



**GEOTECHNICAL INVESTIGATION
WOODCREST RETAIL CENTER
NWC VAN BUREN BOULEVARD AND WASHINGTON STREET
WOODCREST, RIVERSIDE COUNTY, CALIFORNIA**

Prepared for:
VBSB, LLC
c/o LJC Enterprises
P.O. Box 2739
Mammoth Lakes, California 93456

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

April 24, 2026

VBSB, LLC
c/o LJC Enterprises
P.O. Box 2739
Mammoth Lakes, CA 93456

Attention: Mr. Larry Canale

Subject: Report of Geotechnical Investigation
Woodcrest Retail Center
NWC Van Buren Boulevard and Washington Street
Woodcrest, Riverside County, California

Dear Mr. Canale:

Transmitted herewith is a copy of our geotechnical investigation report for the proposed project at the subject site. We understand that you will distribute this report to the appropriate Design Team members and regulatory agencies.

We appreciate the opportunity of providing our services to the Team and look forward to seeing the project through its successful completion. Please do not hesitate to contact us if you have questions on the contents of our report or need further geotechnical assistance.

Very truly yours,
Geotechnical Professionals Inc.



Patrick I.F. McGervey, G.E.
Senior Engineer



Donald A. Cords, G.E.
Principal



Thomas Hill, CEG
Consulting Geologist

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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 retail center at the subject site in Woodcrest, California. The site location is shown on the Site Location Map, Figure 1.

1.2 PROJECT DESCRIPTION

We understand a new retail center is planned for the site covering 9.0 acres. Based on the Conceptual Site Plan by GK Pierce Architects, dated January 21, 2026, the retail center will include a Market (Stater Bros.), a shops building (Shops A), two restaurant buildings with drive-throughs (Pad B and Pad C), and a gas station with associated concession store (Pad A). The existing Farmer Boys and Bank buildings and surrounding drives within the southwest corner of the property are not a part of this report. The buildings will be single-story structures. The Market will cover a footprint of approximately 43,400 square feet (sf) and the other buildings range from 2,500 to 4,700 sf. New paved and landscaped areas are also planned. The proposed site configuration is shown on the Site Plan, Figure 2.

Proposed finish floor elevations and structural loads were not finalized at the time this report was prepared. We have assumed that proposed grades for the buildings will be similar to existing grades with cuts and fills of up to 3 feet. Based on similar past projects, we assume that maximum column and wall loads will be on the order of 100 to 150 kips and 3 to 5 kips per lineal foot, respectively (dead plus live loads).

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

1.3 PURPOSE OF INVESTIGATION

The primary purpose of this investigation and report is to provide an evaluation of the existing geotechnical 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 and pavements.

2.0 SCOPE OF WORK

Our scope of work included review of existing data, subsurface exploration, laboratory testing, engineering analysis and the preparation of this report.

Our subsurface exploration for this site consisted of 10 hollow stem auger borings and 12 exploratory test pits. The borings and test pits were performed as part of our feasibility investigations for the site in July 2020 and October 2020 (respectively) and were utilized in developing the recommendations provided in this report. The exploratory borings were advanced to depths of 6 to 21 feet below existing grades and the test pits to depths of 2 to 9½-feet below existing grades. A description of field procedures and logs of the explorations are presented in the attached Appendices A, and B. The approximate locations of the subsurface explorations are shown on Site Plans, Figures 2 and 3.

Laboratory soil tests were performed on selected representative samples as an aid in soil classification and to evaluate the engineering properties of the soils. The geotechnical laboratory testing program included determinations of moisture content and dry density, grain size analyses, shear strength, expansion potential, corrosivity, R-value and maximum density. Laboratory testing procedures and results are summarized in Appendix C.

Corrosivity testing was performed by HDR under subcontract to GPI. R-value testing was performed by Geologic Associates under subcontract to GPI. Their test results are presented Appendix C.

GPI subcontracted Terra Geosciences to perform a seismic shear-wave study. One seismic shear-wave line was conducted to measure shear wave velocities to a depth of at least 100 feet below the existing ground surface. The results of the seismic survey are presented in Appendix D.

GPI conducted a site reconnaissance and conducted a geologic review of reports in our files, and readily available published geologic information and maps, to assess the potential geologic hazards that could impact the site, including faulting and liquefaction. The results of our review are summarized in this report.

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

3.0 SITE CONDITIONS

3.1 SURFACE CONDITIONS

The site is bounded on the north by a residential neighborhood with several single-family homes, to the east by Washington Street, to the south by Van Buren Boulevard, and to the west by single-family homes and an undeveloped lot.

The site is mostly undeveloped except directly adjacent to the corner of Van Buren Boulevard and Washington Street where there is an existing single-story retail center which includes a gas station. The site is relatively flat with sparse vegetation and isolated areas of mature trees.

Based on historical aerial photographs (historicaerials.com), the site was predominately undeveloped in 1948 (oldest available photograph) with the exception of single family homes and landscaping along Washington Street. The existing single-story retail center was constructed between 1967 and 1978, replacing one of the single family home lots. The single-family home formerly located near the northeast portion of the site along Washington street appears to have been demolished between 2016 and 2018.

In general, the site is relatively flat and slopes gently downward from east to west, with a change in ground surface elevation from about Elevation +1,500 feet to +1,490 feet across the site.

3.2 SUBSURFACE SOIL CONDITIONS

Our field investigation disclosed a subsurface profile consisting of a thin layer of fill soils overlying natural soils. Detailed descriptions of the conditions encountered are shown on the Log of Borings in Appendix A, and the Log of Test Pits in Appendix B.

With the exception of TP-12, the soils consisted of a thin ($\frac{1}{2}$ to 1-foot thick) layer of sandy clay fill soils over natural soils. We encountered sandy clay fill soils in TP-12 to a depth of approximately 7 feet.

The underlying natural soils consist of a layer of residual soil deposits underlain by slightly to moderately weathered bedrock. The residual soils consist predominately of silty sands, sandy clays and clayey sands and were highly variable in depths, extending from approximately 1 to 11 feet below the existing grade with the average being approximately 5 feet. The residual soils were relatively easy to excavate using the backhoe, and consist of decomposed granite formed by the weathering over time of the underlying bedrock.

The underlying weathered bedrock is moderately weathered near the ground surface with less weathering with depth. In general, at depths of $1\frac{1}{2}$ to 11 feet below existing grades, the bedrock becomes less weathered, more intact, and very dense or hard. The weathered bedrock was difficult to excavate using the backhoe with the 24-inch bucket until refusal, and our hollow stem auger borings refused on the bedrock at depths between 6 and 16 feet below existing grades. The geologic map of Riverside shown on Figure 4 characterizes the bedrock at the site as tonalite. We have provided two geologic cross sections bisecting the site detailing the subsurface conditions in Figures 5.1 and 5.2.

We did not encounter significant amounts of clay soils in our borings at the site. The majority of the near surface soils are sandy and based on our laboratory testing exhibit very low expansion potential ($EI = 10$).

3.3 GROUNDWATER CONDITIONS

The historical high groundwater has not been determined in the Woodcrest area by the State of California. Groundwater was not measured at depths of 21 feet below existing grade in the borings however, we encountered a perched groundwater condition in several borings near the bottom of the residual soils or near the depth of refusal to drilling at intact bedrock. Perched groundwater can be anticipated in areas of the site overlying the intact bedrock. Other consultants (Stantec, 2009) have identified a perched groundwater condition at a depth of approximately 14 feet below existing grade at the southeast corner of the site.

4.0 GEOLOGIC - SEISMIC HAZARDS

4.1 OVERVIEW

GPI conducted a site reconnaissance and a geologic review of reports in our files and readily available published geologic information and maps, to assess potential geologic hazards that could impact the site, including faulting and liquefaction. The results of our review are summarized in this section of the report.

4.2 REGIONAL AND LOCAL GEOLOGIC SETTING

Regional Geology

The subject site is located within an uplifted, low relief plain known as Perris Plain. This area is generally bounded by the Box Springs Mountains to the northwest, the San Timoteo Badlands (mountain range) to the northeast, the Bernasconi Hills and Lake Perris (Perris Reservoir) to the east, and Sycamore Canyon to the west. In general, the Perris Plain in the area of the subject site drains to the southwest towards Lake Elsinore.

The subject site is at the eastern margin of a Peninsular Ranges Geomorphic Province structural block known as the Perris Block (Dyett et al., 2021). The Perris structural block is separated from the Santa Ana block to the west by the Elsinore Fault Zone and separated from the San Jacinto Block to the east by the San Jacinto Fault Zone.

In general, the Peninsular Ranges are characterized by northwesterly trending active faults and mountain ranges related to the San Andreas and other major fault systems in the province, such as the Chino, Elsinore, and San Jacinto Faults in relatively near proximity to the site. The province extends from the Santa Monica-Hollywood-Raymond Fault System, within the Los Angeles Basin, southeast to Baja California.

Local Geologic Conditions

Based on our current explorations and regional geologic maps by USGS, the site is underlain by Val Verde tonalite consisting of relatively homogenous, massive to well-foliated, medium to coarse grained biotite-hornblende tonalite. The geologic conditions in the site area are shown on the Regional Geologic Map, Figure 4.

4.3 FAULTING

4.3.1 Nearby Seismogenic Sources

We reviewed the California Geologic Survey (CGS) Fault Activity Map of California and results from the USGS Unified Hazard Tool to identify known nearby active faults. The names and distances of the faults lying within 15 miles of the project site are provided in the following table:

Table 1 – Significant Nearby Faults

Fault Name	Approximate Distance* (mi)
Elsinore	10.7
San Jacinto	11.9
San Andreas	20.1

* Defined as the closest distance to projection of rupture area along fault trace.

4.3.2 Active Faulting Evaluation

The site does not lie within an Alquist-Priolo Earthquake Fault Zone or have a fault located onsite or trending towards the site according to the California Geologic Survey Seismic Hazard Map.

Based on our investigation onsite, the available fault maps of the region, arial photographs, and our current explorations, there is no evidence of surficial fault ruptures onsite, and the potential for fault rupture at the site is low.

4.4 SEISMIC EXPOSURE

The subject site is located in a seismically active area of southern California. The type and magnitude of seismic hazards that may affect the site are dependent on both the distance to causative faults and the intensity and duration of the seismic event. The subject site will likely experience strong ground shaking caused by earthquakes on active, regional faults in the future. The proposed project should be designed and constructed in accordance with the seismic design parameters provided in the applicable building code and presented in the “Seismic Design” section of this report.

4.5 LIQUEFACTION AND LATERAL SPREADING

The site is not located within a zone identified as having a potential for liquefaction by the State, as the quadrangle has not yet been assessed. The site is not located in a zone identified as having a potential for liquefaction by the County of Riverside (Riverside, 2018). Based on the presence of near surface bedrock and significant depth to groundwater, liquefaction will not impact the site.

4.6 SEISMIC SUBSIDENCE

Seismic ground subsidence or dry sand settlement (not related to liquefaction induced settlements) occurs when strong earthquake shaking results in densification of loose to medium dense sandy soils above groundwater. Based on the relatively high density of the natural residual soils encountered above the bedrock and the relatively shallow depths to bedrock, the potential for dry sand seismic settlement to adversely affect the site is considered to be negligible.

4.7 LANDSLIDING

Secondary effects of seismic activity normally considered as possible hazards to a particular site which might occur as a consequence of severe ground shaking include

landsliding and ground lurching. The probability of occurrence depends on the severity of the earthquake, distance from faults, topography, subsoils, and groundwater conditions, in addition to other factors. Based on a review of available published literature, landsliding and ground lurching are considered unlikely at the site.

4.8 EXPANSIVE AND COLLAPSE POTENTIAL

Expansive soils generally consist of clays that can shrink and swell with changes in moisture content. Movement of soils in response to shrinkage and swelling has the potential to impact near-surface improvements such as lightly loaded foundations, floor slabs, and flatwork.

Based on the soils encountered in our explorations and laboratory testing, near surface soils are anticipated to have a low expansion potential when subject to changes in moisture content.

Collapsible soils generally consist of relatively dry, low-density materials that become weaker and more compressible with the addition of water or excessive loading. Based on our the soils encountered onsite, the potential for collapse of soils at this site to impact the project is considered low.

4.9 FLOODING AND EROSION

Various types of seismically induced flooding, which may be considered as potential hazards to a particular site, include flooding due to a tsunami (seismic sea wave), a seiche, or failure of a major water retention structure upstream of the project. The subject site is located approximately 33 miles inland from the Pacific Ocean at an elevation of approximately 1,490 feet above mean sea level. Based on the above, the probability of flooding due to a tsunami is considered to be nonexistent.

The subject site does lie in proximity to two notable bodies of water: Lake Perris (Perris Reservoir) to the east and Lake Matthews to the west. Lake Perris is an artificial lake having an approximate area of 3 square miles and is located approximately 3.5 miles east of the site at closest approach. Lake Perris lies at an elevation of roughly +1560 feet, which is comparable to the ground surface elevation of the proposed development. However, the subject site and Lake Perris are separated by a valley with an elevation of approximately 1450 at its lowest. As such, the probability of flooding due to seiche from Lake Perris is considered to be negligible.

Lake Matthews is located roughly 9.5 miles west of the site and lies at an elevation of approximately +1400 feet (about 100 feet below the ground surface elevation of the site).

Based on the significant elevation difference between the lake and the subject site, the probability of flooding due to seiche from Lake Matthews is considered to be negligible.

