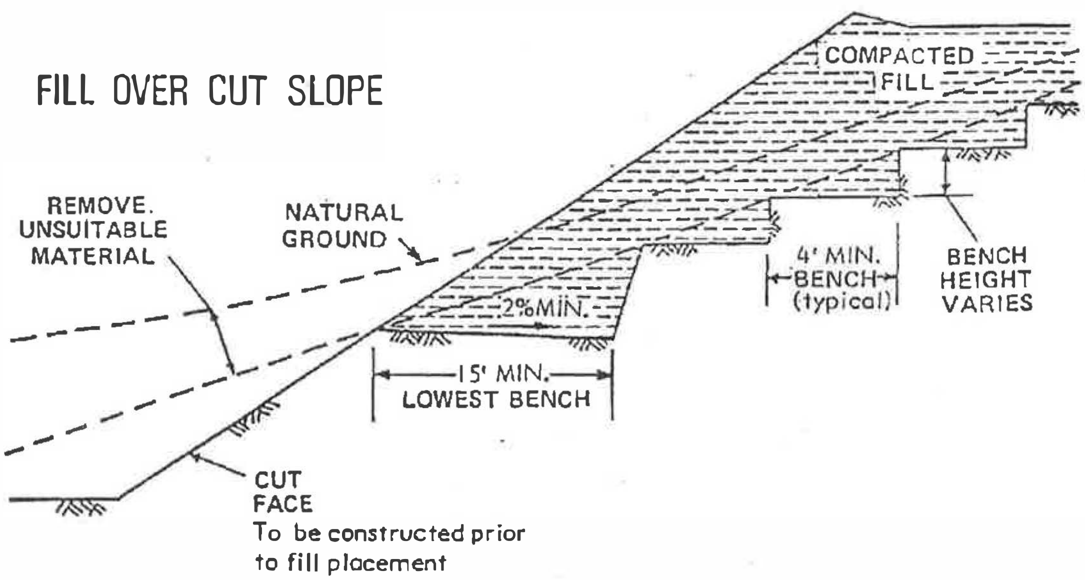
FIGURE 7

BENCHING DETAILS

FILL SLOPE



FILL OVER CUT SLOPE



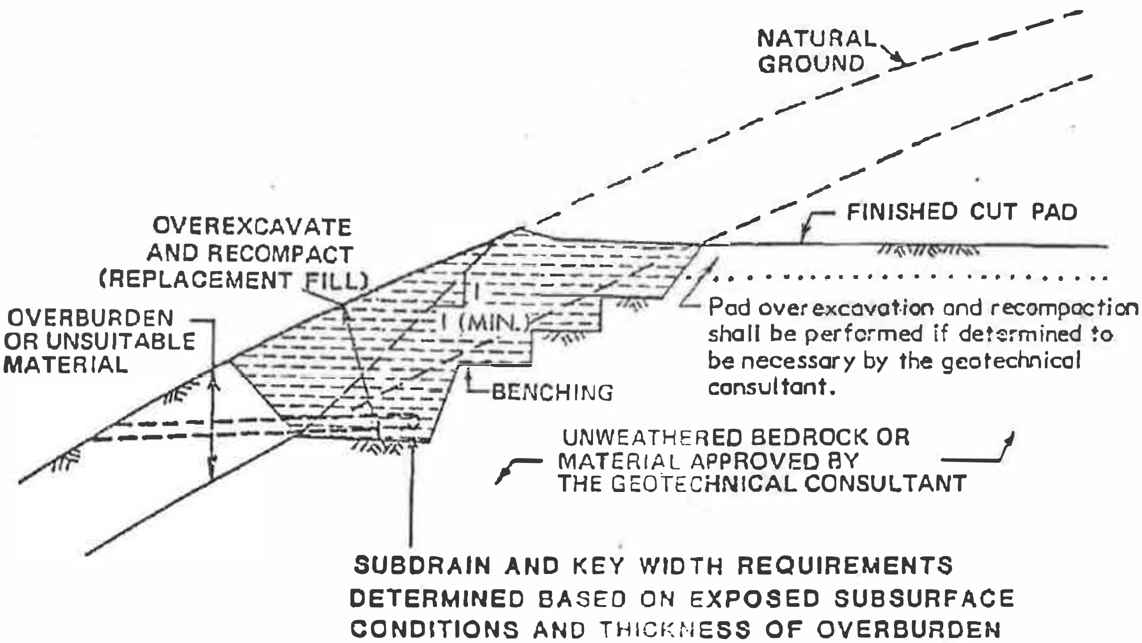
NOTES:

LOWEST BENCH: Depth and width subject to field change based on consultant's inspection.

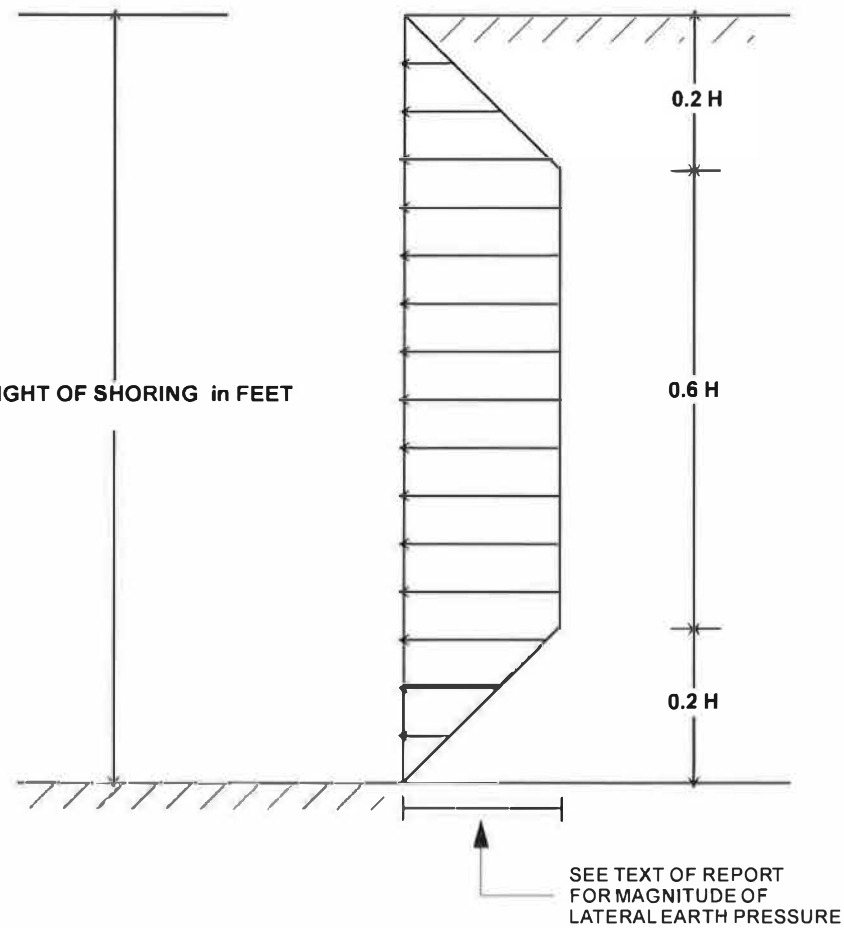
SUBDRAINAGE: Back drains may be required at the discretion of the geotechnical consultant.

GPI GEOTECHNICAL PROFESSIONALS, INC.		BENCHING DETAIL
KIA CALABASAS		
GPI PROJECT NO.: 3162.I	NO SCALE	FIGURE 8

SIDE HILL
CUT PAD DETAIL



GPI GEOTECHNICAL PROFESSIONALS, INC.		SIDE HILL CUT PAD DETAIL
KIA CALABASAS		
GPI PROJECT NO.: 3162.I	NO SCALE	FIGURE 9



It should be noted that the provided lateral earth pressues do not include hydrostatic pressures

GEOTECHNICAL PROFESSIONALS, INC.

KIA CALABASAS

GPI PROJECT NO.: 3162.I

NO SCALE

LATERAL EARTH PRESSURES FOR TIE-BACK SHORING

FIGURE 10

TABLE 1.1

BOREHOLE INFILTRATION TEST RESULTS
Los Angeles County Method (GS200.1, 06/01/11)

Project No. 2730.I — Date: 9/13/16
Client: Calabasas Nissan — Test Date 5/20/16
By AS

NOTE: Slowest or average rate from percolation testing used to calculate infiltration rate

Test Well/ Adj. Boring	Test Duration (min) Δt	Depth of Well (ft)	Depth to Water Initial* (ft)	Depth to Water Final (ft)	Initial Water Depth (ft) d_1	Water Level Drop (ft) Δd	Hole Diameter (inches) *DIA*	Preadjusted Percolation Rate (in/hr)	Reduction Factor R_f**	Infiltration Rate (in/hr) I_t
P-1/B-1	30	9.95	7.82	7.82	2.13	0.00	8	0.0	7.4	0.0
P-1/B-1	30	9.95	7.82	7.82	2.13	0.00	8	0.0	7.4	0.0
P-1/B-1	30	9.95	7.82	7.82	2.13	0.00	8	0.0	7.4	0.0
P-1/B-1	30	9.95	7.82	7.82	2.13	0.00	8	0.0	7.4	0.0
P-1/B-1	30	9.95	7.82	7.82	2.13	0.00	8	0.0	7.4	0.0
P-1/B-1	30	9.95	7.82	7.82	2.13	0.00	8	0.0	7.4	0.0
P-1/B-1	30	9.95	7.82	7.82	2.13	0.00	8	0.0	7.4	0.0
P-1/B-1	30	9.95	7.82	7.82	2.13	0.00	8	0.0	7.4	0.0

* Test well pipe may be higher than ground surface

** $R_f = ((2d_1 - \Delta d)/DIA) + 1$

TABLE 1.2

BOREHOLE INFILTRATION TEST RESULTS
Los Angeles County Method (GS200.1, 06/01/11)

Project No. 2730.I
Client: Calabasas Nissan
By AS

Date: 9/13/16
Test Date 5/20/16

NOTE: Slowest or average rate from percolation testing used to calculate infiltration rate

Test Well/ Adj. Boring	Test Duration (min) Δt	Depth of Well (ft)	Depth to Water Initial* (ft)	Depth to Water Final (ft)	Initial Water Depth (ft) d_1	Water Level Drop (ft) Δd	Hole Diameter (inches) *DIA*	Preadjusted Percolation Rate (in/hr)	Reduction Factor R_f**	Infiltration Rate (in/hr) I_t
P-2/B-2	30	9.42	7.42	7.48	2.00	0.05	8	1.3	6.9	0.2
P-2/B-2	30	9.42	7.42	7.48	2.00	0.06	8	1.4	6.9	0.2
P-2/B-2	30	9.42	7.42	7.47	2.00	0.05	8	1.2	6.9	0.2
P-2/B-2	30	9.42	7.42	7.46	2.00	0.04	8	0.8	6.9	0.1
P-2/B-2	30	9.42	7.40	7.44	2.02	0.04	8	1.0	7.0	0.1
P-2/B-2	30	9.42	7.40	7.44	2.02	0.04	8	0.8	7.0	0.1
P-2/B-2	30	9.42	7.40	7.44	2.02	0.04	8	0.8	7.0	0.1
P-2/B-2	30	9.42	7.40	7.44	2.02	0.04	8	1.0	7.0	0.1

* Test well pipe may be higher than ground surface

** $R_{f=((}2d_1-\Delta d)/DIA)+1$

APPENDIX A

APPENDIX A

EXPLORATORY BORINGS AND TEST PITS

The subsurface conditions at the site were investigated by drilling and sampling a total of 11 exploratory borings and 6 test pits. Two of the hollow stem auger borings (B-101 and B-102) were performed as part of our current field investigation with the remainder being performed as part of our 2016 investigation. The borings were advanced to depths between 20 and 51 feet below the existing ground surface and the test pits were advanced to depths ranging from 5 to 13.5 feet. Four of the borings were terminated prior to the planned depth because of refusal in the dense bedrock. The exploration locations are shown on the Site Plan, Figure 2.

