GEOTECHNICAL PROFESSIONALS INC.

GEOTECHNICAL INVESTIGATION
PROPOSED TRAILER LOT EXPANSION
17486 NISQUALLI ROAD
VICTORVILLE, CALIFORNIA

Prepared for:
BRE Space Paxbello, LLC
3401 Etiwanda Avenue
Jurupa Valley, California 91752

Prepared by:
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Project No. 3149.I

January 4, 2022

GEOTECHNICAL
PROFESSIONALS INC.

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BRE Space Paxbello LLC
3401 Etiwanda Avenue
Jurupa Valley, California 91752

Attention: Taline Agopian
Senior Project Manager, Development

Subject: Report of Geotechnical Investigation
Proposed Trailer Lot Expansion
17486 Nisqualli Road
Victorville, California
GPI Project No. 3149.I

Dear Taline:

Transmitted herewith is our report of geotechnical investigation for the subject project. The report presents the results of our evaluation of the subsurface conditions at the site and recommendations for design and construction.

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

Very truly yours,
Geotechnical Professionals Inc.

Patrick McGervey, P.E.
Project Engineer

Paul R. Schade, G.E.
Principal

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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 trailer lot expansion at the subject site in Victorville, California. The site location is shown on the Site Location Map, Figure 1.

1.2 PROJECT DESCRIPTION

The proposed project will consist of a new paved trailer parking lot and drives across the approximately 8.3-acre site. There will also be a new guard shack building located at the southwest corner of the new parking lot. Floor slabs for the guard shack will be supported on-grade. The project will also include storm water infiltration systems, and landscaping on the remainder of the site.

Proposed finished elevations were not available at the time of preparing this report, however grades are anticipated to be predominately within 2 to 4 feet of existing grades. The finished grades for the proposed guard shack are anticipated to be within 2 to 4 feet of existing grades. Based on similar past projects, we assume that maximum wall loads will be on the order of 2 kips per lineal foot (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 subsurface exploration, field infiltration testing, laboratory testing, engineering analysis and the preparation of this report.

Our subsurface exploration consisted of six hollow stem auger borings and two infiltration test wells. The borings were performed to depths of approximately 4 to 26 feet below existing grade and the percolation wells were installed at depths of 10 to 12 feet below existing grades. Boring B-6 was refused on concrete prior to reaching its desired depth of 5 feet. A description of field procedures and logs of the borings are presented in the attached Appendix A. The procedures and results of the infiltration tests are discussed in this report. The approximate locations of the subsurface explorations are shown on the Site Plan, Figure 2.

Laboratory testing was 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, R-value and maximum density. R-value testing was performed by Geo-Logic under subcontract to GPI. Their test results are presented Appendix B. Corrosivity testing was performed as part of a previous investigation of the adjacent site by others (CHJ, 2016). The results of their testing have been incorporated 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 bound to the north, west, and east by three different industrial/distribution buildings with associated surface trailer parking, and west of local rail spurs adjacent to a drainage channel and rail tracks. The site is predominately vacant with pockets of brush. Stockpiles of soil on the order of 5 feet high are in the southeast corner of the site that are likely associated with the previous cogeneration facility that (based on historical images) appears to have been deconstructed in 2015.

In general, the site slopes gently downward from south to north with a change in ground surface elevation from about Elevation +2902 feet to +2894 feet across the site.

3.2 SUBSURFACE SOIL CONDITIONS

Our field investigation disclosed a subsurface profile consisting of fill soils overlying natural soils. Detailed descriptions of the conditions encountered are shown on the Log of Borings in Appendix A.

We encountered undocumented fills to approximately 2 to 5 feet below existing grade in the explorations. The fill materials encountered consisted of medium dense, dry to slightly moist silty sands and sands with varying amount of gravel. The deeper fill soils were predominately associated with the existing unpaved entrance drive along the southern property line at the site. Limited areas may have deeper undocumented fill soils in the vicinity of the previous cogeneration plant (near boring B-6) in the southeastern corner of the site.

The natural soils consist predominately of silty sand with varying amounts of gravel and possible cobbles to a depth of approximately 13 to 15 feet where we encountered layered clayey sands, silty sands, and gravelly sands. In general, the native soils were dense to very dense and very stiff to hard. The natural soils have moderate to high strength and low compressibility characteristics.