According to a flood map (Map 06065C1410G, dated August 28, 2008) prepared by the Federal Emergency Management Agency (FEMA), the project site is located in Zone D, which is defined as being an area with possible but undetermined flood hazards as not flood hazard analysis has been conducted.

4.10 SEDIMENTATION AND EROSION

The majority of the ground surface at the site is relatively level and is, or will be, covered with asphalt or concrete pavements. As such, erosion is not considered a hazard at the site.

During construction, provisions should be in place to address potential temporary erosion and sedimentation conditions. These provisions will be provided in the design-level geotechnical report and incorporated into the project civil and landscaping plans.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 OVERVIEW

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 geotechnical constraints discussed below are mitigated. The most significant geotechnical issues that will affect the design and construction of the proposed building are as follows:

- Undocumented fills and disturbed/tilled soils were generally encountered to depths of up to 1 foot below existing grade at the site. The fill soils are not considered to be suitable for direct support of foundations or floor slabs without remedial earthwork. For the proposed improvements, we recommend removal and recompaction of the fill and a portion of the upper low-density natural soils to provide uniform support for the planned foundations, and floor slab.
- Significant ripping and slow earthwork operations should be anticipated for excavations deeper than about 3 to 6 feet from the existing ground surface in portions of the site. While not encountered in our borings, buried boulders or outcrops of hard granitic rock near the existing surface may be encountered and may require chipping, blasting, and/or special handling.
- Rock greater than 12 inches in greatest dimension generated from removal of rock outcrops, buried boulders, or ripping of the weathered bedrock will need to be broken to achieve smaller sizes, removed from the site, or incorporated into landscape areas.
- Trench and tank excavations greater than 6 to 8 feet below existing grade may be difficult and will likely require large excavators with rock breakers in portions of the excavations.
- Static groundwater is anticipated to be significantly deeper than any excavations at the site. However, localized perched groundwater may be encountered in trench and tank excavations underlying the residual soils or weathered bedrock. The Contractor should be prepared to control perched groundwater in deeper excavations.
- Storm water infiltration is not to be feasible at the site. The presence of the perched groundwater indicates very poor conditions with respect to subsurface water infiltration.

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

5.2 SEISMIC DESIGN

5.2.1 General

GPI retained Terra Geosciences to perform a seismic shear-wave survey to a depth of at least 100 feet below the existing ground surface (see Appendix D). Terra performed one seismic line to measure shear-wave velocity at the site, located on the southeast side of the site. The location of the seismic shear-wave survey line is shown on the attached Site Plans, Figures 2 and 3. Analysis of the data obtained during the seismic shear-wave survey indicate that the “weighted average” shear-wave velocity within the upper 100 feet of the subject site, V_{s30} , is 2,056 feet per second. The results of the shear-wave velocity testing is presented in Appendix D. In accordance with ASCE 7-22, this shear-wave velocity corresponds to a **Site Class C**.

We understand the seismic design of the proposed development will be in accordance with the California Building Code (CBC), 2025 edition and ASCE 7-22. For the referenced codes, we recommend a Site Class C be used.

Using the Site Class and the appropriate seismic design maps or pertinent website (e.g., <https://ascehazardtool.org/>), the corresponding seismic design parameters from the CBC are as follows:

2025 CBC:

$S_s = 1.50g$	$S_{MS} = 1.54g$	$S_{DS} = 1.03g$
$S_1 = 0.56g$	$S_{M1} = 0.80g$	$S_{D1} = 0.54g$

Other seismic code values can be obtained directly from the tables in the building code using the above Site Class and pertinent website (e.g., <https://ascehazardtool.org/>).

The above seismic code values should be confirmed by the Project Structural Engineer using the Site Class above and the pertinent internet website and tables from the building code. The seismic design method for the structures should be determined by the Project Structural Engineer.

5.2.2 Strong Ground Motion Potential

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 ASCE Hazard Tool website (<https://ascehazardtool.org/>), and the USGS Earthquake Hazard Toolbox website (<https://earthquake.usgs.gov/nshmp/>), we computed that the site could be subjected to a peak ground acceleration (PGA_M) of 0.55g and a mean magnitude 6.8 earthquake.

This acceleration has been computed using the mapped Maximum Considered Geometric Mean peak ground acceleration from ASCE 7-22 (ASCE, 2022) 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.

5.2.3 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 at this site due to faulting is considered unlikely.

5.3 EARTHWORK

The earthwork for the planned improvements is anticipated to consist of clearing and excavation of undocumented fill and upper natural soils, subgrade preparation, and the placement and compaction of fill.

5.3.1 Clearing

Prior to grading, performing excavations or constructing the proposed improvements, the areas to be developed should be stripped of vegetation and cleared of debris and pavements. Buried obstructions, such as footings, abandoned utilities, and tree roots should be removed from areas to be developed. Deleterious material generated during clearing should be removed from the site. Existing vegetation should not be mixed into the soils.

Although not encountered in our explorations, cesspools or septic systems are likely to be encountered during grading due to the prior houses on the site. If cesspools or septic systems are encountered, they should be removed in their entirety. The resulting excavation should be backfilled with properly compacted fill soils. 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 further grading.

5.3.2 Excavations

Excavations at this site will include removals of undocumented fill and disturbed low-density natural soils, footing excavations, and trenching for proposed utility lines.

To provide uniform support for the planned buildings, prior to placement of fills or construction of the buildings, the existing fill within the proposed building pads should be removed and replaced as properly compacted fill. For planning purposes, removals for the building pads should extend to a depth of 3 feet below existing grades or 2 feet below the base of foundations, whichever is deeper. If bedrock materials or competent dense natural soils are encountered shallower than the recommendations above, the foundations may be supported directly on undisturbed bedrock or competent dense natural soils.

In the building pads, overexcavation to the maximum depth of the planned pad utilities with large excavating equipment may be a preferred construction method to the local use of lighter backhoes. Utility subcontractors may have difficulties excavating the narrow plumbing lines in the moderately weathered bedrock underlying the building pad.

Removals below minor structures, such as free-standing walls and trash enclosures, should extend to a depth of 2 feet below existing grades or 1 foot below the base of the foundation, whichever is deeper. If bedrock materials or competent natural dense soils are encountered shallower than the recommended removals, the foundations may be supported directly on the bedrock materials or competent dense soils. For pavement and flatwork subgrade, removals are not anticipated to be required unless loose or disturbed soils are encountered.

The actual depths of removals should be determined in the field during grading by GPI. The soils exposed at the base of the overexcavation should be processed in place as described in the "Subgrade Preparation" section of this report.

The Project Surveyor should accurately stake the corners of the areas to be overexcavated in the field. Where space is available, the base of the excavations should extend laterally at least 5 feet beyond the building lines or edge of foundations, or a minimum distance equal to the depth of overexcavation/compaction below finish grade (i.e., a 1:1 projection below the top outside edge of footings), whichever is greater. Building lines include the footprint of the building and other foundation supported improvements, such as canopies and attached site walls.

The contractor should determine the best method for removal based on the subsurface conditions outlined herein. Significant ripping and slow earthwork operations should be anticipated for excavations greater than about 3 to 6 feet or deeper from the existing ground surface in portions of the site. While not encountered in our borings, buried boulders or outcrops of hard granitic rock near the existing surface may be encountered and may require chipping, blasting, and/or special handling. The weathered granitic bedrock encountered could possibly be excavated using a larger excavator, by using a hydraulic concrete attachment, or ripping with D8R or D9R shank ripper.

Where not removed by the aforementioned excavations, existing utility trench backfill should be removed and replaced as properly compacted fill within the building pads. For planning purposes, removals over the utilities should extend to within 1-foot of the top of the pipe. For utilities that are 5 feet or shallower, the removal should extend laterally 1-foot beyond both sides of the pipe. For deeper utilities, the removals should include a zone defined by a 1:1 projection upward (and away from the pipe) from each side of the pipe. The actual limits of removal will be confirmed in the field. We recommend that known utilities be shown on the grading plan. Wet utilities left in-place outside building areas should be capped to reduce the potential for water to infiltrate into the building pad.

The upper soils at the site are expected to have a slight caving potential when exposed in open cuts. Temporary construction excavations may be made vertically into the undisturbed natural soils without shoring to a depth of 4 feet below adjacent grade. For cuts up to 9 feet deep, the slopes should be properly shored or sloped back to at least $\frac{3}{4}$:1 (horizontal:vertical) or flatter. For deeper cuts up to 15 feet, the slopes should be properly shored or sloped back to at least 1:1 or flatter. The allowable slope inclinations are measured from the toe to the top of the cut. Even at these inclinations, some raveling should be anticipated. The exposed slope face should be kept moist (but not saturated) during construction to reduce local sloughing. 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 adjacent existing site facilities should be properly shored to maintain support of adjacent elements. Excavations and shoring systems should meet the minimum requirements given in the State of California Occupational Safety and Health Standards.

Deeper removals along property lines or adjacent to existing improvements will require shoring or slot cuts. Recommendations for shoring are provided in the "Retaining Structures" section of the report. Removals that will undermine existing adjacent pavements or hardscape may utilize "ABC" slot cuts to depths not greater than 8 feet. Unsurcharged slot cuts up to 8 feet in height should not be wider than 6 feet and should be backfilled to finished grade prior to excavation of the adjacent four slots (two on each side of the excavated slot). A test slot should be performed prior to production slots to confirm the stability of the planned cuts.

5.3.3 Subgrade Preparation

After the recommended cuts and removals are performed and prior to placing fills or construction of the proposed improvements, the subgrade soils should be scarified to a depth of 12 inches, moisture conditioned, and compacted to at least 90 percent of the maximum dry density, determined in accordance with ASTM D1557. Where bedrock materials are encountered at the base of removals, subgrade processing may be omitted.

5.3.4 Material for Fill

The on-site soils are, in general, suitable for use as compacted fill, except for oversized materials greater than 8 inches in diameter. Fills within the depths of the planned footings should not contain particles greater than 3 inches in diameter to reduce difficulties in excavating footings in oversized materials.

Retaining wall backfill and fill placed directly beneath flatwork should consist of granular and non-expansive soils. If encountered, the limited on-site clayey soils should not be placed behind retaining walls or within one foot of finished grade in flatwork areas.

Imported fill material should be predominately granular (contain no more than 40 percent fines - portion passing No. 200 sieve) and non-expansive (E.I. less than 20). The import should also exhibit an R-value of at least 20 if used in proposed paved areas. 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 prior to importing. Each proposed import source should be sampled, tested and accepted for use prior to delivery of the soils to the site. Soils imported prior to acceptance by GPI may be rejected if not suitable.

Both imported and existing on-site soils to be used as fill should be free of debris and pieces larger than 8 inches in greatest dimension (3 inches if placed within the depth of the planned footings). If the on-site concrete is crushed to be re-used in compacted fill, we recommend the material be crushed to 3-inch minus in size and blended with the onsite soils prior to use.

5.3.5 Placement and Compaction of Fills

Fill soils within the footprints of the building pads should be placed in horizontal lifts, moisture-conditioned, and mechanically compacted to densities equal to at least 95 percent of the maximum dry density, determined in accordance with ASTM D1557. Backfill placed outside of the building footprints should be compacted to densities equal to at least 90 percent of the maximum dry density. The optimum lift thickness will depend on the compaction equipment used and can best be determined in the field.

The following uncompacted lift thickness can be used as preliminary guidelines.

Plate compactors	4-6 inches
Small vibratory or static rollers (5-ton±) or track equipment	6-9 inches
Scrapers, Heavy loaders, and large vibratory rollers	9-12 inches

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

In general, on-site soils should be placed at moisture contents of 1 to 3 percent over the optimum moisture content. Current moisture contents of the upper soils are generally highly variable with moistures ranging from approximately 1.5 to 17 percent. Moisture conditioning (drying and wetting) will likely be required.

Compacted fills should not be allowed to dry out prior to covering. If the fills are allowed to dry out prior to covering, additional moisture conditioning and processing (scarification/recompaction) will be required. A representative of GPI should observe and test the finished subgrade within 24 hours of concrete placement for floor slabs, hardscape, and pavements.

5.3.6 Shrinkage and Subsidence

Shrinkage is the loss of soil volume caused by compaction of fills to a higher density than before grading. Subsidence is the settlement of in-place subgrade soils caused by loads generated by large earthmoving equipment. For earthwork volume estimating purposes, an average shrinkage value of 0 to 5 percent may be assumed for the surficial soils. Subsidence is expected to be negligible. These values are estimates only and exclude losses due to removal of vegetation, boulders or debris. Actual shrinkage and subsidence will depend on the types of earthmoving equipment used and should be determined during grading.

5.3.7 Trench/Wall Backfill

Utility trench backfill consisting of the on-site materials or imported soil, or wall backfill consisting of granular material should be mechanically compacted in lifts. The limited on-site clays are not suitable for placement as retaining wall backfill. Lift thickness should not exceed those values given in the "Placement and Compaction of Fills" section of this report. Moisture conditioning (wetting and drying) of the on-site soils will be required prior to re-use as backfill. 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.

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).

5.3.8 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 and in-place compaction to evaluate the overall compaction of the soils. If foundations are to be supported on dense natural soils, in-place density testing should be performed to confirm the soil density prior to placement of concrete. Soils that do not meet minimum compaction requirements should be reworked and tested prior to placement of additional fill.