The borings were drilled using bucket EZ-bore auger, limited access auger rig, and truck-mounted hollow-stem auger equipment. Borings B-7 and B-9 were drilled using a large diameter bucket auger. Relatively undisturbed samples were obtained using a brass-ring lined sampler (ASTM D3550). The drive sampler has an inside diameter of 2.42 inches and an outside diameter of 3.25 inches. The brass rings have an inside diameter of 2.42 inches and a height of 1-inch. In addition, relatively disturbed bulk samples were obtained at various depths. The drive sampler is driven into the soil using a drop of 12 inches with a driving weight of the Kelly bar as shown.

RIG TYPE	DEPTH (ft)	KELLY BAR DRIVING WEIGHT (lbs)
Bucket Auger 24" (EZ-Bore)	0-29 30-57 58-85 86 – deeper	3615 2395 1310 450
Bucket Auger 24"	0-24 25 – deeper	1590 825

The number of blows needed to drive the sampler was recorded as the penetration resistance. It should be noted that the number of blows, in this case, is much lower than the standard penetration resistance because of the greater driving weight. At select locations bulk samples were taken from the auger's cuttings and placed in sealed containers.

Borings B-1, B-2, B-5, B-6, B-8, B-101, and B-102 were drilled using the hollow-stem auger. Relatively undisturbed samples were obtained using a brass ring-lined sampler driven into the soil by a 140-pound "free-fall" hammer dropping 30 inches. The number of blows needed to drive the sampler into the soil was recorded as the penetration resistance. Due to the use of a "free-fall" hammer (rather than a hammer attached to a rope), the blow-counts recorded with the (D) sampler are approximately equal to the Standard Penetration Test blow-count (N_{60}). Drives less than 12 inches are denoted with the blow count per length of drive.

At selected locations, disturbed samples were obtained using a split-spoon sampler by means of the Standard Penetration Test (SPT, ASTM D 6066). The spoon sampler was driven into the soil by a 140-pound hammer dropping 30 inches. After an initial seating drive of 6 inches, the number of blows needed to drive the sampler into the soil a depth of 12 inches was recorded as the penetration resistance. These values are the raw uncorrected blowcounts.

Borings B-3 and B-4 were drilled using the limited access spiral auger rig. Relatively undisturbed samples were obtained using a brass-ring lined sampler (ASTM D3550). The drive sampler has an inside diameter of 2.42 inches and an outside diameter of 3.25 inches. The brass rings have an inside diameter of 2.42 inches and a height of 1-inch. In addition, relatively disturbed bulk samples were obtained at various depths. The drive sampler is driven into the soil using a drop of 12 inches with a driving weight of the Kelly bar. The number of blows needed to drive the sampler was recorded as the penetration resistance. It should be noted that the number of blows, in this case, is much lower than the standard penetration resistance because of the greater driving weight. At select locations bulk samples were taken from the auger's cuttings and placed in sealed containers.

The test pits were performed using a backhoe with a 24-inch wide bucket. The test pits were excavated to depths of 5 to 16 feet to expose the subsurface materials for observation and sampling by our Certified Engineering Geologist. Upon completion, the test pits were backfilled with the excavated materials.

The field exploration for the investigation was 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 boring 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 and test pits are presented in Figures A-1 through A-17 in this appendix. Borings B-3, B-4, B-7 and B-9, as well as the six test pits, were down-hole logged by our Engineering Geologist to evaluate bedding conditions.

The boring locations were laid out in the field by measuring from existing features at the site. Upon completion, the borings were backfilled with the excavated soil cuttings. The ground surface elevation at the boring location was estimated from Google Earth, and the concept study plan from AHT Architects, and should be considered approximate.

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.	
			B	0	5-INCH AC	
					Fill: **CLAY (CL)** dark brown, very moist, very stiff, with sand, with siltstone gravel fragments	
18.5		39	D			1085
18.5	93	27	D	5	Natural COLLUVIUM (Qc): **CLAY (CL)** dark brown, very moist, very stiff, with sand, with siltstone fragments, abundant white caliche	
20.8	97	27	D		@ 7 feet, trace siltstone gravel fragments	1080
13.6	98	24	D	10	**SANDY CLAY (CL)** dark brown, moist, very stiff, trace siltstone gravel, abundant caliche, minor porosity	1075
25.4	90	47	D	15	@ 15 feet, wet, hard, with gravel	1070
15.8 8.9	105 111	50/5"	D	20	@ 20 feet, moist	
					UPPER TOPANGA FORMATION (Ttus): **SANDSTONE** brown, moist, very dense, with gravel, weathered	1065
19.4		43	S	25	**WEATHERED SILTSTONE** light brown, wet, medium dense	1060
15.8	80	50/5"	D	30	@ 30 feet, brown, moist, hard	1055
12.5		68	S	35	**SANDSTONE** grey, very moist, very dense, mottled with orange-brown iron oxide	1050

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

DATE DRILLED: 11-10-22

EQUIPMENT USED: 8 " HOLLOW STEM AUGER

GROUNDWATER LEVEL (ft): NOT ENCOUNTERED

GPI

PROJECT NO.: 3162.I

KIA CALABASAS

LOG OF BORING NO. B-101

FIGURE A-1

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)		*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
	15.5	106	50/3"	D	40		@ 40 feet, wet	
								1045
	14.3		50/6"	S	45		**SILTSTONE** grey, moist, hard	
								1040
	12.0	120	82	D	50		**SANDSTONE** grey, wet, very dense, trace orange-brown	
							Total Depth 51 feet	

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

DATE DRILLED:
11-10-22

EQUIPMENT USED:
8 " HOLLOW STEM AUGER

GROUNDWATER LEVEL (ft):
NOT ENCOUNTERED

GPI

PROJECT NO.: 3162.I

KIA CALABASAS

LOG OF BORING NO. B-101

FIGURE A-1

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
			B	0	4-INCH AC OVER 5-INCH BASE	1105
					Fill: **SANDY SILT (ML)** brown, moist, stiff	
12.0	89	13	D		Natural COLLUVIUM (Qc): **SILTY SAND (SM)** brown, moist, loose	
9.3	102	37	D	5	**SANDY SILT (ML)** brown, slightly moist, very stiff	1100
6.5	110	51	D		@ 7 feet, hard	
21.4	99	41	D	10	UPPER TOPANGA FORMATION (Ttus): **SILTSTONE** light brown, very moist, very stiff, weathered	1095
6.0	114	50/4"	D	15	**SANDSTONE** light grey, slightly moist, very dense	1090
14.3	115	50/3"	D	20	**SILTSTONE** grey, moist, hard	1085
7.9		50/4"	S	25	**SANDSTONE** brown, moist, very dense	1080
8.6	102	50/4"	D	30		1075
					Total Depth 31 feet	

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

DATE DRILLED:
11-10-22

EQUIPMENT USED:
8 " HOLLOW STEM AUGER

GROUNDWATER LEVEL (ft):
NOT ENCOUNTERED

PROJECT NO.: 3162.I

KIA CALABASAS

LOG OF BORING NO. B-102

FIGURE A-2

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	3" PCC over 3" AB	
					Fill: **SANDY CLAY (CL)** dark brown, very moist, hard	
21.6	101	18	D			
			B			80
21.6	97	26	D	5	Natural: COLLUVIUM (Qc):**SANDY CLAY (CL)** dark brown, very moist, very stiff	
19.9	100	23	D		Below 7 feet, hard	
						75
19.8	102	30	D	10		
18.7		19	S			
30.1		20	S		Below 13 feet, wet	70
27.2		18	S	15		
37.8		25	S			65
				20	Total Depth 20 feet Well installed to depth of 10 feet	

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

DATE DRILLED:
5-18-16
EQUIPMENT USED:
8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 3162.I
KIA CALABASAS

LOG OF BORING NO. B-1

FIGURE A-3

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
					0	Fill: **SANDY CLAY (CL)** brown, moist, stiff	
	9.9	87	12	D			
				B			100
					5		
	14.2	92	16	D		@ 5 feet, hard	
						Natural: COLLUVIUM (Qc): **SILTY SAND (SM)** brown, slightly moist, medium dense	
	9.1	92	16	D			
							95
					10		
	9.7	103	45	D		UPPER TOPANGA FORMATION (Ttus):**SANDSTONE** light brown, slightly moist, very dense	
	8.1		53	S			
	5.2		50/5"	S			90
					15		
	5.8		50/5"	S			
	6.3		50/5"	S			85
					20	Total depth 20 feet Well installed to depth of 9.5 feet	

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

DATE DRILLED:
5-18-16

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

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KIA CALABASAS

LOG OF BORING NO. B-2

FIGURE A-4

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
				0	Fill: **SILT (ML)** brown, loose, dry	160
14.7			B		Natural: UPPER TOPANGA FORMATION (Ttus): **SILTSTONE** yellow brown and olive grey, moist @ 2.4 feet, B: N82E, 12NW, continuous 1-2" thick tuffaceous bed Below 4 feet, very hard	
12.0			B	5		155
11.8			B			
15.8			B	10	@ 10 feet, very moist @ 11 feet, B: N84W, 18NE	150
21.5			B		@ 13 feet, wet	
2.8			B	15	**CONGLOMERATE** hard, dry, cemented gravel bed, well rounded, fine to medium **SANDSTONE** hard, grey, dry, fine to medium grained, massive throughout, moderately cemented	145
2.3			B	20		140
6.7			B	25	Below 25 feet, slightly moist 26 to 28.5 feet, orange brown, oxidized zone	135
6.7			B	30		130
8.1			B	35	Total Depth 35 feet Refusal on hard, cemented layer	

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

DATE DRILLED:
5-24-16

EQUIPMENT USED:
24 " Limited Access Auger

GROUNDWATER LEVEL (ft):
Not Encountered

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KIA CALABASAS

LOG OF BORING NO. B-3

FIGURE A-5

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
				B	0	Fill: **GRAVEL (GP)** with scattered organic debris	160
	3.2	96	42/12"	D		Natural: UPPER TOPANGA FORMATION (Ttus):**SANDSTONE** orange brown, dry, highly oxidized, fine grained, massive, caliche in thin veinlets, fracture fillings	
	3.0			B	5	Below 1.5 feet, J: N60E, 90deg, grey, fractures iron oxide stained and filled with caliche, joints 2-4" spacing	155
	3.0			B			
						@ 8.5 feet, FZ: N81W, 53NE, steeply dipping fault/shear zone with sandstone above and gray siltstone below	
	14.8			B	10	**SILTSTONE** grey, moist, massive, zone of shearing	150
						@ 11.5 feet, S: N73E, 47NW	
	19.1	109	20/11"	D		@ 12.5 feet, N70E, 77NW Below 13 feet, wet	
					15		145
	20.3	109	26/12"	D			
						@ 17.5 feet, S: N72E, 63NW, slicken sides sub horizontal	
	18.7	109	20/12"	D	20		140
						Total Depth 22 feet Refusal on hard, cemented layer	

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

DATE DRILLED:
5-24-16

EQUIPMENT USED:
24 " Limited Access Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 3162.I

KIA CALABASAS

LOG OF BORING NO. B-4

FIGURE A-6

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
			B	0	2" AC over 4" AB	110
25.4	70	13	D		Fill: **SANDY CLAY (CH)** brown with white, very moist, stiff, caliche	
19.7	67	19	D	5		105
19.5	84	32	D		Natural: COLLUVIUM (Qc):**SANDY CLAY (CL)** greyish brown with white, wet, very stiff, caliche shale fragments	
19.4	94	46	D	10	@ 10 feet, slightly moist, hard	100
27.7	84	46	D			
				15		95
35.9	77	34	D		UPPER TOPANGA FORMATION (Ttus):**SILTSTONE** white with brown streaks, wet, very stiff, caliche	
21.3	98	48	D	20	@ 20 feet, hard	90
15.2	115	97/11"	D	25	@ 25 feet, greyish brown, very moist, hard, trace caliche	85
15.6	111	50/5"	D	30		80
					Total depth of 31 feet	

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

DATE DRILLED:
5-19-16
EQUIPMENT USED:
8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft):
Not Encountered