5.4 FOUNDATIONS

5.4.1 Foundation Type

As discussed previously, the proposed structures can be supported on conventional spread footings founded on either properly compacted fill or competent natural materials.

5.4.2 Allowable Bearing Pressures

Based on the shear strength and elastic settlement characteristics of the natural and recompacted on-site soils, a static allowable net bearing pressure of up to 5,000 pounds per square foot (psf) may be used for both continuous footings and isolated column footings for the buildings. These bearing pressures are for dead-plus-live-loads, and may be increased one-third for short-term, transient, wind and seismic loading. The actual bearing pressure used may be less than the value presented above and can be based on economics and structural loads to determine the minimum width for footings as discussed below. The maximum edge pressures induced by eccentric loading or overturning moments should not be allowed to exceed these recommended values.

For minor structures, such as site walls and property line screen walls, where reduced excavation depths have been recommended, we recommend a maximum allowable bearing capacity of 2,000 psf be used with minimum footing widths and depths of 18 inches.

5.4.3 Minimum Footing Width and Embedment

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

STATIC BEARING PRESSURE (psf)	MINIMUM FOOTING WIDTH (inches)	MINIMUM FOOTING* EMBEDMENT (inches)
5,000	48	24
4,000	36	24
3,000	24	18
2,000	12	18

* Refers to minimum depth below lowest adjacent grade at the time of foundation construction. If the floor slab is in-place prior to fully loading the footings, depth of interior footings may be taken from the top of the floor slab.

A minimum footing width of 12 inches and embedment of 18 inches should be used even if the actual bearing pressure is less than 2,000 psf.

5.4.4 Estimated Settlements

Total static settlement of isolated pad or continuous wall footings (up to 150 kips for columns and 3 to 5 kips per lineal foot for walls) is expected to be less than $\frac{3}{4}$ -inch. Differential static settlement between similarly loaded column footings or along a 40-foot span of a continuous footing is expected to be on the order of $\frac{1}{4}$ -inch or less.

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

5.4.5 Lateral Load Resistance

Soil resistance to lateral loads will be provided by a combination of frictional resistance between the bottom of footings and underlying soils and by passive soil pressures acting against the embedded sides of the footings. For frictional resistance, a coefficient of friction of 0.40 may be used for design. In addition, an allowable lateral bearing pressure equal to an equivalent fluid weight of 350 pounds per cubic foot may be used, provided the footings are poured tight against compacted fill. These values may be used in combination without reduction.

5.4.6 Foundation Inspection

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

5.5 BUILDING FLOOR SLABS

A vapor/moisture retarder should be placed under slabs that are to be covered with moisture-sensitive floor coverings (parquet, vinyl tile, etc.) or will be storing moisture sensitive supplies. Currently, common practice is to use a 15-mil polyolefin product such as Stego Wrap for this purpose. Whether to place the concrete slab directly on the vapor barrier or place a clean sand layer between the slab and vapor barrier is a decision for the Project Architect, as it is not a geotechnical issue. If covered by sand, the sand layer should be about 2 inches thick and contain less than 5 percent by weight passing the No. 200 sieve. Based on our explorations and laboratory testing, the soils at the site are not

suitable for this purpose. The function of the sand layer is to protect the vapor retarder during construction and to aid in the uniform curing of the concrete. This layer should be nominally compacted using light equipment. The sand placed over the vapor retarder should only be slightly moist. If the sand gets wet (for example as a result of rainfall or excessive moistening) it must be allowed to dry prior to placing concrete. Care should be taken to avoid infiltration of water into the sand layer after placement of the concrete slab, such as at slab cut-outs and other exposures. A sand layer is not required beneath the vapor retarder, but we take no exception if one is provided.

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

5.6 RETAINING STRUCTURES

The following recommendations are provided for walls or shoring less than 8 feet in height. We recommend that walls be backfilled with non-expansive, granular soils (less than 40 percent passing the No. 200 sieve). Based on our subsurface explorations, such material is anticipated to be readily available onsite.

Active earth pressures can be used for designing cantilevered walls or shoring that can yield laterally at least $\frac{1}{2}$ -percent of the wall height under the imposed loads. For level, drained backfill, derived from granular, non-expansive soils, a lateral pressure of an equivalent fluid weighing of 36 pounds per cubic foot may be used. This value can also be used for design of temporary cantilevered shoring.

At-rest pressures should be used for restrained walls that remain rigid enough to be essentially non-yielding. For select, non-expansive, level, drained backfill, a lateral pressure of an equivalent fluid weighing 56 pounds per cubic foot can be used.

As outlined in the California Building Code, site retaining walls 6 feet or taller should be designed to resist seismic lateral earth pressures. A lateral pressure equivalent to a fluid with a unit weight of 20 pounds per cubic foot may be used. This pressure should be combined with the active earth pressure presented above. If the retaining walls are designed using the at-rest pressure provided above, only the difference between the active plus seismic pressures and the at-rest pressure needs to be included as the seismic pressure.

The recommended pressures are based on the assumption that the supported earth will be fully drained, preventing the build-up of hydrostatic pressures. For traditional backfilled retaining walls, a drain consisting of perforated pipe and 1 cubic foot of gravel per lineal foot, wrapped in filter fabric should be used. The fabric (non-woven filter fabric, Mirafi 140N or equivalent) should be lapped at the top.

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.

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

5.7 EXTERIOR CONCRETE AND MASONRY FLATWORK

Exterior concrete and masonry flatwork should be supported on non-expansive, compacted fill which appears to be readily available onsite. This includes the slabs for trash enclosures, generators, etc. The use of the clayey soils within 2-feet of the slab subgrade should not be permitted unless the client indicates that differential heave is tolerable (if so, this should be stated on the project civil plans). This includes exterior sidewalks, stamped concrete, trash enclosures, non-traffic pavement, pavers, etc. Prior to placement of concrete, the subgrade should be prepared as recommended in the "Subgrade Preparation" section of this report.

5.8 PAVEMENTS

A test on the upper soils resulted in an R-value of 64. To account for the variability of the onsite soils an R-value of 50 was used in the design of the onsite pavements. The California Division of Highways Design Method was used for design of the recommended preliminary pavement sections. The following preliminary pavement sections are recommended for planning purposes only.

PAVEMENT AREA	TRAFFIC INDEX	SECTION THICKNESS (inches)	
		ASPHALT/PORTLAND CONCRETE	AGGREGATE BASE COURSE
Asphalt Concrete			
Automobile Parking & Drives	4.0/5.0	3.0	4.0
Truck/Bus Drives	6.0	4.0	4.0
Heavy Truck/Bus Drives	7.0	4.0	5.0
Portland Cement Concrete			
Automobile Parking & Drives	4.0	5.5	--
Truck/Bus Drives	6.0	6.0	--
Heavy Truck/Bus Drives	7.0	6.5	--

If vehicular pavers are to be used for the project, the paver and leveling sand should be supported on at least the thickness of aggregate base shown above for the appropriate asphalt concrete traffic index.

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

The portland cement concrete used for paving should have a compressive strength of at least 3,500 psi at the time the pavement is subjected to vehicular traffic.

The pavement base course should be compacted to at least 95 percent of the maximum dry density (ASTM D1557). 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 will be properly drained. The design of paved areas should incorporate measures to prevent moisture build-up within the base course and subgrade, 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.

5.9 CORROSION

Laboratory testing by HDR (Appendix C) on a selected sample indicates soluble sulfate content of 22 mg/kg. For the 2025 CBC, foundation concrete should conform to the requirements outlined in ACI 318, Section 4.3 for category S0 levels of soluble sulfate exposure. Chloride levels in the sample tested was found to be 56 mg/kg. Due to the potential exposure to moisture, we recommend at least a Category C1 be used in design.

Resistivity testing of representative samples of the onsite soils indicates that they are moderately corrosive to buried ferrous metals. GPI does not practice corrosion protection engineering. If corrosion protection recommendations are required, a corrosion engineer should be consulted to provide recommendations to protect these elements from corrosion.

5.10 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 buildings. We also recommend landscape areas adjacent to the building or hardscape areas be drained, with consideration of installing an impermeable liner.

5.11 STORMWATER INFILTRATION

Based on the presence of the perched groundwater and relatively shallow bedrock, storm water infiltration is not to be feasible at the site. Guidelines for storm water infiltration by the County of Riverside (Riverside, 2011) requires that infiltration should be at least 5 feet higher than the bedrock (impermeable layer) which was encountered as shallow as 1½ feet below existing grades in our explorations.

5.12 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.

6.0 LIMITATIONS

This report, exploration logs, and other materials resulting from GPI's efforts were prepared exclusively for VBSB, LLC 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 projects other than the currently proposed development, as it may not contain sufficient or appropriate information for such uses. If this report or portions of this report are provided to contractors or included in specifications, it should be understood that they are provided for information only. This report cannot be utilized by another entity without the express written permission of GPI.

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

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

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

Respectfully submitted,
Geotechnical Professionals Inc.



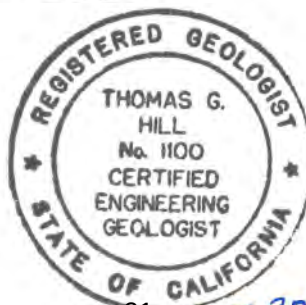
Patrick I.F. McGervey, G.E.
Senior Engineer



Donald A. Cords, G.E.
Principal



Thomas Hill, CEG
Consulting Geologist



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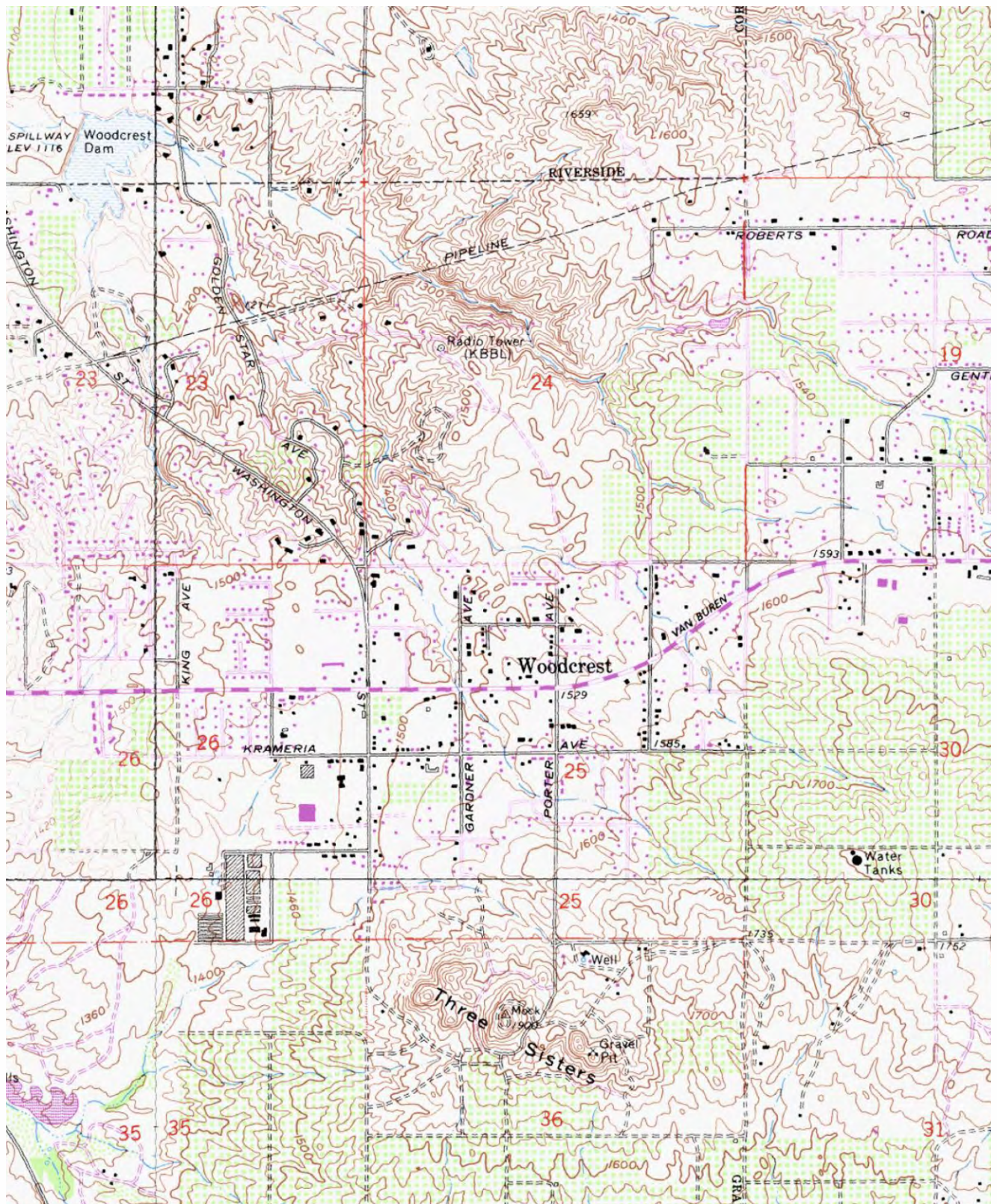
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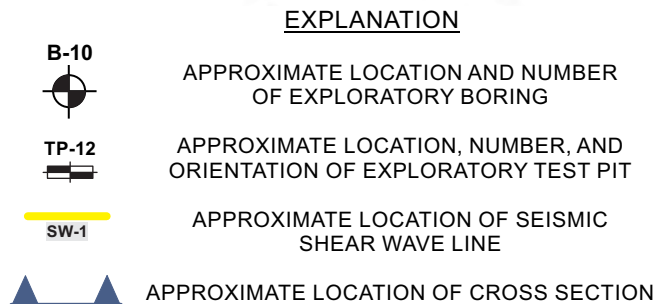
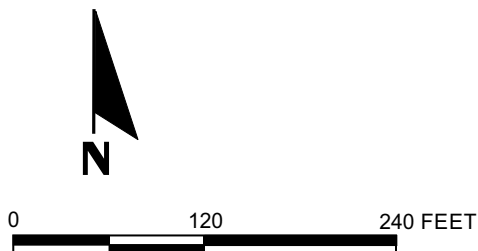
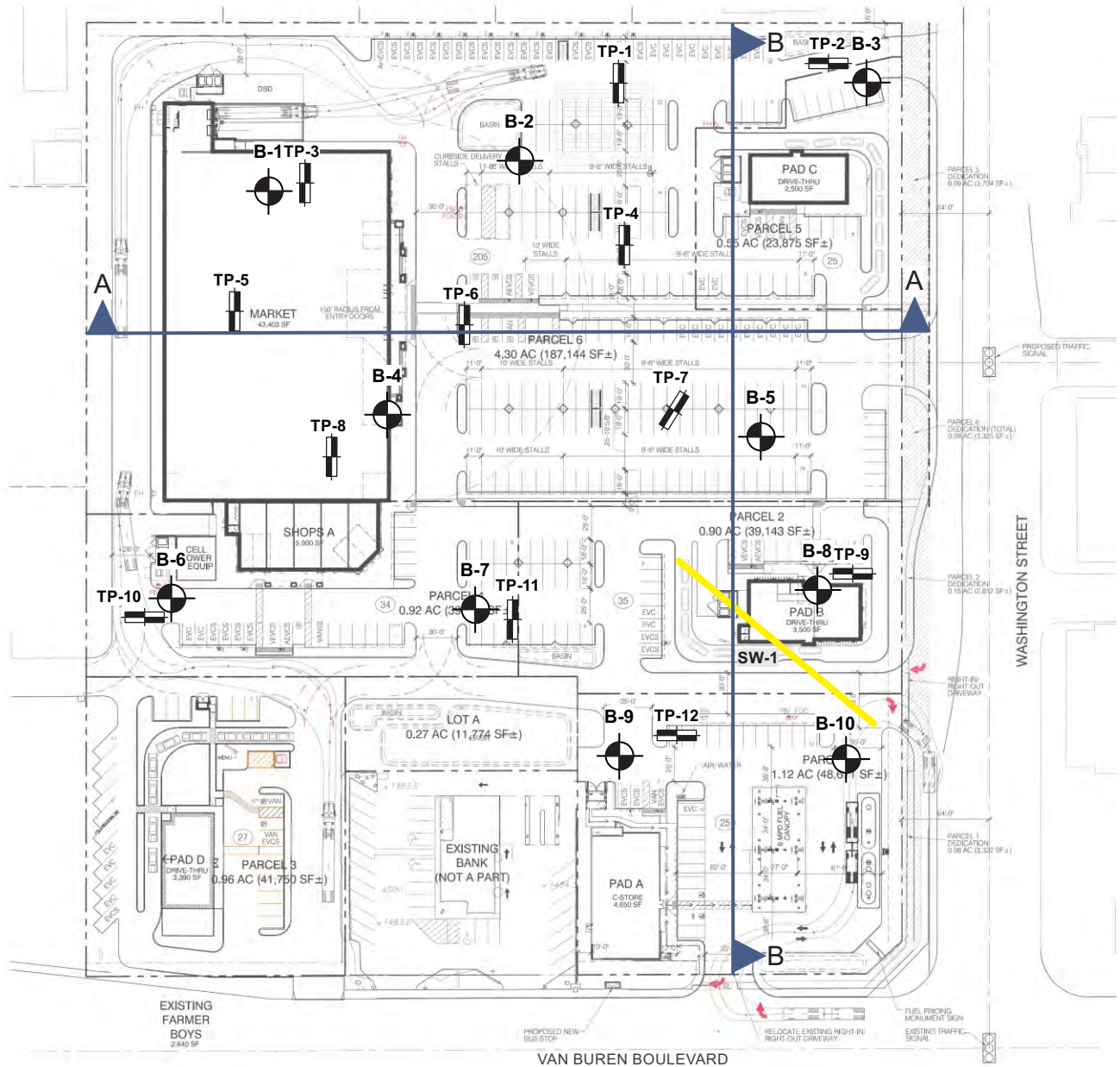
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BASE PLAN REPRODUCED FROM CONCEPTUAL SITE PLAN SCHEME Ev9 BY GK PIERCE ARCHITECTURE DATED JAN 21, 2026



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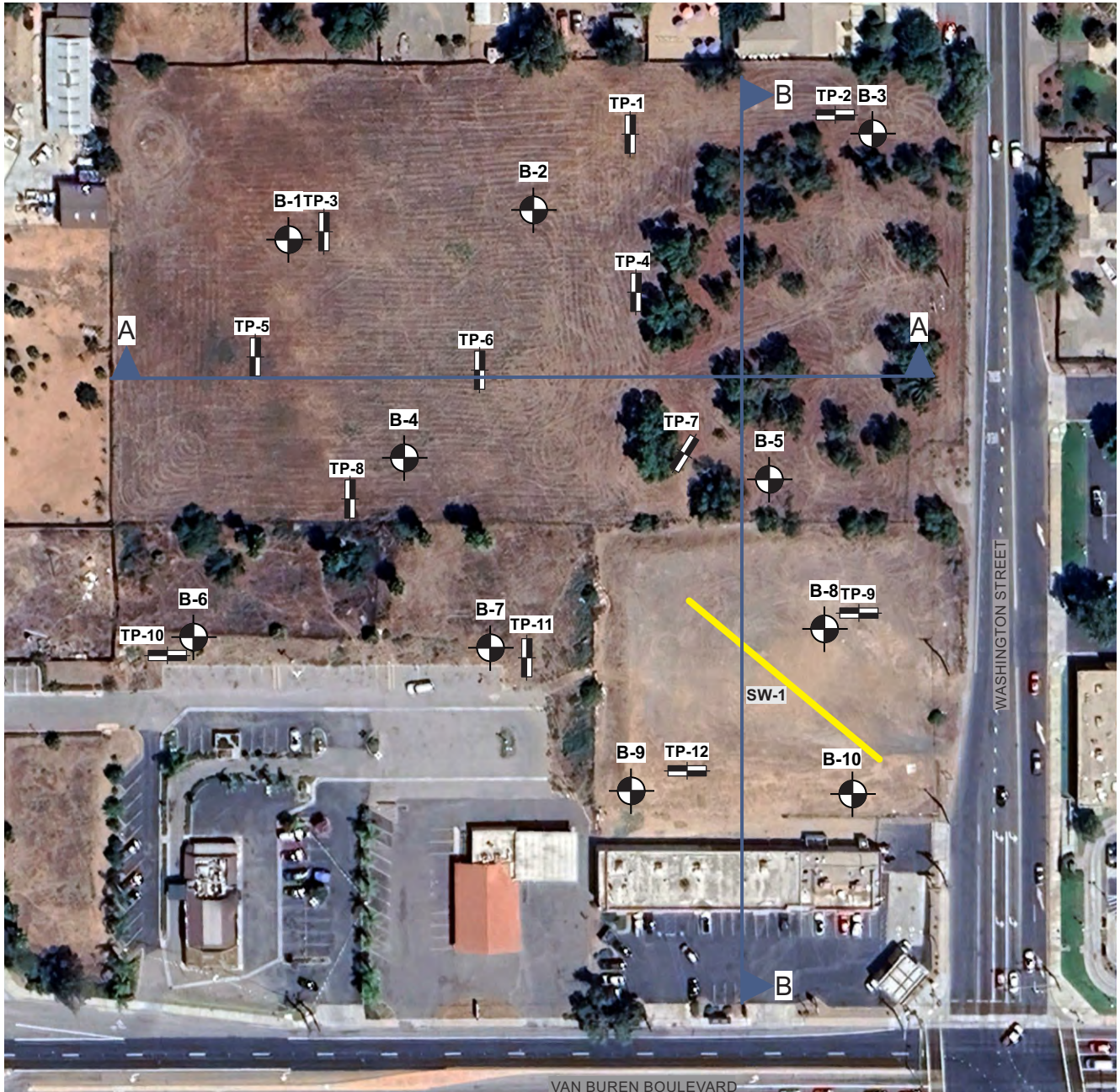
WOODCREST RETAIL

GPI PROJECT NO.: 2995.1

SCALE: 1" = 120'

SITE PLAN (Proposed)

FIGURE 2



EXPLANATION



B-10

APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING



TP-12

APPROXIMATE LOCATION, NUMBER, AND ORIENTATION OF EXPLORATORY TEST PIT

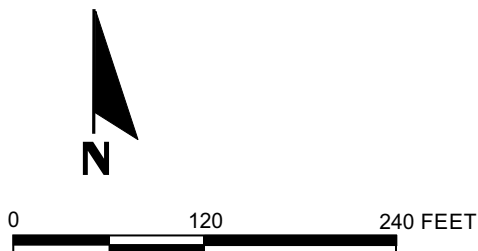


SW-1

APPROXIMATE LOCATION OF SEISMIC SHEAR WAVE LINE



APPROXIMATE LOCATION OF CROSS SECTION



BASE PLAN REPRODUCED FROM GOOGLE EARTH 2025



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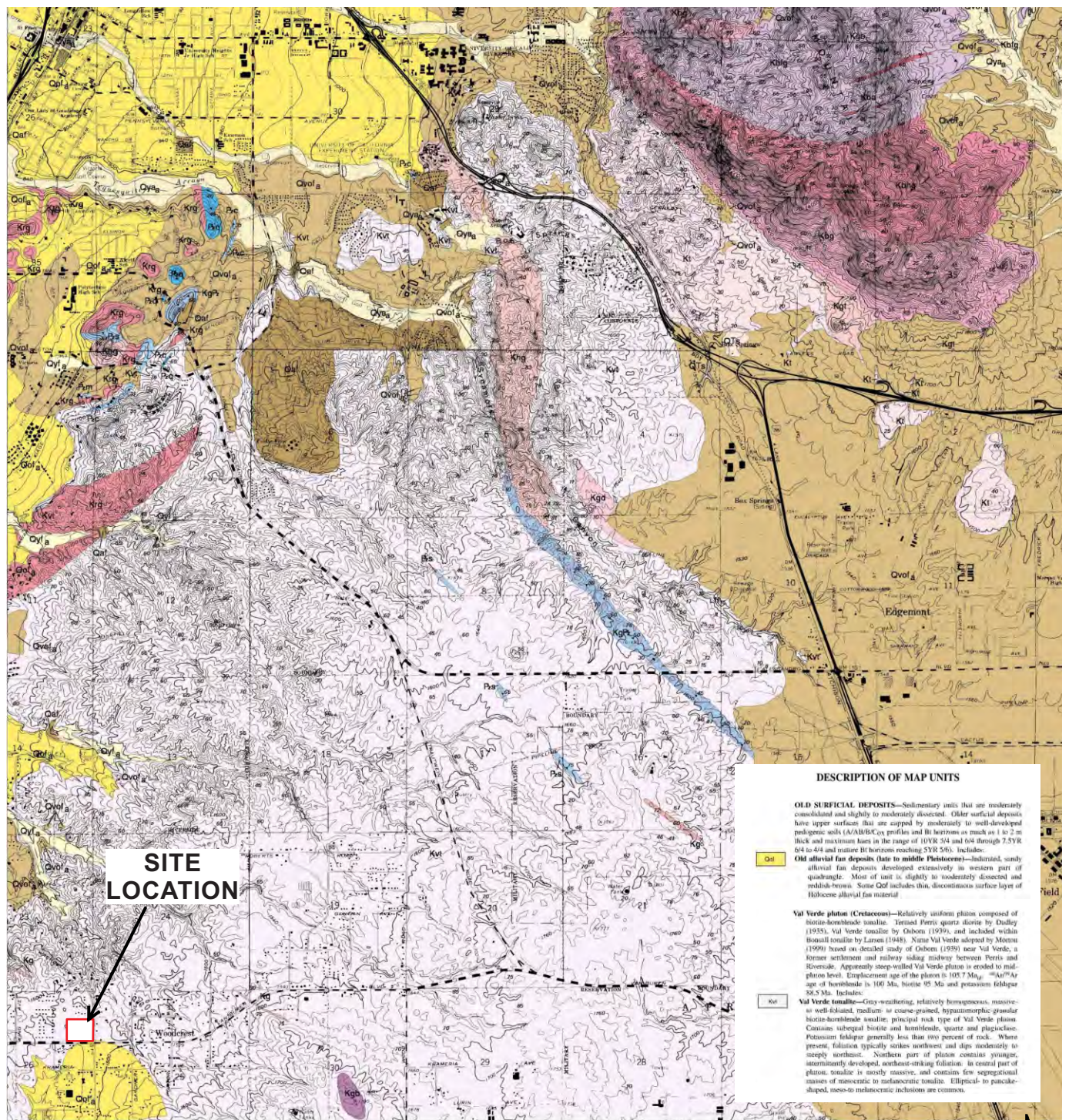
WOODCREST RETAIL