GPI

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KIA CALABASAS

LOG OF BORING NO. B-5

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	3" AC over 4" AB	
14.7	79	10	D		Fill: **SANDY CLAY (CH)** dark brown, slightly moist, firm, with roots, trace gravel	
						135
12.1	78	20	D	5	Natural: COLLUVIUM (Qc): **SILT (ML)** dark brown with white, moist, stiff, porosity, trace sand, caliche	
9.6	85	24	D		@ 7 feet, very stiff	
						130
11.2	95	80/10"	D	10	UPPER TOPANGA FORMATION (Ttus):**SILTSTONE** light brown with white, moist, hard	
10.3	104	90	D			
						125
				15	@ 15 feet light brown, trace sand	
9.1	107	50/6"	D		@ 16 feet, slightly moist	
						120
7.8	92	50/3"	D	20	**SANDSTONE** light brown, moist, hard	
						115
9.1 5.8	99	50/1"	D	25		
						110
8.8	105	50/3"	D	30	@ 30 feet, trace gravel	
						105
18.1	118	50/3"	D	35	**SANDY SILTSTONE** light brown, wet, hard	
						100
7.2	124	50/4"	D		Total depth of 40 feet	

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

DATE DRILLED:
5-19-16

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
34

GPI

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KIA CALABASAS

LOG OF BORING NO. B-6

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	Fill:**SILTY CLAY** light brown, dry	
						Natural: COLLUVIUM (Qc):**SILTY CLAY (CL)** dark brown, moist, stiff, colluvium, with sparse shale fragments,	150
						UPPER TOPANGA FORMATION (Ttus):**SILTSTONE** grey with whitish caliche, massive-poorly bedded, weathered, hard, B:N70E, 34NW	
	6.2	114	5/9"	D	5		
	16.4	108	5/12"	D		**SANDSTONE** light brownish grey, slightly moist, fine grained, massive, very tight, no visible fractures	145
						SILTSTONE grey, very moist, very hard, poorly bedded B: N66E, 38NW @ 9.5 feet B: N61E, 38NW @ 10.5 feet, B: N56E, 33NW	
	16.5	108	5/12"	D	10		
						SANDSTONE yellow brown, very moist, hard, massive	140
	12.7	107	5/6"	D			
					15		
	12.3	115	5/8"	D			
						Total depth of 18 feet Refusal on hard, cemented layer	135

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

DATE DRILLED:
5-18-16

EQUIPMENT USED:
28 " EZ-Bore

GROUNDWATER LEVEL (ft):
Not Encountered

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KIA CALABASAS

LOG OF BORING NO. B-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.	
			B	0	3" AC over 4" AB	
11.0	85	21	D		Natural: COLLUVIUM (Qc):**SANDY CLAY (CL)** dark brown with white, slightly moist, stiff, porosity	135
14.8	100	57	D	5	UPPER TOPANGA FORMATION (Ttus):**SILTSTONE** light brown with white, moist, hard, weathered, caliche	
6.2	97	34	D		@ 7 feet, slightly moist, very stiff, with sand	130
5.9	98	84/12"	D	10	Below 10 feet, hard, dry	
4.8	86	67/10"	D		Below 13 feet, light yellow brown, no caliche	125
4.8	94	68/10"	D	15		120
24.1	97	50	D	20	@ 20 feet, wet	115
11.2	107	50/5"	D	25	**SILTSTONE** light brownish yellow, slightly moist, hard	110
10.5	95	50/3"	D	30	**SANDSTONE** light brown, very moist, hard	
					Total Depth 31 feet	

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

DATE DRILLED:
5-19-16

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
31

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LOG OF BORING NO. B-8

FIGURE A-10

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)
				0
10.9	84	5/6"	D	
8.3	86	5/6"	D	5
13.5	100	4/6"	D	
15.8	100	5/6"	D	10
4.2	99	4/6"	D	
				15
0.8	234	5/10"	D	
6.3	83	4/5"	D	20
6.6	89	5/6"	D	25
15.6	110	5/6"	D	30
14.3	116	9/12"	D	35

DESCRIPTION OF SUBSURFACE MATERIALS

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

DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
3" AC over 5" AB	
UPPER TOPANGA FORMATION (Ttus): **SILTSTONE** mottled grey and brown, moist, hard, massive, vaguely bedded, with whitish caliche on fracture surfaces	145
@ 3.5 feet, B: N52W, 16NE	
@ 5.5 feet, B: N53W, 12NE	
@ 7.5 feet, B: N53W, 13NE	140
SANDSTONE grey, dry, fine to medium grained, massive, poorly cemented, easily friable, micaceous	135
Below 13.5 feet, yellow brown, tight, unfractured, trace rounded gravel	
	130
CONGLOMERATE slightly moist, hard, cemented, massive, well rounded with cobble	
SANDSTONE yellow brown, moist, tight, fine grained, massive, unfractured	125
	120
CONGLOMERATE cemented bed, clasts rounded, undulatory channel into sandstone below Generalized Bedding: N50W, 10NE	
SANDSTONE grey, wet, fine grained, massive, tight	115
From 35 to 36.5 feet, carbonate cemented zone, very hard, irregular	
@ 37.5 feet, 1/4-inch thick clayey shear zone, irregular S: N65W, 46NE	110

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

DATE DRILLED: 5-18-16

EQUIPMENT USED: 28 " EZ-Bore

GROUNDWATER LEVEL (ft): 46

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KIA CALABASAS

LOG OF BORING NO. B-9

FIGURE A-11

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
	15.0	119	11/12"	D	40	**SANDSTONE** grey, wet, fine grained, massive, tight @ 40 feet, S: N51W, 41NE, sheared clayey zone, grey @ 42 feet, S: N55W, 44NE, oxidation along shear surface, continuous, thin clay shear	105
	12.2	111	10/6"	D	45		100
						Total Depth 49 feet	

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

DATE DRILLED:
5-18-16
EQUIPMENT USED:
28 " EZ-Bore
GROUNDWATER LEVEL (ft):
46

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LOG OF BORING NO. B-9

FIGURE A-11

PROBE (in)	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
					0	Fill: **SILTY SAND (SM)** yellow brown, slightly moist, with rootlets, 3/4" PVC water lines, porous	
						Natural: COLLUVIUM (Qc) **SILTY SAND (SM)** orange brown, slightly moist, porous, with roots to 1/2-inch diameter Gradational contact with sandstone below	110
					5		
						UPPER TOPANGA FORMATION (Ttus):**SANDSTONE** yellow and orange brown, fine grained, irregular discontinuous pebble and cobble beds, slightly moist, very dense	
						Total Depth 6 feet	

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

DATE DRILLED:
5-24-16
EQUIPMENT USED:
24" Backhoe
GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 3162.I
KIA CALABASAS

LOG OF BORING NO. TP-1

FIGURE A-12

PROBE (in)	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					0	Fill: **SILTY CLAY (CH)** brown, slightly moist to moist, firm to stiff, with sand and whitish shale fragments, trace porosity with caliche as small irregular masses, tubules	110
					5		105
					10		
						Natural: COLLUVIUM (Qc): **SILTY CLAY (CL)** dark brown, moist, stiff, with shale fragments, calcium abundant as irregular tubules, few roots	100
						Total Depth 13.5 feet	

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

DATE DRILLED:
5-24-16
EQUIPMENT USED:
24" Backhoe
GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 3162.I
KIA CALABASAS

LOG OF BORING NO. TP-2

FIGURE A-13

PROBE (in)	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: **CLAYEY SILT (ML)** brown, dry to slightly moist, loose, with shale fragments, crumbly with roots to 1-inch diameter	160
					5	UPPER TOPANGA FORMATION (Ttus):**SILTSTONE** yellow brown and grey, hard, massive, vaguely bedded, moderately fractured @ 2 feet, B: N36W, 16NE @ 3.5 feet, B: N71W, 15NE, white, weathered 1 to 2-inch thick tuff bed, continuous B: N66W, 15NE	
						Total Depth 5 feet	

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

DATE DRILLED: 5-24-16
EQUIPMENT USED: 24" Backhoe
GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 3162.I
KIA CALABASAS

LOG OF BORING NO. TP-3

FIGURE A-14

PROBE (in)	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					0	Natural: COLLUVIUM (Qc): **SILTY CLAY (CL)** dark brown, slightly moist, firm to stiff, porous, few roots, with shale fragments Below 1.5 feet, abundant white caliche in thin veinlets	135
					5		
					10	UPPER TOPANGA FORMATION (Ttus): **SILTSTONE** olive grey, very tight, highly weathered into 1/2 to 1-inch diameter pieces with caliche, no continuous structure, massive Below 9 feet, less weathered, massive siltstone	130
						Total Depth 11 feet	

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

DATE DRILLED: 5-24-16
EQUIPMENT USED: 24" Backhoe
GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 3162.I
KIA CALABASAS

LOG OF BORING NO. TP-4

FIGURE A-15

PROBE (in)	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: **SILTY CLAY (CL)** dark brown, dry, many roots to 1/2-inch diameter	
						UPPER TOPANGA FORMATION (Ttus):**SILTSTONE** yellow brown, hard, poorly bedded	145
					5		
							140
					10	@ 9.5 feet, 6-inch thick orange weathering carbonate bed, overall slightly fractured, very tight @ 11 feet, B: N76E, 13NW	
							135
					15	**SANDSTONE** grey, slightly moist, easily friable, massive	
						Total Depth 16 feet	

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

DATE DRILLED:
5-24-16

EQUIPMENT USED:
24" Backhoe

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 3162.I
KIA CALABASAS

LOG OF BORING NO. TP-5

FIGURE A-16

PROBE (in)	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					0	Natural: COLLUVIUM (Qc): **SILTY CLAY (CL)** dark brown, slightly moist, firm, very porous, trace shale, roots and rootlets to 1/2-inch diameter	
					5	**SANDY SILT (ML)** yellow brown, slightly moist, very porous, with shale fragments	120
					10		115
						UPPER TOPANGA FORMATION (Ttus):**SANDSTONE** yellow brown, fine grained, massive, slightly moist	
						Total Depth 11.5 feet	

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

DATE DRILLED:
5-24-16

EQUIPMENT USED:
24" Backhoe

GROUNDWATER LEVEL (ft):
Not Encountered

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PROJECT NO.: 3162.I
KIA CALABASAS

LOG OF BORING NO. TP-6

FIGURE A-17

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 dried in accordance with ASTM D2216. 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.

GRAIN SIZE DISTRIBUTION

Selected soil samples were dried, weighed, soaked in water until individual soil particles were separated, and then washed on the No. 200 sieve. That portion of the material retained on the No. 200 sieve was oven-dried and weighed to determine the percentage of the material passing the No. 200 sieve. (ASTM D1140) The percentages passing the No. 200 sieve are tabulated below.

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	PERCENT PASSING No. 200 SIEVE
B-1	11	Sandy Clay (CL)	59
B-2	11	Silty Sand (SM)	32
B-101	0-5	Clay (CL) with Sand	78
B-101	10	Sandy Clay (CL)	51
B-102	5	Sandy Silt (ML)	50
B-102	15.5	Siltstone	50

ATTERBERG LIMITS

Liquid and plastic limits were determined for select samples in accordance with ASTM D4318. The results of the Atterberg Limits tests are presented in Figure B-1.

CONSOLIDATION

One-dimensional consolidation testing was performed on undisturbed samples in accordance with ASTM D 2435. After trimming the ends, the samples were placed in the consolidometer and loaded to 0.25 and 0.4 ksf. Thereafter, the samples were incrementally loaded to a maximum load of 25.6 and 32 ksf. The samples were inundated at 1 and 1.6 ksf. Sample deformation was measured to 0.0001 inch. Rebound behavior was investigated by unloading the samples back to 0.25 and 0.4 ksf. Results of the consolidation tests, in the form of percent consolidation versus log pressure, are presented in Figures B-2 to B-5.

DIRECT SHEAR

Direct shear tests were performed on undisturbed samples in accordance with ASTM D3080. 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. For samples B-4 @ 20-feet, and B-6 @ 13-feet multiple passes were made on the specimens at a strain rate of 0.025, and 0.04 inches per minute respectively. Shear stress and sample deformation were monitored throughout the tests. The results of the direct shear tests are presented in Figures B-6 to B-13.