GPI PROJECT NO.: 2995.1

SCALE: 1" = 120'

SITE PLAN (Existing)

FIGURE 3



BASE MAP REPRODUCED FROM THE GEOLOGIC MAP OF THE RIVERSIDE EAST 7.5' QUADRANGLE PREPARED BY DOUGLAS M. MORTON & BRETT COX; DATED 1988



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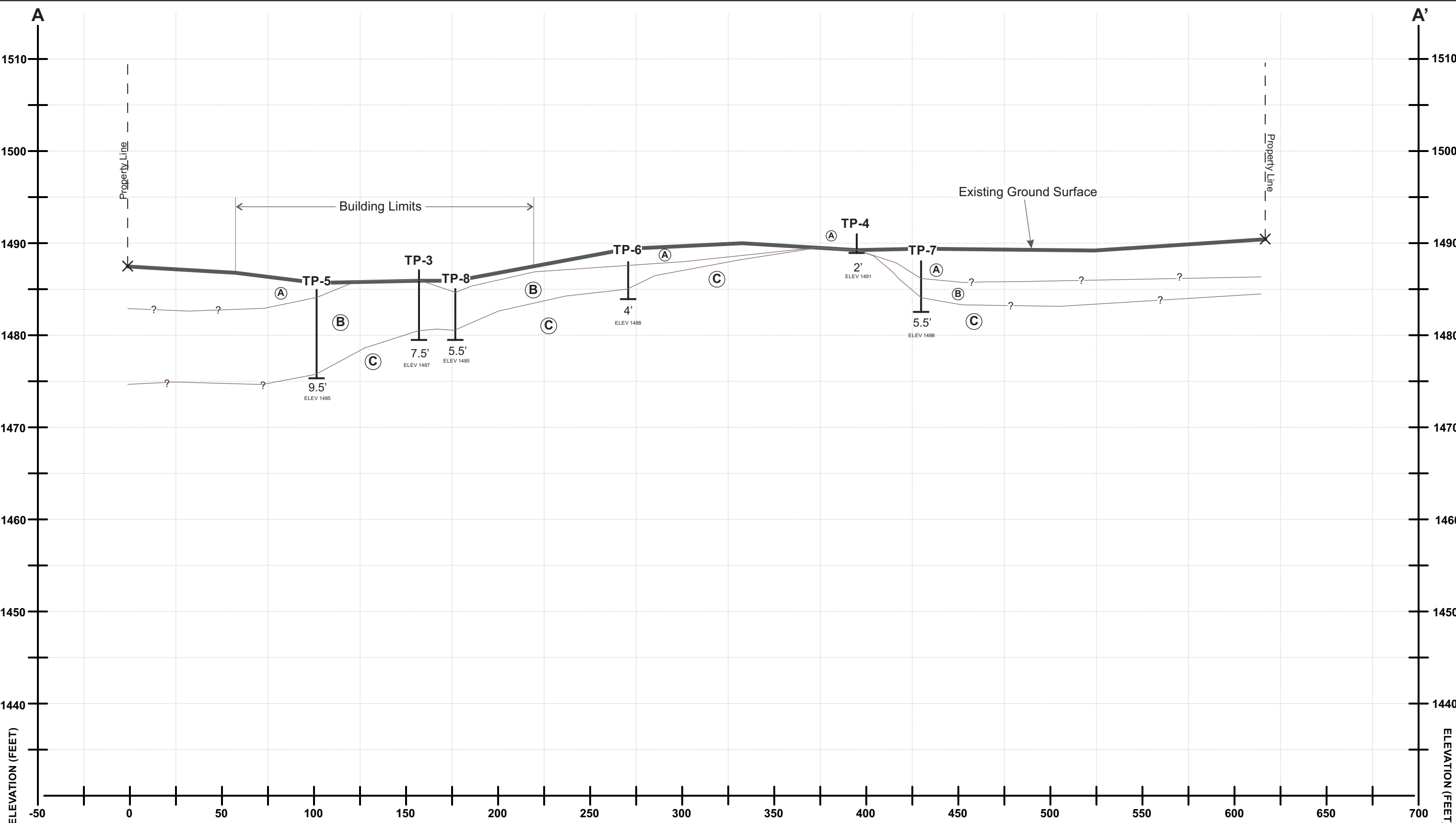
WOODCREST RETAIL

GPI PROJECT NO.: 2995.I

SCALE: 1" = 5000'

REGIONAL GEOLOGIC MAP

FIGURE 4



EXPLANATION

- (A) Fill Soils
- (B) Residual Soils: Sandy Clay and Clayey Sand
- (C) Weathered Granite Bedrock

Note: This section is based upon information obtained at test pits exploration 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.



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WOODCREST RETAIL

GPI PROJECT NO.: 2995.I

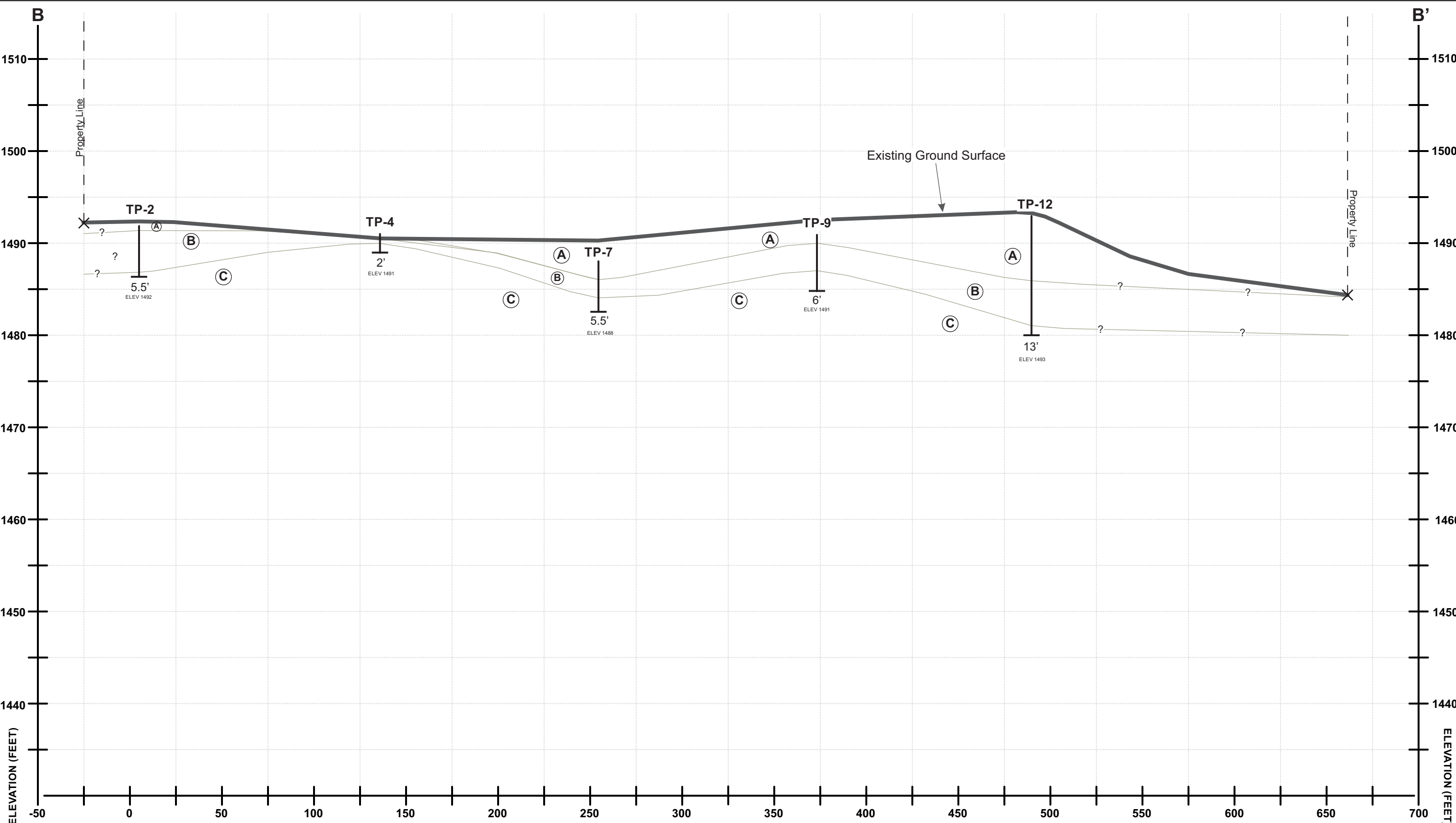
SCALE AS SHOWN

SUBSURFACE CROSS SECTION A-A'

FIGURE 5.1

HORIZONTAL: 1" = 50'

VERTICAL: 1" = 10'



EXPLANATION

- (A) Fill Soils
- (B) Residual Soils: Sandy Clay and Clayey Sand
- (C) Weathered Granite Bedrock

Note: This section is based upon information obtained at test pits exploration 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.



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WOODCREST RETAIL

GPI PROJECT NO.: 2995.I

SCALE AS SHOWN

HORIZONTAL: 1" = 50'
VERTICAL: 1" = 10'

SUBSURFACE CROSS SECTION B-B'

FIGURE 5.2

APPENDIX A

APPENDIX A

EXPLORATORY BORINGS

The subsurface conditions at the site were investigated by drilling and sampling ten exploratory borings. The borings were advanced to depths of 6 to 21 feet below the existing ground surface. The exploration locations are shown on the Site Plans, Figures 2 and 3.

The borings were drilled using truck-mounted hollow-stem auger equipment. Relatively undisturbed samples were obtained using a brass-ring lined sampler (ASTM D 3550). The brass-rings have an inside diameter of 2.42 inches. The ring samples were driven into the soil by a 140-pound hammer dropping 30 inches. The number of blows needed to drive the sampler into the soil was recorded as the penetration resistance.

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, employing two turns of rope around the cathead. 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.

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 are presented in Figures A-1 to A-10 in this appendix.

The boring locations were laid out in the field by measuring from existing features at the site. The borings were backfilled with the excavated soil cuttings. The ground surface elevations at the boring locations were estimated from Google Earth 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.		
	10.5 17.4 5.5 3.6 2.2 2.2	102 96 76 50/5" 50/3" 50/4"	50/5"	B	0		<u>Residual Soils:</u> SILTY SAND (SM) brown, moist, very dense, decomposed granite	1490
				D	5		SANDY CLAY (CL) red brown, moist, hard, decomposed granite	1485
				S			<u>Weathered Granitic Bedrock:</u> SAND WITH SILT (SP-SM) light brown, slightly moist, very dense, highly weathered SAND (SP) light brown, slightly moist to dry, very dense, highly weathered @ 12 feet, brown- grey, moderately weathered @ 15 feet, perched groundwater	1480
				S	10			
				S				
				S				
				S	15			
		Refusal @ 15.5 feet						
SAMPLE TYPES ☐ Rock Core ☐ Standard Split Spoon ☐ Drive Sample ☐ Bulk Sample ☐ Tube Sample						DATE DRILLED: 7-17-20 EQUIPMENT USED: 8 " Hollow Stem Auger GROUNDWATER LEVEL (ft): Not Encountered		
						PROJECT NO.: 2995.I WOODCREST RETAIL		
						LOG OF BORING NO. B-1		
						FIGURE A-1		

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS		ELEVATION (FEET)
						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		Residual Soils: SILTY SAND (SM) brown, moist, decomposed granite Weathered Granitic Bedrock: CLAYEY SAND (SC)/SILTY SAND (SM) orange brown, moist to slightly moist, very dense, moderately weathered SAND WITH SILT (SP-SM) brown, slightly moist to dry, very dense, hard rock pieces, moderately weathered @ 14 feet, perched groundwater @ 15 feet, grey, very moist, slightly weathered, difficult drilling and chattering Total Depth 16 feet	1490
	13.0	112	90/8"	D				
	3.6	122						
	3.7	121	50/4"	D	5			
	2.3	109	50'4"	D				
	2.6	116	51	D	10			1485
	10.8	119	50/4"	D	15			1480
SAMPLE TYPES ☐ Rock Core ☒ Standard Split Spoon ☐ Drive Sample ☐ Bulk Sample ☐ Tube Sample						DATE DRILLED: 7-17-20 EQUIPMENT USED: 8 " Hollow Stem Auger GROUNDWATER LEVEL (ft): Not Encountered		PROJECT NO.: 2995.I WOODCREST RETAIL
 LOG OF BORING NO. B-2 FIGURE A-2								

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS		ELEVATION (FEET)
						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.		
	16.1	107	64	D	0		<u>Residual Soils:</u> SILTY SAND (SM) brown, moist, decomposed granite	1500
	13.7	110			<u>Weathered Granitic Bedrock:</u> SILTY SAND (SM) brown, wet to very moist, very dense, moderately weathered			
	6.5	110	98/9"	D	5		SAND (SP) light brown, moist, very dense, moderately weathered @ 7 feet, dry <	

	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.	
	4.3	136	80/10"	D	0	<u>Residual Soils:</u> SILTY SAND (SM) orange brown, slightly moist. decomposed granite	
			50/4"	D	5	<u>Weathered Granitic Bedrock:</u> SAND WITH SILT (SP-SM) orange brown, slightly moist to moist, very dense	1485
			50/4"	D			
			50/4"	D	10		1480
			50/3"	D	15	@ 15 feet, grey, wet, perched groundwater	1475
						Refusal @ 16 feet	