COMPACTION TEST

A maximum dry density/optimum moisture test was performed in accordance with ASTM D 1557 on a representative bulk sample of the site soils. The test results are as follows:

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)
B-5	0-5	Sandy Clay (CH)	108	17.0
B-101	0 – 5	Clay (CL) with Sand	108	18

R-VALUE

Suitability of the near-surface soils for pavement support was evaluated by conducting an R-Value test. The test was performed in accordance with ASTM D 2844 by Geologic Associates under subcontract to GPI. The result of the test is as follows:

BORING NO.	DEPTH (ft.)	SOIL DESCRIPTION	R-VALUE BY EXPANSION
B-1	0-5	Sandy Clay (CL)	7

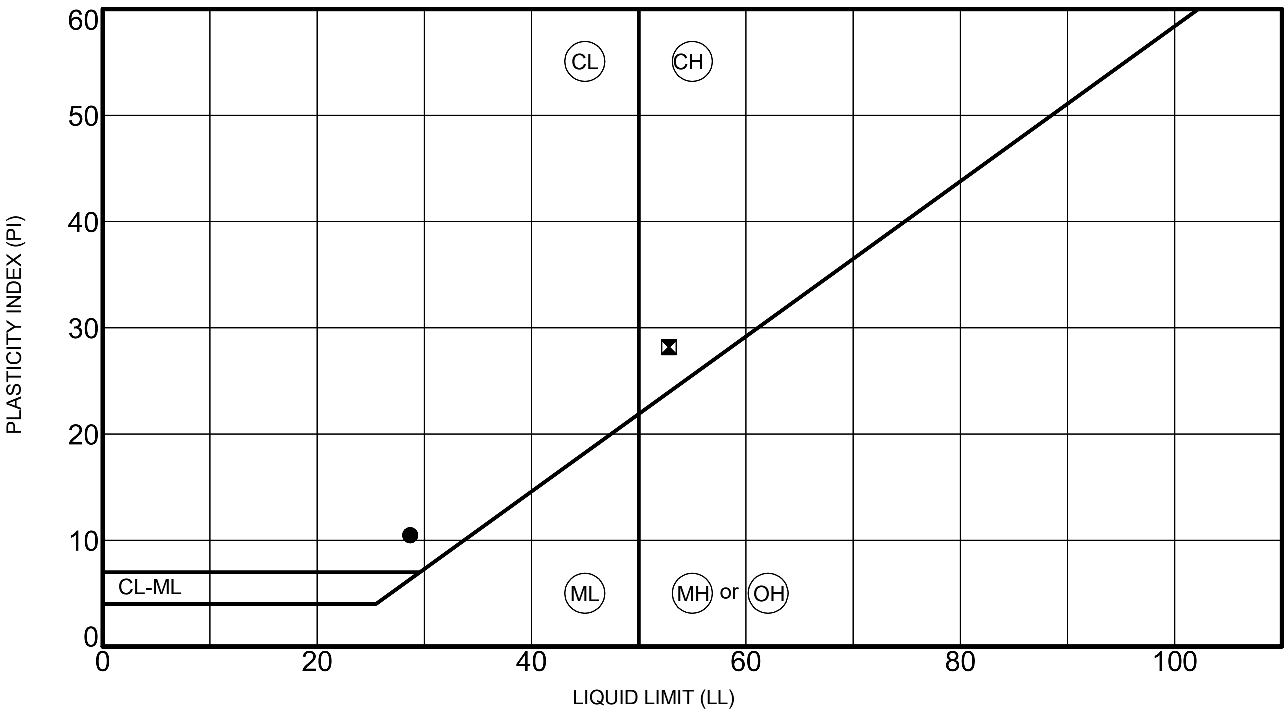
EXPANSION INDEX

An expansion index test was performed on a bulk sample. The test was performed in accordance with ASTM D4829, 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-1	0-5	Sandy Clay (CL)	7
B-101	0 – 5	Clay (CL) with Sand	65

CORROSIVITY

Soil corrosivity testing was performed by HDR a 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	0.0	29	18	10		CLAY (CL)
⧗	B-5	0.0	53	25	28		SANDY CLAY (CH)

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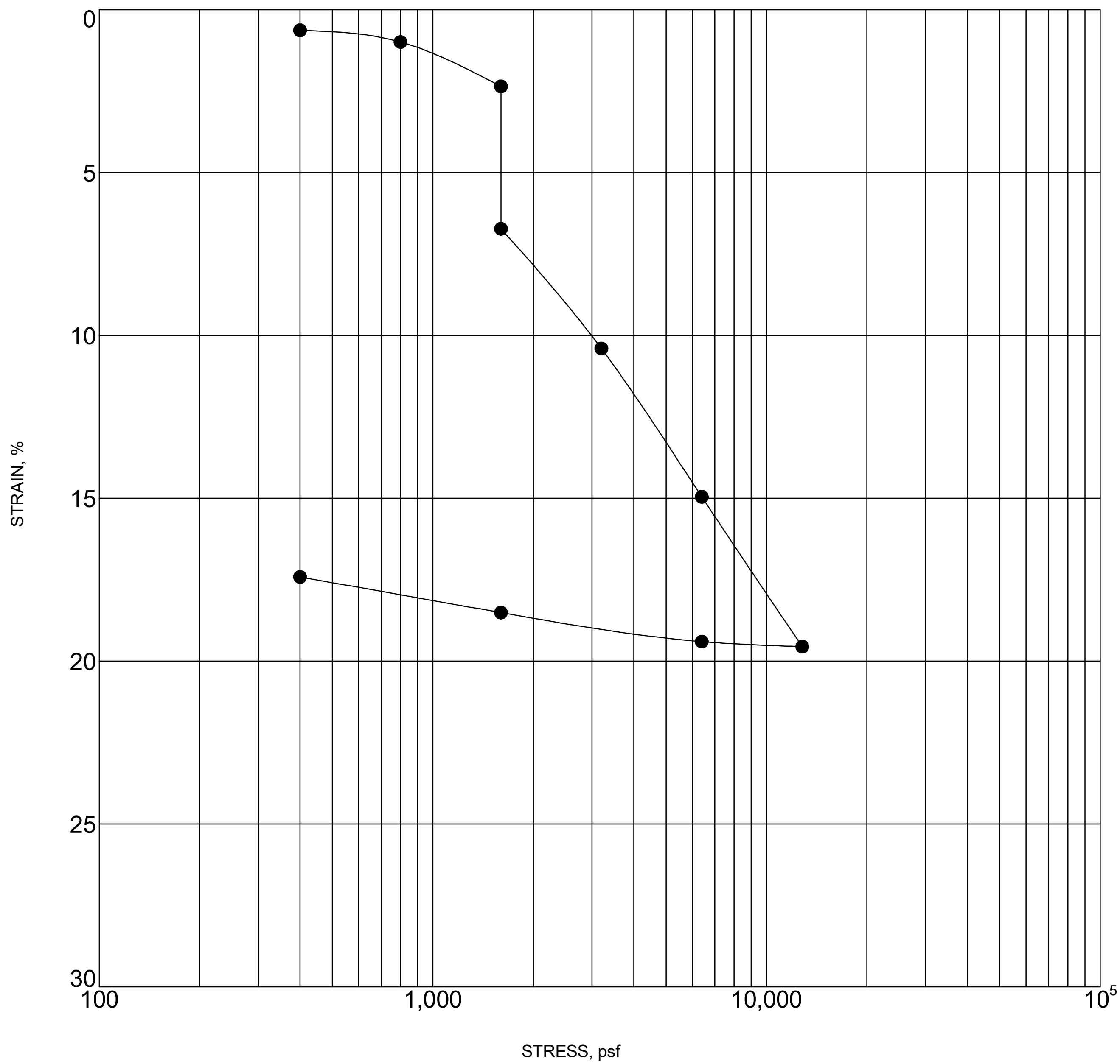
ATTERBERG LIMITS TEST RESULTS

FIGURE B-1



Sample inundated at 1600 psf

	Sample Location		Classification	DD,pcf	MC,%
●	B-5	5.0	FILL - SANDY CLAY (CH)	67	19.7

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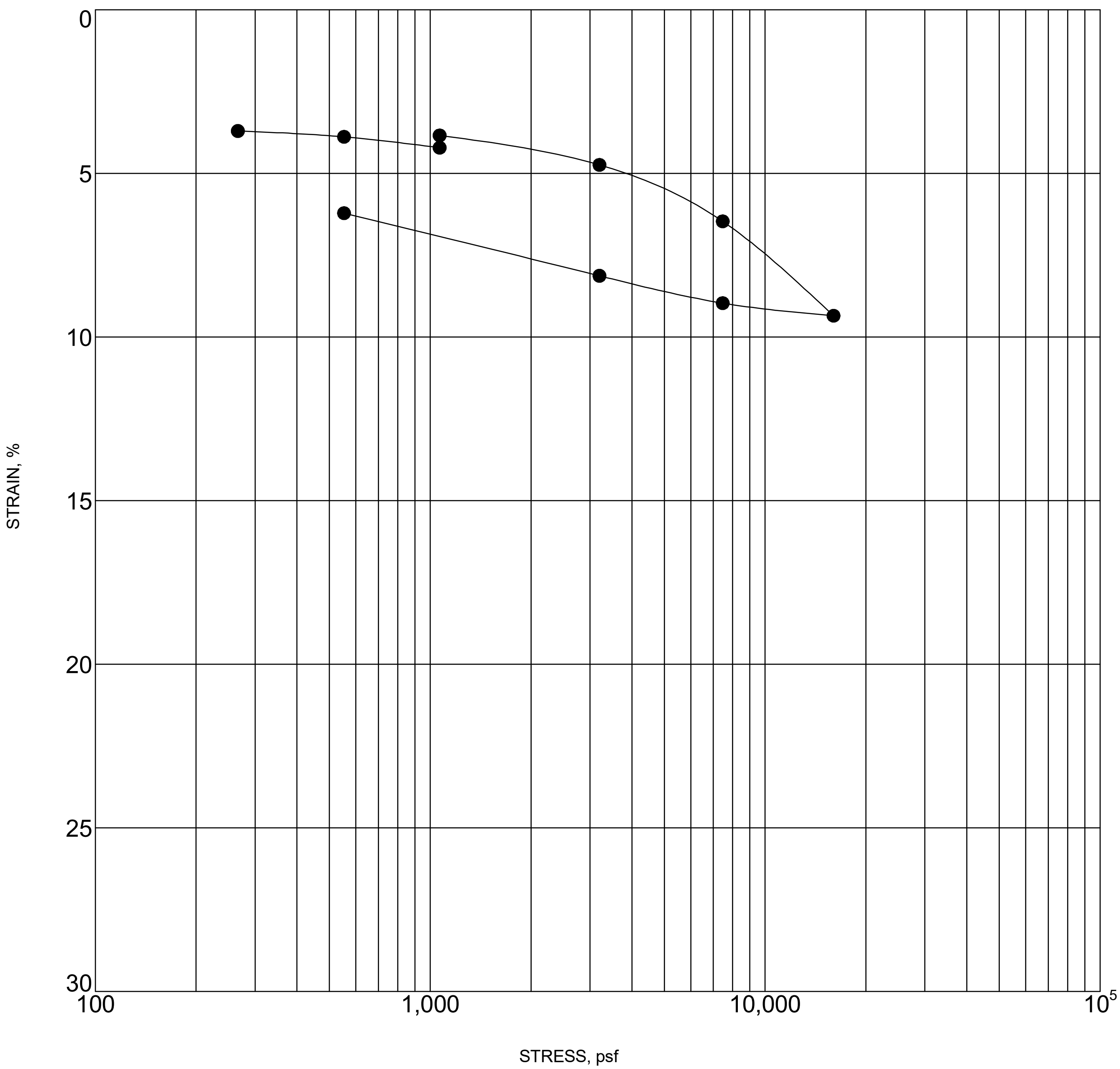
CONSOLIDATION TEST RESULTS

FIGURE B-2



Sample inundated at 1000 psf

	Sample Location		Classification	DD,pcf	MC,%
●	B-5	13.0	SANDY CLAY (CL)	84	27.7

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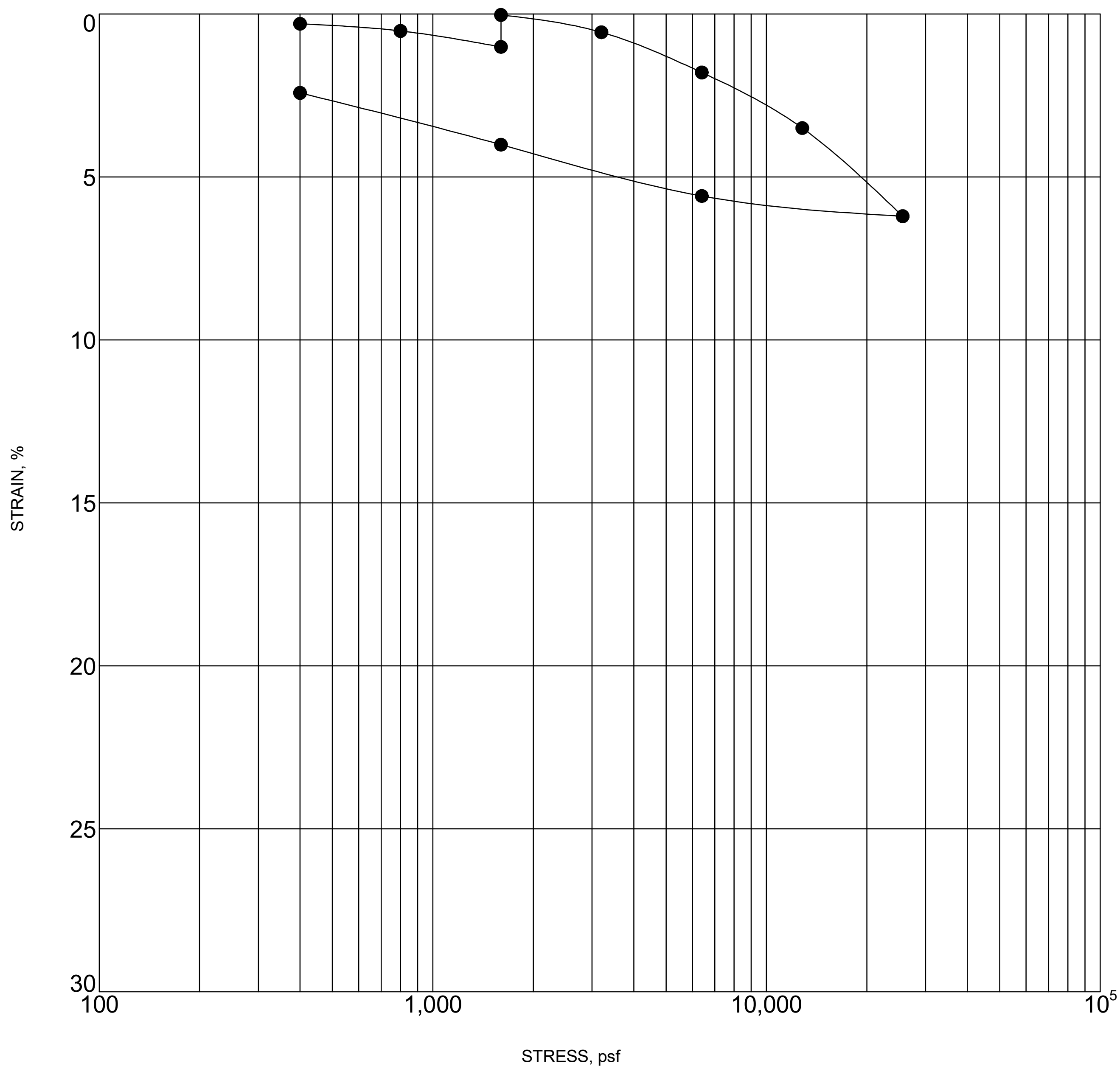
CONSOLIDATION TEST RESULTS

FIGURE B-3



Sample inundated at 1600 psf

	Sample Location		Classification	DD,pcf	MC,%
●	B-101	5.0	CLAY (CL)	93	18.5

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PROJECT NO.: 3162.I

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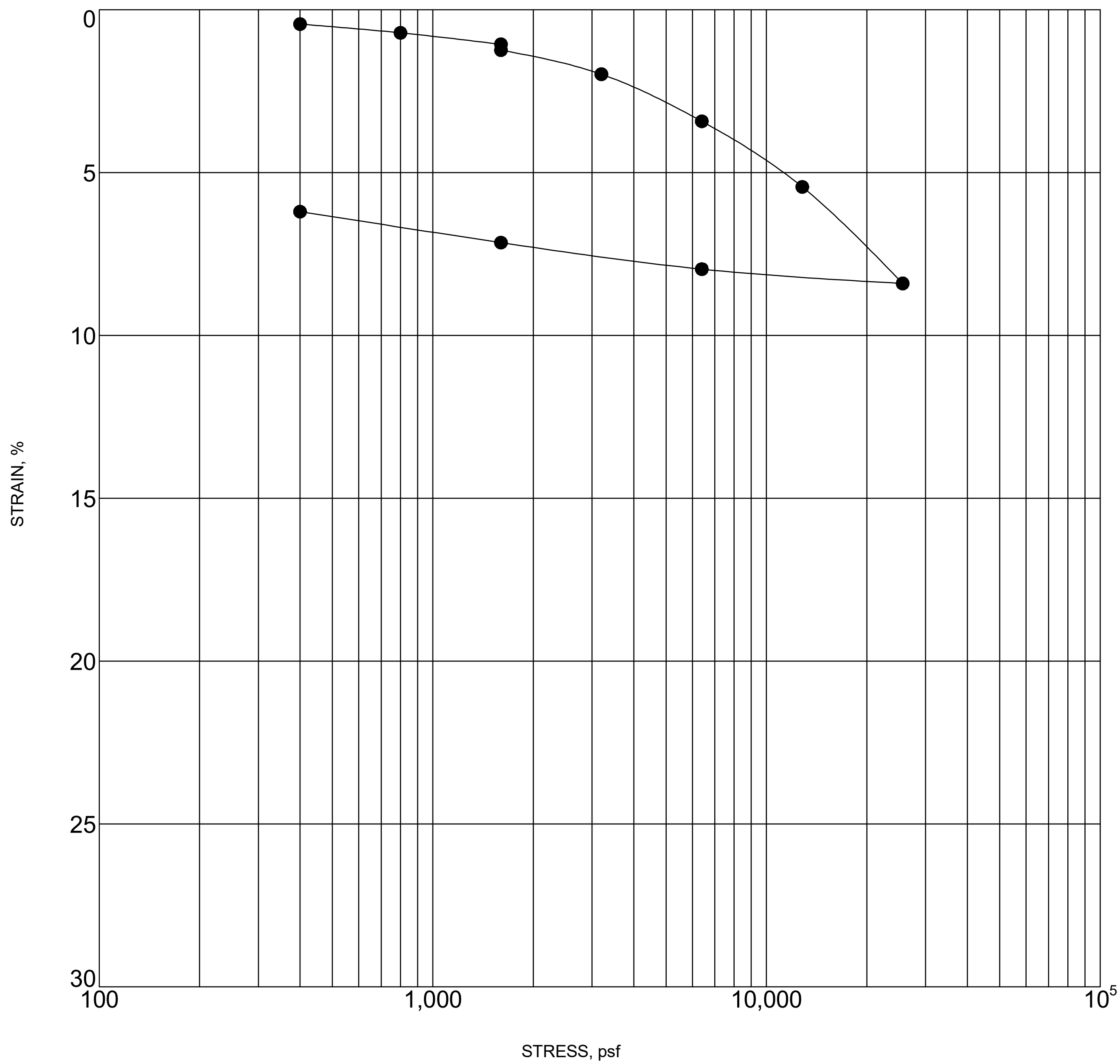
CONSOLIDATION TEST RESULTS

FIGURE B-4



Sample inundated at 1600 psf

	Sample Location		Classification	DD,pcf	MC,%
●	B-101	10.0	SANDY CLAY (CL)	98	13.6

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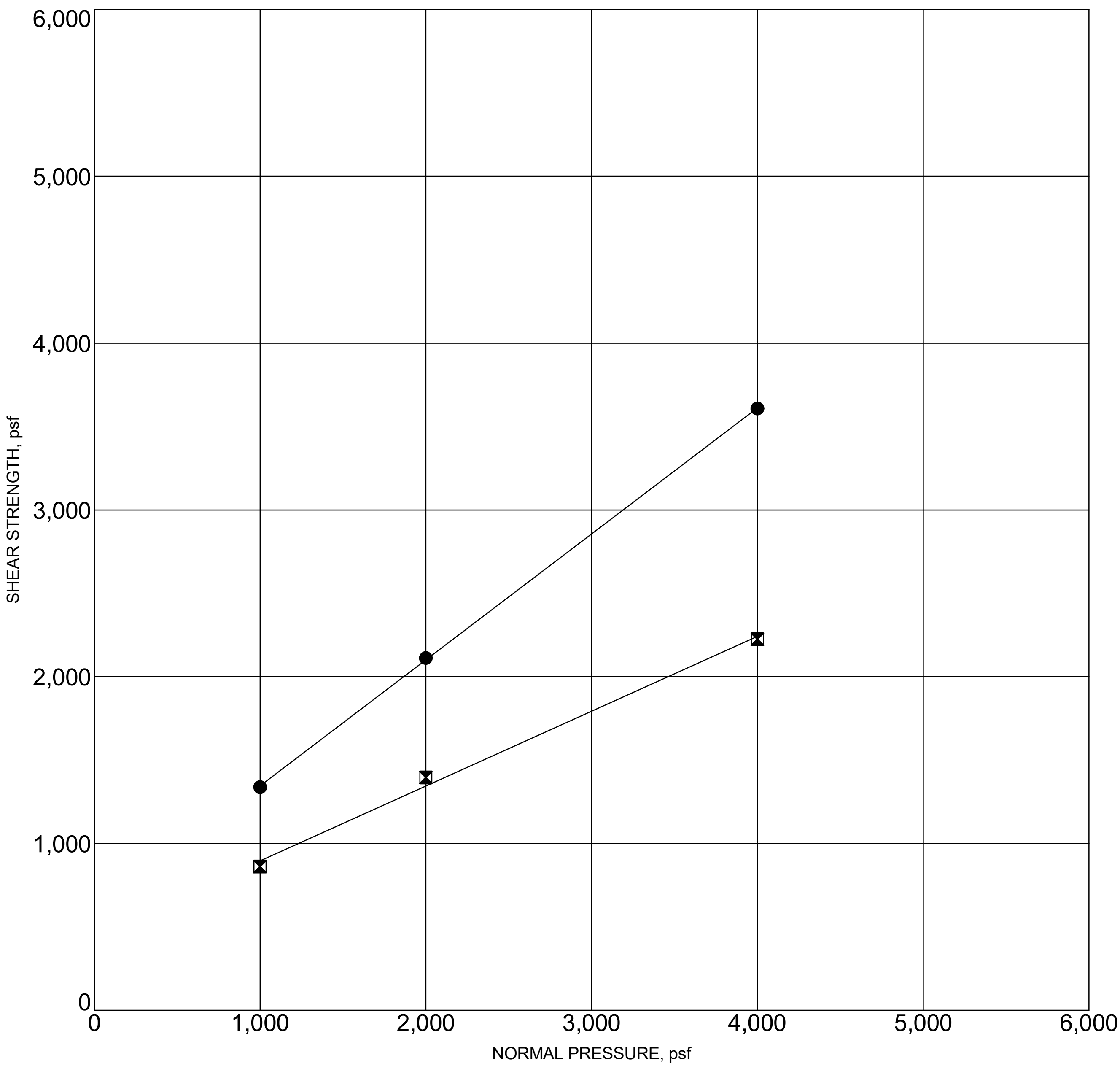
CONSOLIDATION TEST RESULTS