SAMPLE TYPES
☐ Rock Core
☒ Standard Split Spoon
☐ Drive Sample
☐ Bulk Sample
☐ Tube Sample

DATE DRILLED:
 7-17-20
EQUIPMENT USED:
 8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft):
 Not Encountered

LOG OF BORING NO. B-4

PROJECT NO.: 2995.I

WOODCREST RETAIL

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.		
			B	0		<u>Residual Soil:</u> SAND WITH SILT (SP-SM) brown, moist, decomposed granite	1495
7.7	106	80/10"	D			<u>Weathered Granitic Bedrock:</u> SILTY SAND (SM) orange brown, moist to dry, very dense, moderately weathered	
1.5	113					SAND WITH SILT (SP-SM) grey, dry, very dense	
2.5	122	90/9"	D	5		@ 7 feet, difficult drilling and chattering, pieces of rock, slightly weathered	1490
2.8	103	50/4"	D			Total Depth 8 feet	

SAMPLE TYPES

C

S

D

B

T

Rock Core

Standard Split Spoon

Drive Sample

Bulk Sample

Tube Sample

DATE DRILLED:

7-17-20

EQUIPMENT USED:

8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):

Not Encoutered

GPI

PROJECT NO.: 2995.I

WOODCREST RETAIL

LOG OF BORING NO. B-5

FIGURE A-5

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
						This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
	2.9	124	50/4"	D	0	<u>Residual Soils:</u> SILTY SAND (SM) brown, moist, decomposed granite	1480
			50/4"	D	5	<u>Weathered Granitic Bedrock:</u> SAND WITH SILT (SP-SM) gray brown, dry, very dense, moderately weathered	
						Refusal @ 6 feet	

SAMPLE TYPES
☐ Rock Core
☐ Standard Split Spoon
☐ Drive Sample
☐ Bulk Sample
☐ Tube Sample

DATE DRILLED:
7-17-20
EQUIPMENT USED:
8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft):
Not Encountered



PROJECT NO.: 2995.I
WOODCREST RETAIL

LOG OF BORING NO. B-6
FIGURE A-6

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
						This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
	3.3	129	87/9"	D	0	<u>Residual Soils:</u> SILTY SAND (SM) brown, moist, decomposed granite	1485
						<u>Weathered Granitic Bedrock:</u> SAND WITH SILT (SP-SM) orange brown, dry, very dense, moderately weathered	
	4.6	120	92/10"	D	5	@ 5 feet, slightly moist	
	8.1	120	82/11"	D		@ 7 feet, moist, slightly weathered, hard drilling and chattering	
						Total Depth 8 feet	

SAMPLE TYPES

☐ C Rock Core

☐ S Standard Split Spoon

☐ D Drive Sample

☐ B Bulk Sample

☐ T Tube Sample

DATE DRILLED:
7-17-20

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered



PROJECT NO.: 2995.I
WOODCREST RETAIL

LOG OF BORING NO. B-7

FIGURE A-7

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS		ELEVATION (FEET)	
						This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.			
	16.2	105	29	D	0		<u>Residual Soils:</u> SILTY SAND (SM) brown, moist, decomposed granite	1490	
	4.4	174	50/4"	D	<u>Weathered Granitic Bedrock:</u> SAND WITH SILT (SP-SM) light brown, slightly moist, medium dense, highly weathered				
	32.9	86	73	D	@ 5 feet, very dense, moderately weathered				
					@ 7 feet, with clay, wet, slightly weathered, hard drilling and chattering				
						Total Depth 8 feet			
SAMPLE TYPES						DATE DRILLED:			PROJECT NO.: 2995.I
☐ Rock Core						7-17-20			WOODCREST RETAIL
☐ Standard Split Spoon						EQUIPMENT USED:			LOG OF BORING NO. B-8
☐ Drive Sample						8 " Hollow Stem Auger			
☐ Bulk Sample						GROUNDWATER LEVEL (ft):		FIGURE A-8	
☐ Tube Sample						Not Encountered			

	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.		
	7.9	123	30	D	0		<u>Residual Soils:</u> SILTY SAND (SM) brown, moist, decomposed granite	1490 1485
	9.0	123	30	D	5		<u>Weathered Granitic Bedrock:</u> SILTY SAND (SM) brown, moist to very moist, medium dense, highly weathered	
	17.4	110	20	D			@ 7 feet, wet	
	14.6	119	58/11"	D	10		SAND WITH SILT (SP-SM) brown, wet, very dense, slightly weathered, difficult drilling and chattering	
							Total Depth 11 feet	

SAMPLE TYPES

- ☒ Rock Core
- ☒ Standard Split Spoon
- ☒ Drive Sample
- ☒ Bulk Sample
- ☒ Tube Sample

DATE DRILLED:
7-17-20

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encoutered

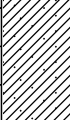


PROJECT NO.: 2995.I

WOODCREST RETAIL

LOG OF BORING NO. B-9

FIGURE A-9

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS		ELEVATION (FEET)
						This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.		
	16.6	110	31	D	0		1490	
								
	5.3 2.6	59 60	50/4"	D	5			
			50/3"	D				
						Total Depth @ 8 feet		

APPENDIX B

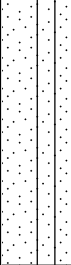
APPENDIX B

EXPLORATORY TEST PITS

The subsurface conditions at the site were investigated by excavating and sampling 12 exploratory test pits. The excavations were advanced to depths of approximately 2 to 9 ½ feet below the existing ground surface using a JCB 1400 Site Master Backhoe with a 24-inch wide bucket. The approximate locations of the explorations are shown on the Site Plans, Figures 2 and 3.

The field explorations were performed under the continuous technical supervision of GPI's representative, who visually inspected the site, maintained detailed logs of the test pits, and classified the soils encountered. Bulk samples of the subsurface soils were obtained and bagged at select locations within the excavations. The soils encountered in the test pits were classified in the field and through further examination in the laboratory in accordance with the Unified Soils Classification System. Detailed logs of the explorations are presented in Figures B-1 to B-12 in this Appendix.

The test pit locations were laid out in the field by measuring from existing site features. The ground surface elevations at the boring locations were estimated from Google Earth and should be considered approximate. The test pit excavations were backfilled as temporary fill without proper compaction. We placed a PVC pipe in the temporary fill of each test pit location prior to backfilling.

	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 test pit and at the time of trenching. 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: SANDY CLAY (CL) red brown, moist, with brush	1490
							<u>Residual Soils:</u> SANDY CLAY (CL) red brown, moist to very moist, decomposed granite	
				B	5		<u>Weathered Granitic Bedrock:</u> SAND WITH SILT (SP-SM) grey, slightly moist, moderately weathered, difficulty excavating	
							Backhoe refused @ 7.5 feet	

SAMPLE TYPES ☐ C Rock Core ☐ S Standard Split Spoon ☐ D Drive Sample ☐ B Bulk Sample ☐ T Tube Sample	DATE TRENCHED: 10-28-20 EQUIPMENT USED: 24" Backhoe Bucket GROUNDWATER LEVEL: Not Encountered	 LOG OF TEST PIT NO. TP-01	PROJECT NO.: 2995.I WOODCREST RETAIL
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FIGURE B-1

	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 test pit and at the time of trenching. 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: SANDY CLAY (CL) red brown, moist, with brush	1490
							<u>Residual Soils:</u> SANDY CLAY (CL) red brown, moist to very moist, decomposed granite	
				B			<u>Residual Soils:</u> SILTY SAND (SM) brown, moist to very moist, decomposed granite	
					5		<u>Weathered Granitic Bedrock:</u> SAND WITH SILT (SP-SM) grey, slightly moist, moderately weathered, difficulty excavating Backhoe refused @ 5.5 feet	

SAMPLE TYPES

☐ C Rock Core

☐ S Standard Split Spoon

☐ D Drive Sample

☐ B Bulk Sample

☐ T Tube Sample

DATE TRENCHED: 10-28-20

EQUIPMENT USED: 24" Backhoe Bucket

GROUNDWATER LEVEL: Not Encountered



PROJECT NO.: 2995.I

WOODCREST RETAIL

LOG OF TEST PIT NO. TP-02

FIGURE B-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 test pit and at the time of trenching. 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: SANDY CLAY (CL) red brown, moist, with brush	1485
				B			Residual Soils: SANDY CLAY (CL) red brown, moist to very moist, decomposed granite	
							CLAYEY SAND (SC) red brown, moist, decomposed granite	
					5		Weathered Granitic Bedrock: SAND WITH SILT (SP-SM) grey, slightly moist, moderately weathered, difficulty excavating	1480
							Backhoe refused @ 7.5 feet	

SAMPLE TYPES ☐ C Rock Core ☐ S Standard Split Spoon ☐ D Drive Sample ☐ B Bulk Sample ☐ T Tube Sample		DATE TRENCHED: 10-28-20 EQUIPMENT USED: 24" Backhoe Bucket GROUNDWATER LEVEL: Not Encountered			PROJECT NO.: 2995.I WOODCREST RETAIL
LOG OF TEST PIT NO. TP-03					FIGURE B-3

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS		ELEVATION (FEET)
						This summary applies only at the location of this test pit and at the time of trenching. 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: SANDY CLAY (CL) red brown, moist, with brush	1490
							<u>Residual Soils:</u> SANDY CLAY (CL) red brown, slightly moist, decomposed granite	
							<u>Weathered Granitic Bedrock:</u> SAND WITH SILT (SP-SM) grey, slightly moist, moderately weathered, difficulty excavating Backhoe refused @ 2 feet	

SAMPLE TYPES ☐ Rock Core ☐ Standard Split Spoon ☐ Drive Sample ☐ Bulk Sample ☐ Tube Sample	DATE TRENCHED: 10-28-20 EQUIPMENT USED: 24" Backhoe Bucket GROUNDWATER LEVEL: Not Encountered	 LOG OF TEST PIT NO. TP-04	PROJECT NO.: 2995.I WOODCREST RETAIL
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FIGURE B-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 test pit and at the time of trenching. 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: SANDY CLAY (CL) red brown, moist, with brush	1480
				B	Residual Soils: SANDY CLAY (CL) red brown, moist to very moist, decomposed granite			
					5		CLAYEY SAND (SC) red brown, moist, decomposed granite	
							<u>Weathered Granitic Bedrock:</u> SAND WITH SILT (SP-SM) grey, slightly moist, moderately weathered, difficulty excavating Backhoe refused @ 9.5 feet	

SAMPLE TYPES ☐ C Rock Core ☐ S Standard Split Spoon ☐ D Drive Sample ☐ B Bulk Sample ☐ T Tube Sample	DATE TRENCHED: 10-28-20 EQUIPMENT USED: 24" Backhoe Bucket GROUNDWATER LEVEL: Not Encountered		PROJECT NO.: 2995.I WOODCREST RETAIL
		LOG OF TEST PIT NO. TP-05	
		FIGURE A-5	

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS		ELEVATION (FEET)
						This summary applies only at the location of this test pit and at the time of trenching. 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: SANDY CLAY (CL) red brown, moist, with brush	1485
							<u>Residual Soils:</u> CLAYEY SAND (SC) red brown, moist to very moist, decomposed granite	
							<u>Weathered Granitic Bedrock:</u> SAND WITH SILT (SP-SM) grey, moist, moderately weathered, difficulty excavating Backhoe refused @ 4 feet	

SAMPLE TYPES ☐ Rock Core ☐ Standard Split Spoon ☐ Drive Sample ☐ Bulk Sample ☐ Tube Sample	DATE TRENCHED: 10-28-20 EQUIPMENT USED: 24" Backhoe Bucket GROUNDWATER LEVEL: Not Encountered	 LOG OF TEST PIT NO. TP-06	PROJECT NO.: 2995.I WOODCREST RETAIL
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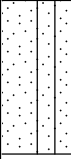
	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS		ELEVATION (FEET)
						This summary applies only at the location of this test pit and at the time of trenching. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.		
					0		Natural: SANDY CLAY (CL) brown, moist	1485
							<u>Residual Soils:</u> CLAYEY SAND (SC) red brown, moist, decomposed granite	
			B	5	<u>Weathered Granitic Bedrock:</u> SAND WITH SILT (SP-SM) grey, slightly moist, moderately weathered, difficulty excavating			
							Backhoe refused @ 5.5 feet	

SAMPLE TYPES ☐ C Rock Core ☐ S Standard Split Spoon ☐ D Drive Sample ☐ B Bulk Sample ☐ T Tube Sample	DATE TRENCHED: 10-28-20 EQUIPMENT USED: 24" Backhoe Bucket GROUNDWATER LEVEL: Not Encountered		PROJECT NO.: 2995.I WOODCREST RETAIL
LOG OF TEST PIT NO. TP-07			FIGURE 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 test pit and at the time of trenching. 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: SANDY CLAY (CL) red brown, moist, with brush	1480
				B			<u>Residual Soils:</u> CLAYEY SAND (SC) red brown, moist, decomposed granite	
					5		<u>Weathered Granitic Bedrock:</u> SAND WITH SILT (SP-SM) grey, slightly moist, moderately weathered, difficulty excavating Backhoe refused @ 5.5 feet	