FIGURE B-5



● **PEAK STRENGTH**
Friction Angle= 37 degrees
Cohesion= 590 psf

ULTIMATE STRENGTH
Friction Angle= 24 degrees
Cohesion= 448 psf

Sample Location		Classification	DD,pcf	MC,%
B-4	13.0	SILTSTONE	109	19.1

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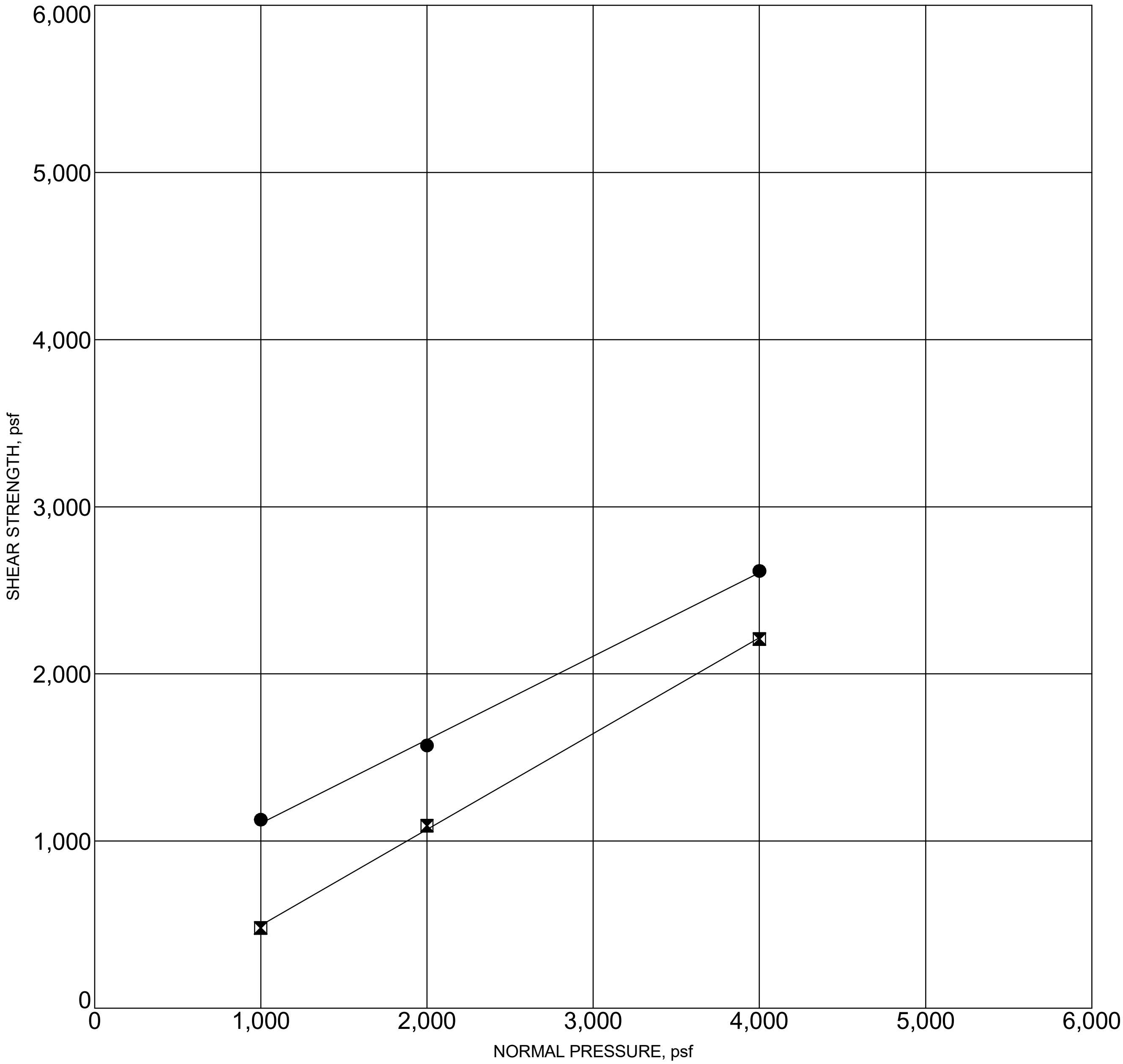
DIRECT SHEAR TEST RESULTS

FIGURE B-6



● **PEAK STRENGTH**
Friction Angle= 27 degrees
Cohesion= 606 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 29 degrees
Cohesion= 0 psf

Sample Location		Classification	DD,pcf	MC,%
B-4	20.0	SILTSTONE	109	18.7

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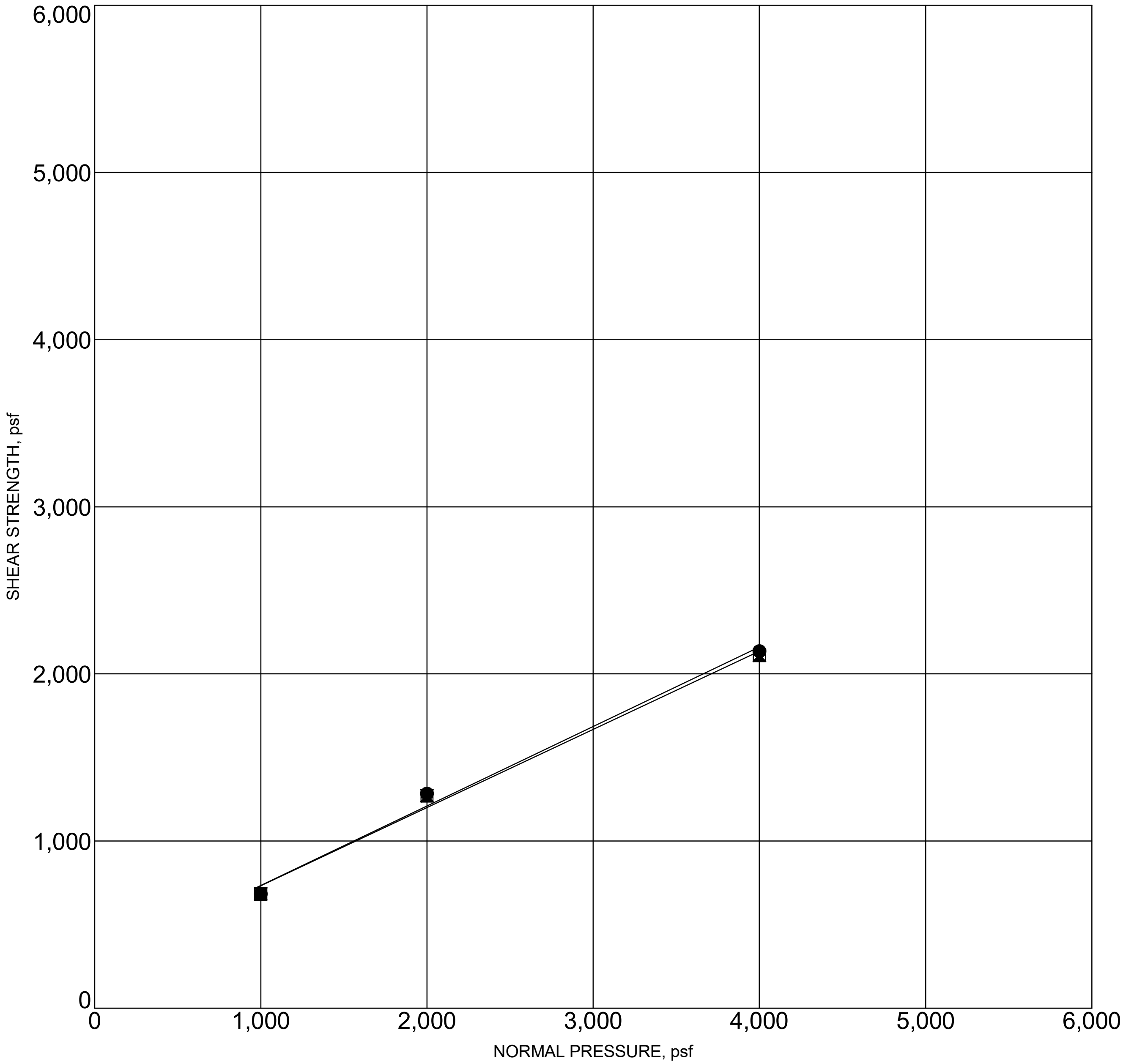
DIRECT SHEAR TEST RESULTS

FIGURE B-7



● **PEAK STRENGTH**
Friction Angle= 25 degrees
Cohesion= 258 psf

ULTIMATE STRENGTH
Friction Angle= 25 degrees
Cohesion= 264 psf

Sample Location		Classification	DD,pcf	MC,%
B-5	7.0	SANDY CLAY (CL)	84	19.5

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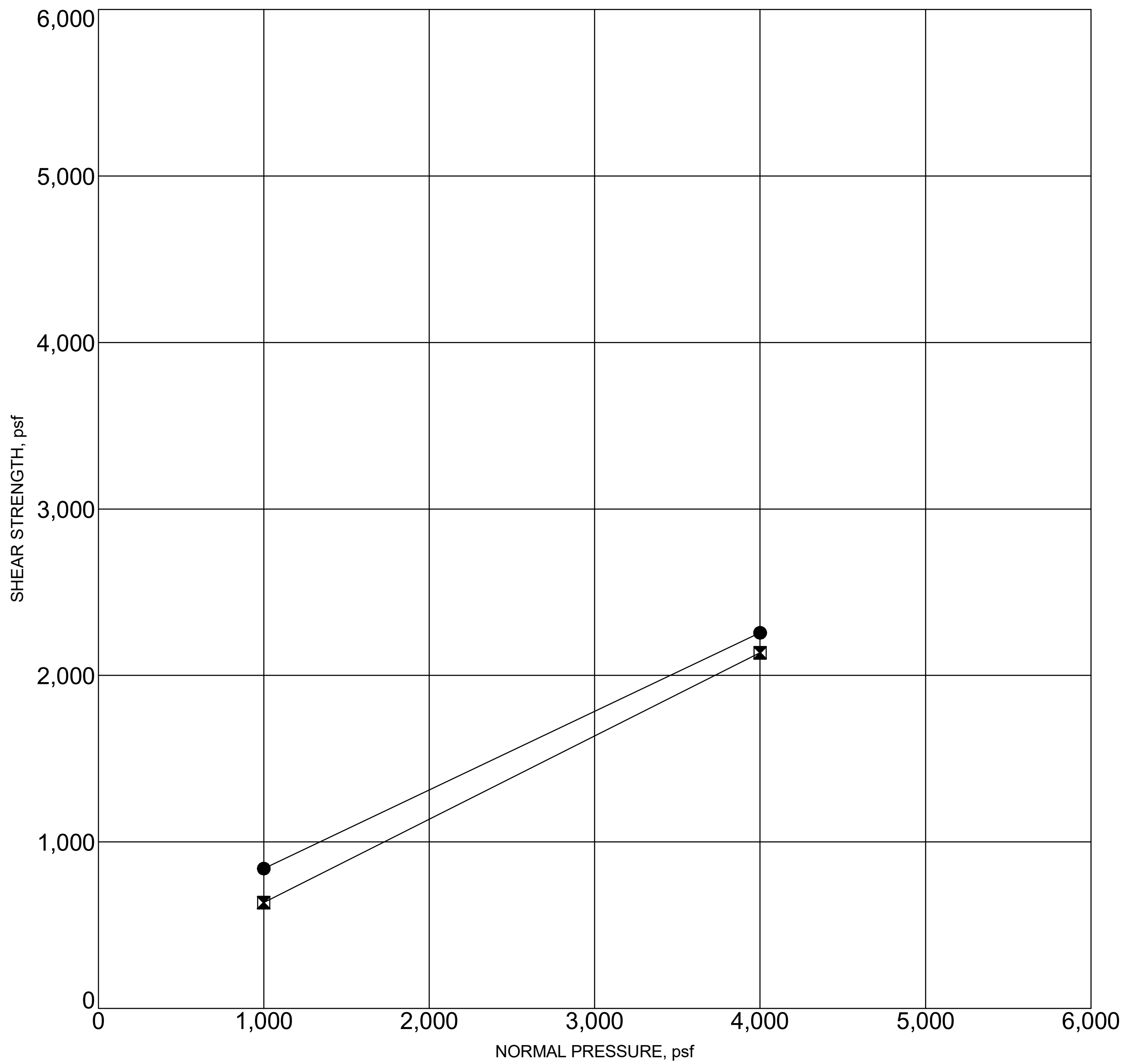
DIRECT SHEAR TEST RESULTS

FIGURE B-8



PEAK STRENGTH
Friction Angle= 25 degrees
Cohesion= 368 psf

ULTIMATE STRENGTH
Friction Angle= 27 degrees
Cohesion= 136 psf

Sample Location		Classification	DD,pcf	MC,%
B-6	13.0	SILTSTONE	104	10.3

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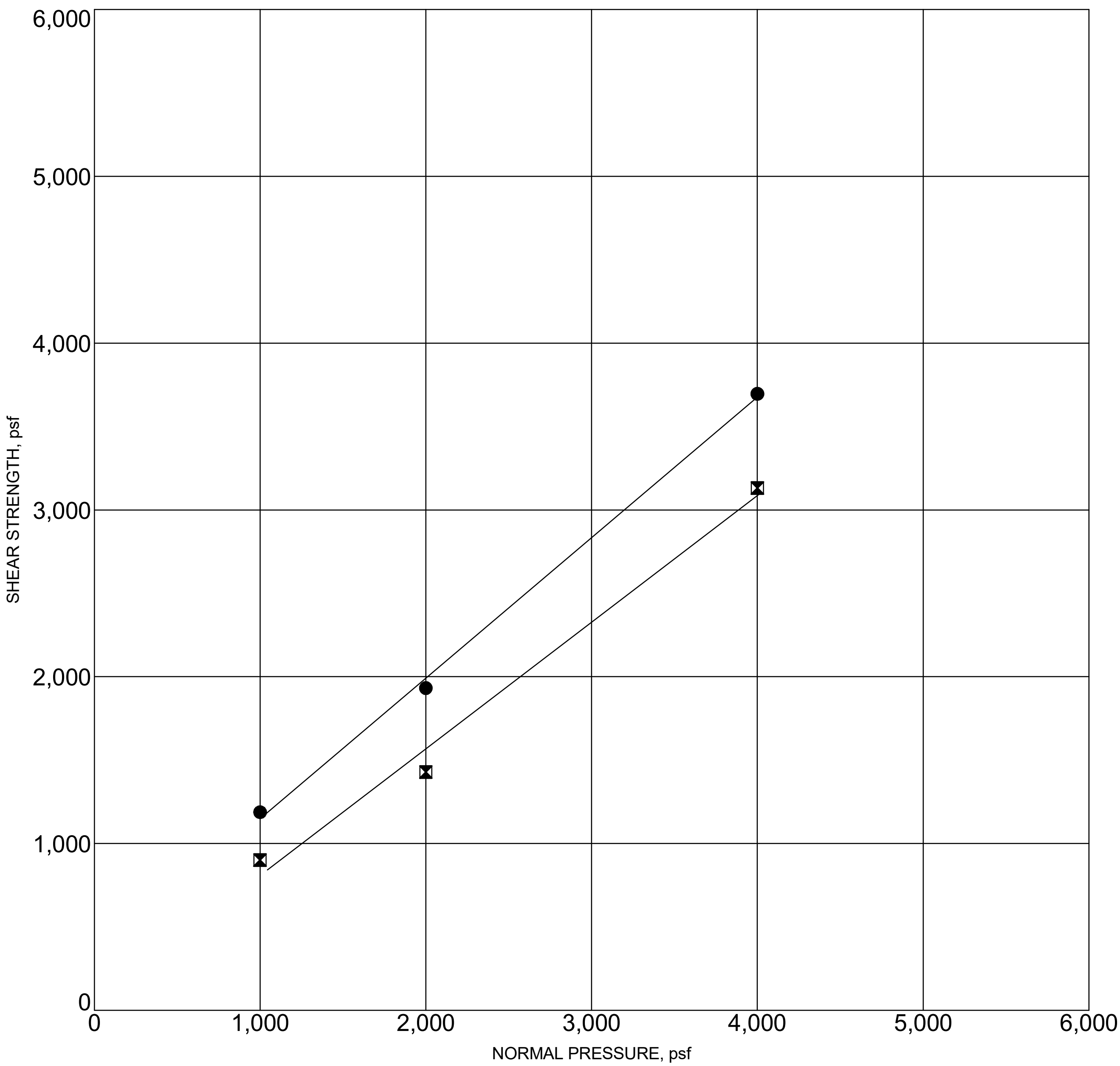
DIRECT SHEAR TEST RESULTS

FIGURE B-9



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

⧗ **ULTIMATE STRENGTH**
Friction Angle= 37 degrees
Cohesion= 48 psf

Sample Location		Classification	DD,pcf	MC,%
B-8	10.0	SILTSTONE	98	5.9

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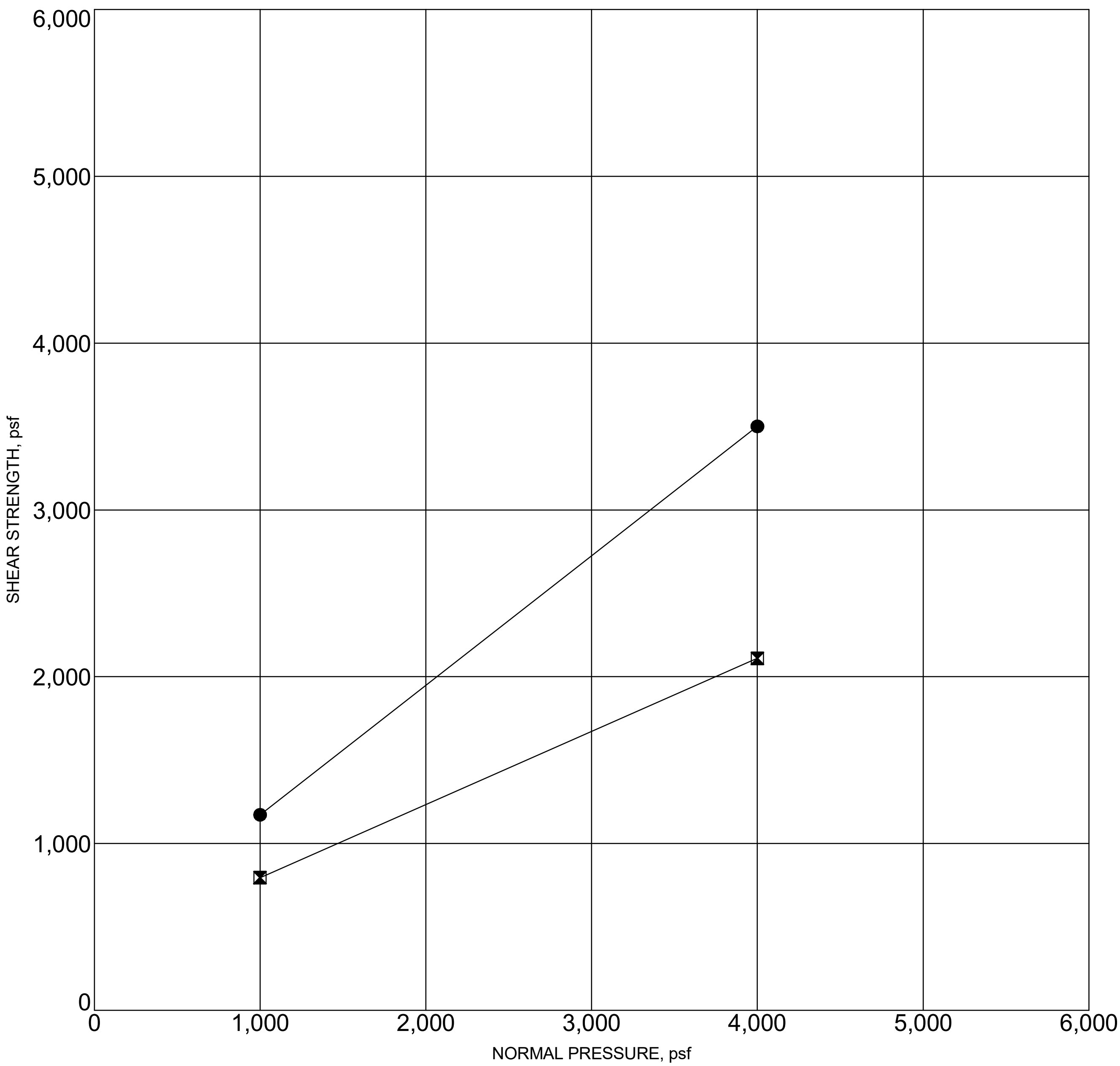
DIRECT SHEAR TEST RESULTS

FIGURE B-10



● **PEAK STRENGTH**
Friction Angle= 38 degrees
Cohesion= 396 psf

ULTIMATE STRENGTH
Friction Angle= 24 degrees
Cohesion= 357 psf

Sample Location		Classification	DD,pcf	MC,%
B-9	7.0	SILTSTONE	100	13.5

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PROJECT NO.: 3162.I

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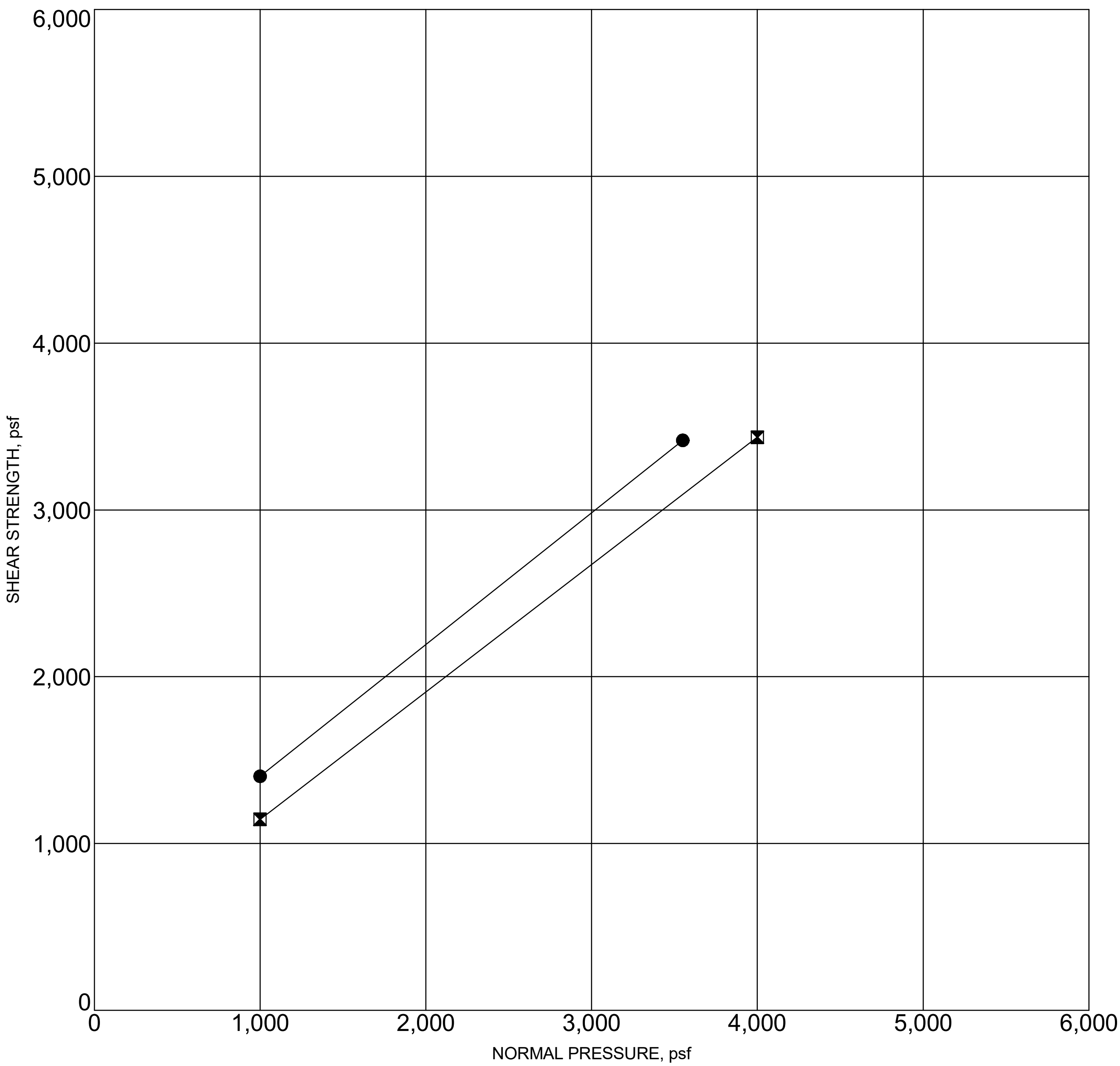
DIRECT SHEAR TEST RESULTS