SAMPLE TYPES ☐ C Rock Core ☐ S Standard Split Spoon ☐ D Drive Sample ☐ B Bulk Sample ☐ T Tube Sample	DATE TRENCHED: 10-28-20 EQUIPMENT USED: 24" Backhoe Bucket GROUNDWATER LEVEL: Not Encountered	 LOG OF TEST PIT NO. TP-08	PROJECT NO.: 2995.I WOODCREST RETAIL
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FIGURE B-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 test pit and at the time of trenching. 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: CLAYEY SAND (SC) brown, moist	1490
							<u>Residual Soils:</u> SILTY SAND (SM) white with brown streaks, dry to slightly moist, decomposed granite	
							CLAYEY SAND (SC) red brown, moist, decomposed granite	
					5		<u>Weathered Granitic Bedrock:</u> SAND WITH SILT (SP-SM) grey, slightly moist, moderately weathered, difficulty excavating	1485
							Backhoe refused @ 6 feet	

SAMPLE TYPES ☐ Rock Core ☐ Standard Split Spoon ☐ Drive Sample ☐ Bulk Sample ☐ Tube Sample	DATE TRENCHED: 10-28-20 EQUIPMENT USED: 24" Backhoe Bucket GROUNDWATER LEVEL: Not Encountered	 LOG OF TEST PIT NO. TP-09	PROJECT NO.: 2995.I WOODCREST RETAIL
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FIGURE B-9

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS		ELEVATION (FEET)
						This summary applies only at the location of this test pit and at the time of trenching. 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		Residual Soils: CLAYEY SAND (SC) red brown, slightly moist to moist	
							Weathered Granitic Bedrock: SAND WITH SILT (SP-SM) grey, slightly moist, moderately weathered, difficulty excavating Backhoe refused @ 2 feet	

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

DATE TRENCHED: 10-28-20

EQUIPMENT USED:
 24" Backhoe Bucket

GROUNDWATER LEVEL:
 Not Encountered



PROJECT NO.: 2995.I
 WOODCREST RETAIL

LOG OF TEST PIT NO. TP-10

FIGURE B-10

	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 test pit and at the time of trenching. 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		<u>Residual Soils:</u> CLAYEY SAND (SC) red brown, moist, decomposed granite	1480
							<u>Weathered Granitic Bedrock:</u> SAND WITH SILT (SP-SM) grey, moist, moderately weathered, difficulty excavating	
					5		Backhoe refused @ 5 feet	

SAMPLE TYPES
☐ Rock Core
☐ Standard Split Spoon
☐ Drive Sample
☐ Bulk Sample
☐ Tube Sample

DATE TRENCHED: 10-28-20

EQUIPMENT USED:
24" Backhoe Bucket

GROUNDWATER LEVEL:
Not Encountered


LOG OF TEST PIT NO. TP-11

PROJECT NO.: 2995.I
WOODCREST RETAIL

FIGURE B-11

	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 test pit and at the time of trenching. 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: SANDY CLAY (CL) red brown, moist, trace debris	1490
					5		
						<u>Residual Soils:</u> SANDY CLAY (CL) red brown, moist to very moist, decomposed granite	
					10	CLAYEY SAND (SC) red brown, very moist, decomposed granite	1485
						<u>Weathered Granitic Bedrock:</u> SAND WITH SILT (SP-SM) grey, slightly moist, moderately weathered, difficulty excavating	
						Backhoe refused @ 13 feet	1480
SAMPLE TYPES ☐ C Rock Core ☐ S Standard Split Spoon ☐ D Drive Sample ☐ B Bulk Sample ☐ T Tube Sample						DATE TRENCHED: 10-28-20 EQUIPMENT USED: 24" Backhoe Bucket GROUNDWATER LEVEL: Not Encountered	PROJECT NO.: 2995.I WOODCREST RETAIL LOG OF TEST PIT NO. TP-12 FIGURE B-12

APPENDIX C

APPENDIX C

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

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. The percentages passing the No. 200 sieve are tabulated below.

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	PERCENT PASSING No. 200 SIEVE
B-2	2	Silty Sand (SM)	14
TP-2	3-5	Clayey Sand (SC)	45
TP-10	0-2	Clayey Sand (SC)	34

DIRECT SHEAR

Direct shear tests were performed on undisturbed and remolded bulk samples in accordance with ASTM D3080. The bulk sample was remolded to approximately 95 percent of the maximum dry density. The test specimens 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 tests. The results of the direct shear tests are presented in Figures B-1 to B-4.

COMPACTION TEST

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

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	OPIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)
B-1	0-5	Silty Sand (SM)	8.5	137
TP-2, 5 & 10	0-5	Silty Sand (SM)	9.0	135

EXPANSION INDEX TEST

An expansion index tests was performed on representative bulk samples. The test was performed in accordance with ASTM D4829 to assess the expansion potential of the on-site fill soils. The results of the tests are summarized below.

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	EXPANSION INDEX, EI	EXPANSION POTENTIAL
TP-2, 5 & 10	0-5	Silty Sand (SM)	10	Very Low

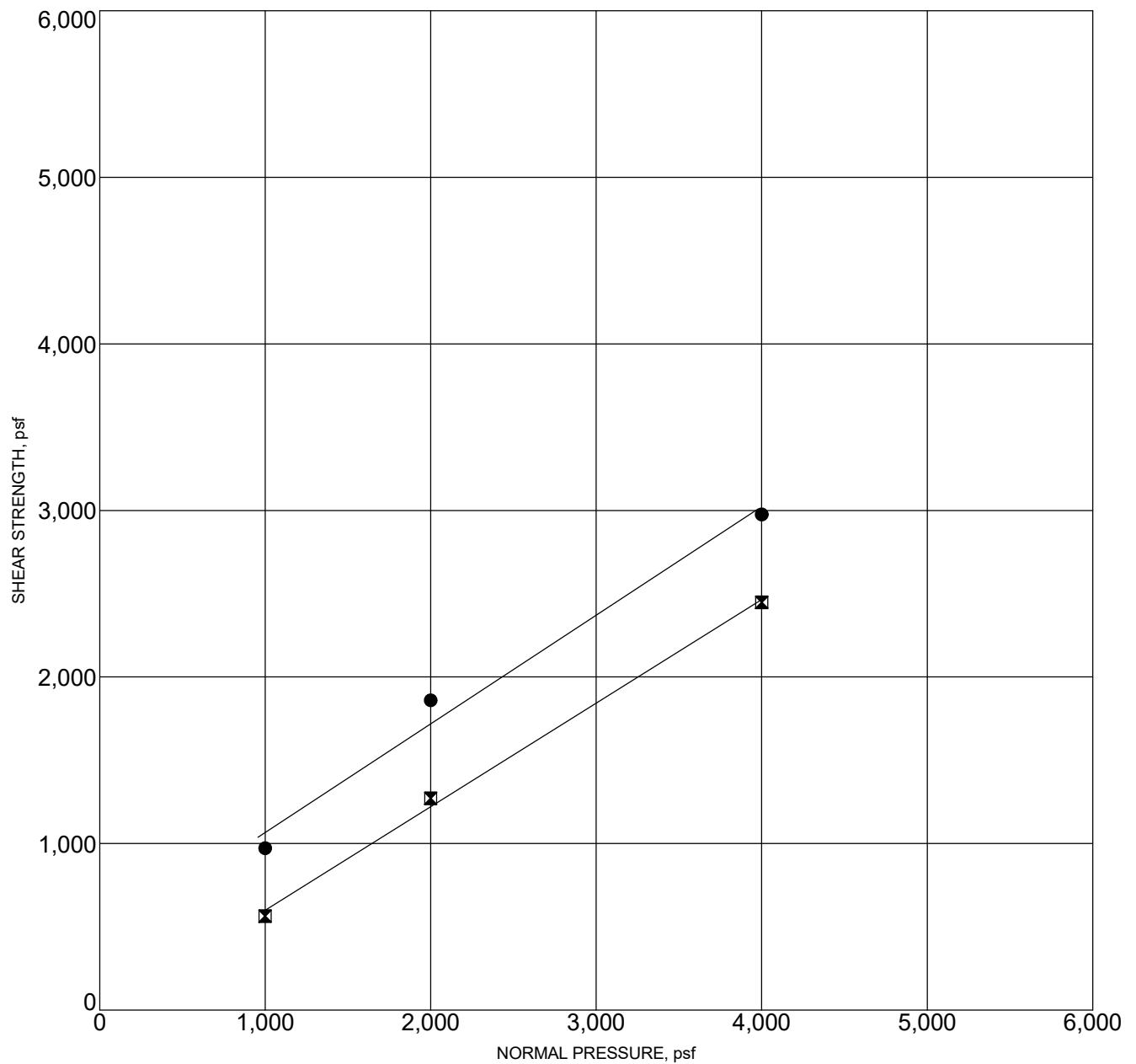
R-VALUE

Suitability of the near-surface soils for pavement 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
TP-10	0-2	Clayey Sand (SC)	64

CORROSIVITY

Soil corrosivity testing was performed by HDR a soil sample provided by GPI. The test results are summarized in Table 1 of this Appendix.



● **PEAK STRENGTH**

Friction Angle= 33 degrees

Cohesion= 414 psf

✕ **ULTIMATE STRENGTH**

Friction Angle= 32 degrees

Cohesion= 0 psf

Note: Samples remolded to 95% of maximum dry density

Sample Location		Classification	DD,pcf	MC,%
B-1	0-5	SILTY SAND (SM)	130	8.5

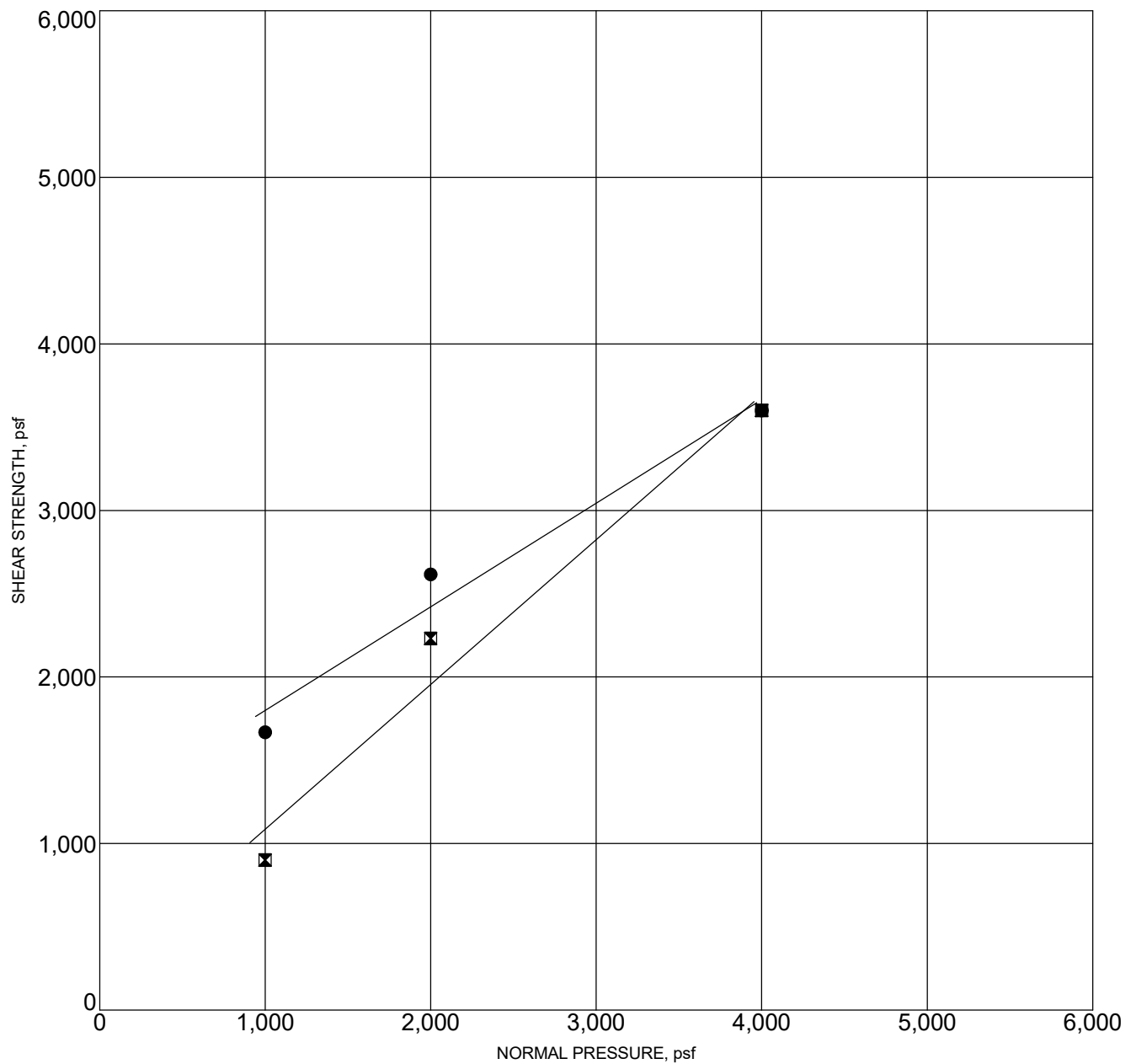
PROJECT: WOODCREST RETAIL

PROJECT NO.: 2995.I



DIRECT SHEAR TEST RESULTS

FIGURE C-1



● **PEAK STRENGTH**
Friction Angle= 32 degrees
Cohesion= 1176 psf

✕ **ULTIMATE STRENGTH**
Friction Angle= 41 degrees
Cohesion= 216 psf

Sample Location		Classification	DD,pcf	MC,%
B-4	1.0	<u>Weathered Bedrock</u> SAND WITH SILT (SP-SM)	136	4.3