FIGURE B-11



● **PEAK STRENGTH**
Friction Angle= 38 degrees
Cohesion= 613 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 37 degrees
Cohesion= 381 psf

Sample Location		Classification	DD,pcf	MC,%
B-9	13.0	SANDSTONE	99	4.2

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PROJECT NO.: 3162.I

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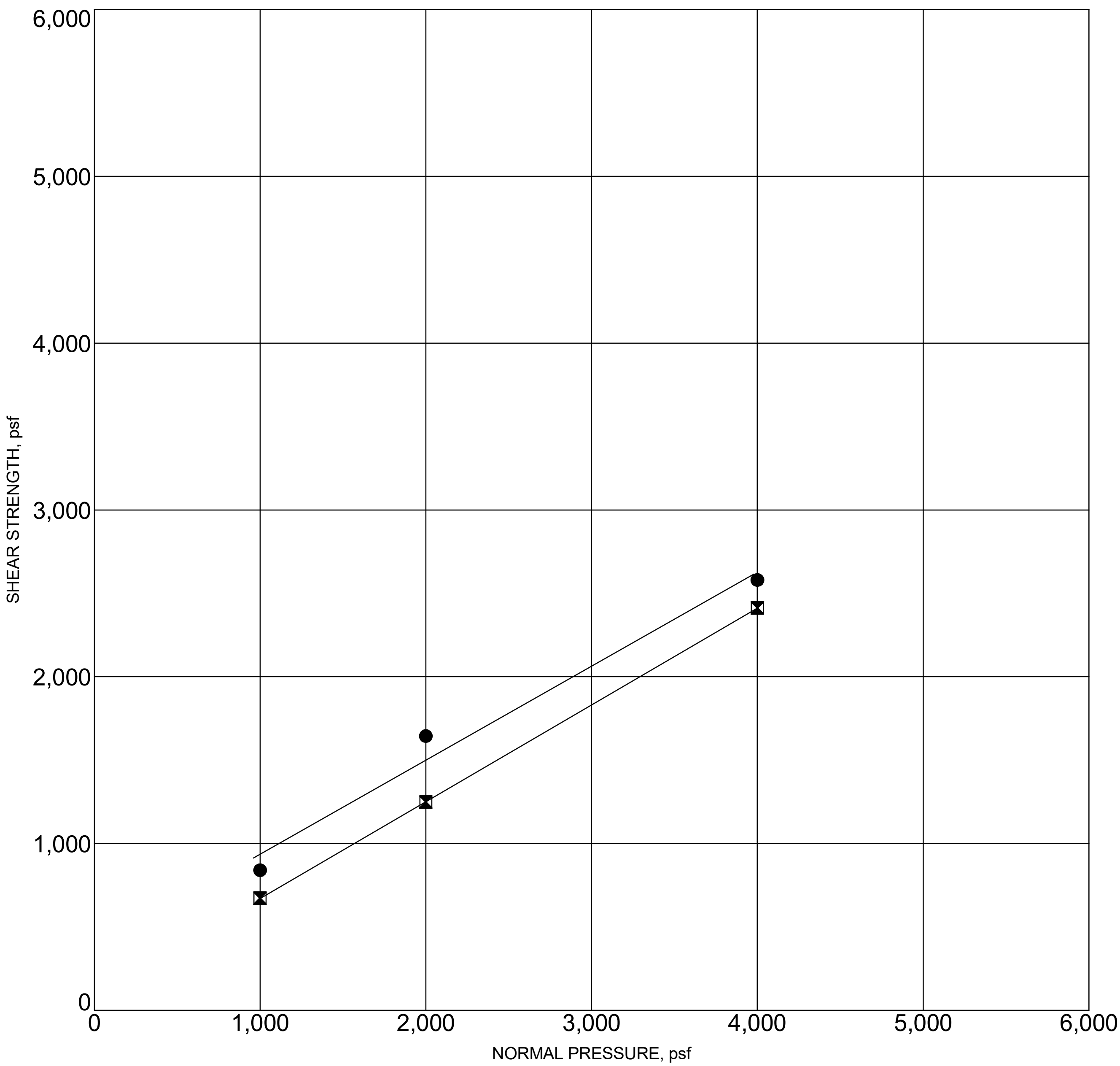
DIRECT SHEAR TEST RESULTS

FIGURE B-12



● **PEAK STRENGTH**
Friction Angle= 29 degrees
Cohesion= 372 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 30 degrees
Cohesion= 90 psf

Sample Location		Classification	DD,pcf	MC,%
B-101	7.0	CLAY (CL)	97	20.8

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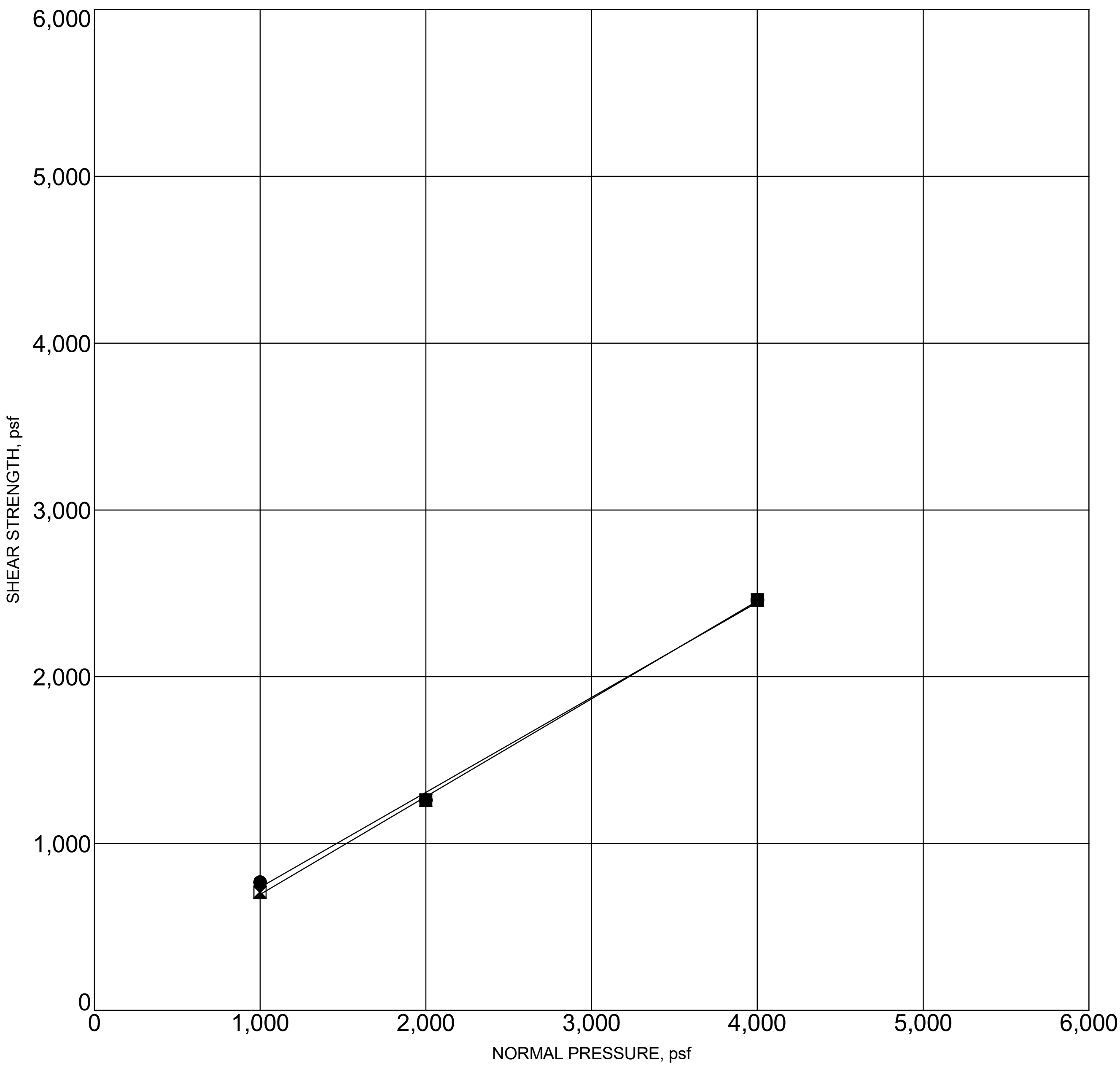
DIRECT SHEAR TEST RESULTS

FIGURE B-13



● **PEAK STRENGTH**
Friction Angle= 30 degrees
Cohesion= 168 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 30 degrees
Cohesion= 108 psf

Sample Location		Classification	DD,pcf	MC,%
B-102	5.0	SANDY SILT (ML)	102	9.3

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PROJECT NO.: 3162.I

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DIRECT SHEAR TEST RESULTS

FIGURE B-14

Table 1 - Laboratory Tests on Soil Samples

Geotechnical Professionals, Inc.
Calabasas
Your #2730.I, HDR Lab #16-0402LAB
31-May-16

Sample ID			B-2 @ 0-5'	B-8 @ 0-5'
Resistivity		**Units**		
as-received		ohm-cm	12,000	52,000
saturated		ohm-cm	440	1,040
pH			6.7	7.0
Electrical Conductivity		mS/cm	1.51	0.29
Chemical Analyses				
Cations				
calcium	Ca^{2+}	mg/kg	780	146
magnesium	Mg^{2+}	mg/kg	107	7.3
sodium	Na^{1+}	mg/kg	323	27
potassium	K^{1+}	mg/kg	150	57
Anions				
carbonate	CO_3^{2-}	mg/kg	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	174	250
fluoride	F^{1-}	mg/kg	2.0	1.4
chloride	Cl^{1-}	mg/kg	438	14
sulfate	SO_4^{2-}	mg/kg	1,750	65
phosphate	PO_4^{3-}	mg/kg	3.5	ND
Other Tests				
ammonium	NH_4^{1+}	mg/kg	ND	ND
nitrate	NO_3^{1-}	mg/kg	2,240	616
sulfide	S^{2-}	qual	na	na
Redox		mV	na	na

Resistivity per ASTM G187, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.
Electrical conductivity in millisiemens/cm and chemical analyses 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

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Table 1 - Laboratory Tests on Soil Samples

Geotechnical Professionals, Inc.
KIA Calabasas
Your #3162.I, HDR Lab #22-1094LAB
21-Nov-22

Sample ID			B-101 @ 0-5'
Resistivity		**Units**	
as-received		ohm-cm	4,400
saturated		ohm-cm	1,000
pH			7.9
Electrical Conductivity		mS/cm	0.16
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	69
magnesium	Mg^{2+}	mg/kg	ND
sodium	Na^{1+}	mg/kg	65
potassium	K^{1+}	mg/kg	13
ammonium	NH_4^{1+}	mg/kg	ND
Anions			
carbonate	CO_3^{2-}	mg/kg	ND
bicarbonate	HCO_3^{1-}	mg/kg	305
fluoride	F^{1-}	mg/kg	3.1
chloride	Cl^{1-}	mg/kg	11
sulfate	SO_4^{2-}	mg/kg	76
nitrate	NO_3^{1-}	mg/kg	1.3
phosphate	PO_4^{3-}	mg/kg	ND
Other Tests			
sulfide	S^{2-}	qual	na
Redox		mV	na

Resistivity per ASTM G187, pH per ASTM G51, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.

Electrical conductivity in millisiemens/cm and chemical analyses 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

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