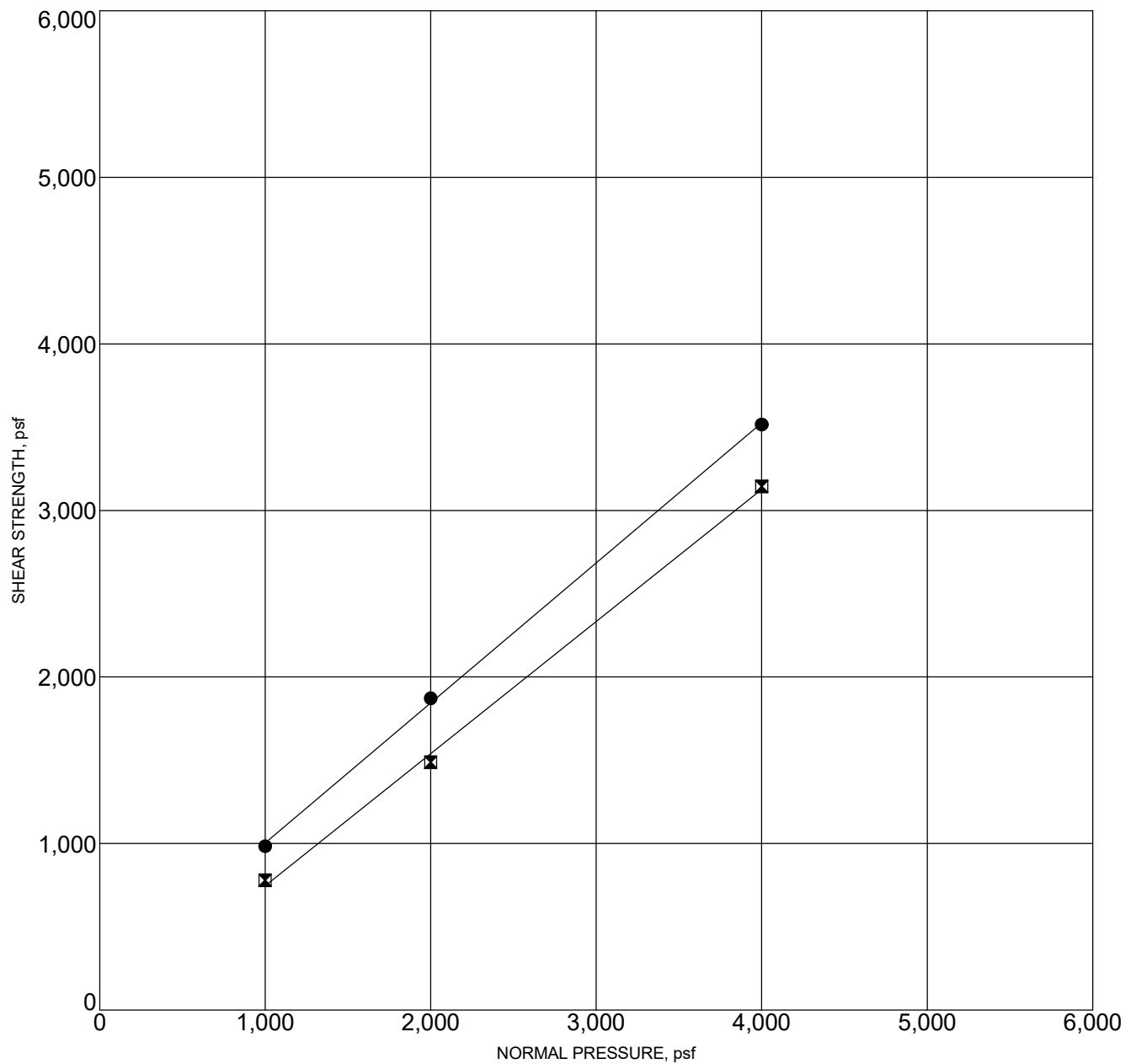
PROJECT: WOODCREST RETAIL

PROJECT NO.: 2995.I



DIRECT SHEAR TEST RESULTS

FIGURE C-2



● **PEAK STRENGTH**
 Friction Angle= 40 degrees
 Cohesion= 162 psf

✕ **ULTIMATE STRENGTH**
 Friction Angle= 38 degrees
 Cohesion= 0 psf

Sample Location		Classification	DD,pcf	MC,%
B-9	5.0	<u>Weathered Bedrock</u> SILTY SAND (SM)	123	9.0

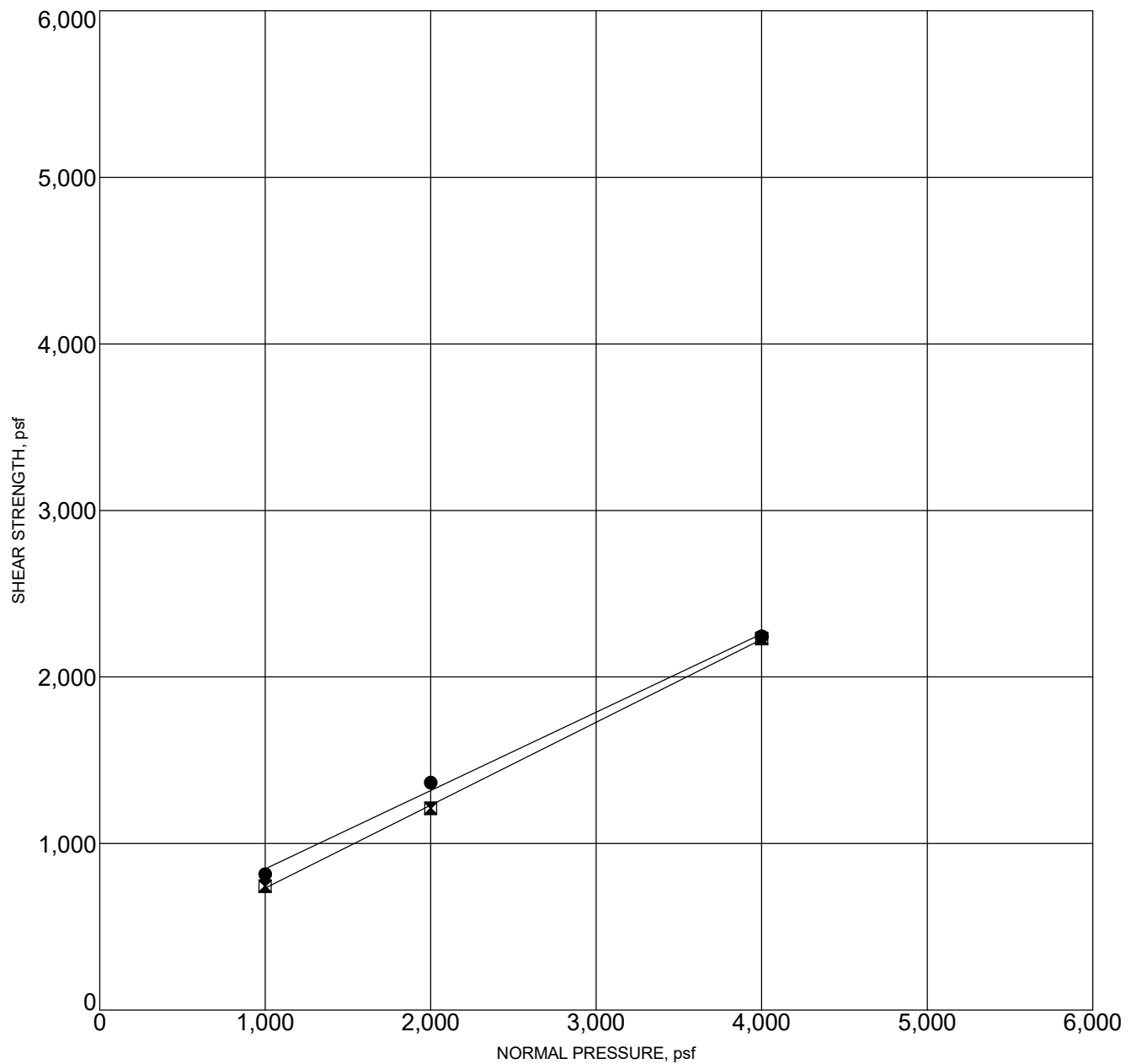
PROJECT: WOODCREST RETAIL

PROJECT NO.: 2995.I



DIRECT SHEAR TEST RESULTS

FIGURE C-3



● **PEAK STRENGTH**

Friction Angle= 25 degrees

Cohesion= 377 psf

✕ **ULTIMATE STRENGTH**

Friction Angle= 26 degrees

Cohesion= 234 psf

Note: Samples remolded to 90% of maximum dry density.

Sample Location		Classification	DD,pcf	MC,%
TP-10,15,& 2	0-5	SILTY SAND (SM)	122	9.0

PROJECT: WOODCREST RETAIL

PROJECT NO.: 2995.I



DIRECT SHEAR TEST RESULTS

FIGURE C-4



Table 1 - Laboratory Tests on Soil Samples

Geotechnical Professionals, Inc.
Woodcrest Retail
Your #2995.I, HDR Lab #21-0097LAB
19-Feb-21

Sample ID

TP-5
@ 0.5-2

Resistivity	Units	
as-received	ohm-cm	3,240
saturated	ohm-cm	2,040

pH 7.3

Electrical

Conductivity mS/cm 0.11

Chemical Analyses

Cations

calcium	Ca ²⁺	mg/kg	38
magnesium	Mg ²⁺	mg/kg	12
sodium	Na ¹⁺	mg/kg	77
potassium	K ¹⁺	mg/kg	5.8
ammonium	NH ₄ ¹⁺	mg/kg	ND

Anions

carbonate	CO ₃ ²⁻	mg/kg	ND
bicarbonate	HCO ₃ ¹⁻	mg/kg	174
fluoride	F ¹⁻	mg/kg	19
chloride	Cl ¹⁻	mg/kg	56
sulfate	SO ₄ ²⁻	mg/kg	22
nitrate	NO ₃ ¹⁻	mg/kg	20
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.

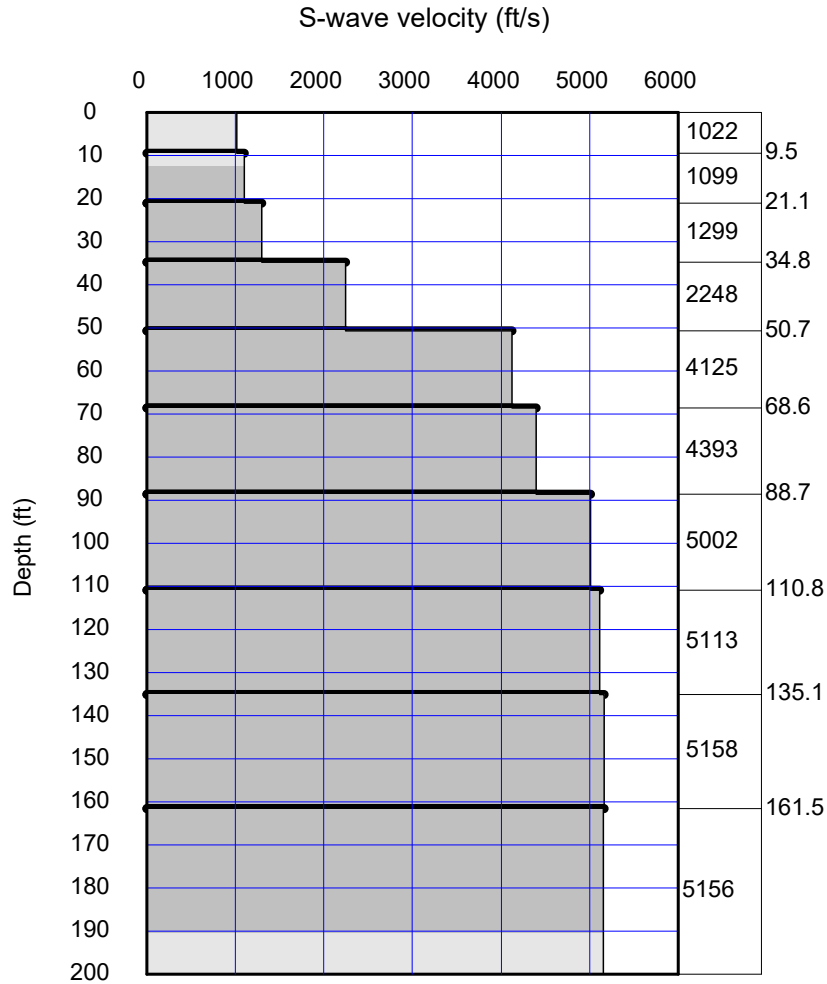
Redox = oxidation-reduction potential in millivolts

ND = not detected

na = not analyzed

APPENDIX D

SEISMIC LINE SW-1



SHEAR-WAVE VELOCITY MODEL: Average Vs 100ft = 2,056.1 ft/sec

Site Classification (ASCE 7-22 Ch. 20)- "C" ("Very Dense Sand or Hard Clay" profile)

Client: Geotechnical Professionals Inc.

Project Name: Woodcrest Retail Center, Woodcrest, California

Survey Line End Coordinates: 33.88334, -117.36695 / 33.88303, -117.36647

Date: 4/7/26

TG Project No. 264231-1

 **TERRA
GEOSCIENCES**
P.O. Box 1090
Loma Linda, CA 92354