Groundwater was not encountered in our explorations drilled to a maximum depth of 26 feet below ground surface. Published data by the California Department of Water Resources indicates groundwater is deeper than 100 feet below the ground surface.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 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 were reported to depths of up to 2 to 5 feet below existing grade in the vicinity of the proposed guard shack building. The fill soils are not considered to be suitable for direct support of foundations or floor slabs without remedial earthwork. For the proposed guard shack, 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.

- Current moisture contents of the upper soils are generally well below the optimum moisture content so that moisture conditioning (wetting) will be required.

- The upper on-site soils are predominantly dry to slightly moist, medium dense silty sands and sands with silt. As such, the soils are considered to be susceptible to caving in open cuts and excavations. Care should be taken to maintain support of the soils and structures left in-place adjacent to planned excavations.

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

4.2 SEISMIC DESIGN

4.2.1 General

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

We assume the seismic design of the proposed development will be in accordance with the 2022 California Building Code (CBC) criteria. Based on the results of our investigation, a Site Class D may be used for the seismic design of the proposed building.

4.2.2 Strong Ground Motion Potential

Based on published information (geohazards.usgs.gov), the most significant fault in the proximity of the site is the San Andreas (San Bernardino N.), which is located about 18 miles from the site.

During the life of the project, the site will likely be subject to strong ground motions due to earthquakes on nearby faults. Based on the USGS website (earthquake.usgs.gov), we computed that the site could be subjected to a peak ground acceleration (PGA_M) of 0.55g for a mean magnitude 7.0 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:

2022 CBC:

$S_S = 1.20g$	$S_{MS} = F_a * S_S = 1.22g$	$S_{DS} = 2/3 * S_{MS} = 0.82g$
$S_1 = 0.46g$	$S_{M1} = F_V * S_1 = 0.85g$	$S_{D1} = 2/3 * S_{M1} = 0.56g$

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. The Project Structural Engineer should also evaluate the period of the proposed structure with respect to the T_S value above when reviewing whether a site-specific response analysis will be requested.

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 at this site due to faulting is considered unlikely.

4.2.4 Liquefaction and Seismic Settlement

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. Additionally, the site is not located in a zone identified as having a potential for liquefaction by the County. Due to the deep historic groundwater levels, we do not anticipate liquefaction induced settlement to negatively impact the site.

Seismic ground subsidence, not related to liquefaction, occurs when loose, granular soils above the groundwater are densified during strong earthquake shaking. Based on our analyses, we estimate a potential dry seismic settlement of less than ¼-inch. The differential seismic settlement is estimated to be less than ¼-inch across a span of 60 feet.

4.3 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.

4.3.1 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. Buried obstructions, such as abandoned utilities, and tree roots should be removed from areas to be developed. Deleterious material generated during the clearing operation should be removed from the site. Existing vegetation should not be mixed into the soils.

Although not encountered in our explorations, 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.

4.3.2 Excavations

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

Building Pad, Pavements and Minor Structures

To provide uniform support for the planned building, prior to placement of fills or construction of the building, the existing fill and a portion of the upper natural soils within the proposed building pad should be removed and replaced as properly compacted fill. For planning purposes, removals for the building pad should extend to a depth of 3 feet below existing grades and at least 2 feet below the base of foundations, whichever is deeper.

Removals below minor structures, such as free-standing walls and trash enclosures, should extend to a depth of 2 feet below existing grade or 1 foot below the base of the foundation, whichever is deeper. For pavement and flatwork subgrade, removals should extend at least 1 foot below existing grades or the proposed subgrade, whichever is deeper.

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.

Excavation of the soils at the site should be readily achieved using conventional methods. The contractor should determine the best method for removal based on the subsurface conditions outlined herein.

Lateral Limits

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.

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

The sandy soils at the site are expected to have a moderate to severe caving potential when exposed in open cuts. We recommend the following maximum slope inclinations for temporary excavations:

Excavation Height (ft)	Slope (h:v)
<3	Vertical
<8	¾:1
<15	1:1

If cuts greater than 15 feet are planned, we should be contacted to provide further recommendations. 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.

Slot Cuts

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. Unsurcharged slot cuts up to 6 feet in height should not be wider than 8 feet. The slot cuts should be backfilled to finished grade prior to excavation of the adjacent four slots (two on each side of the excavated slot). We can provide slot widths for other slot heights if required. A test slot should be performed prior to production slots to confirm the stability of the planned cuts.

4.3.3 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. Moisture conditioning (wetting) of the on-site soils anticipated.

4.3.4 Material for Fill

The upper on-site soils are, in general, suitable for use as compacted fill with some moisture conditioning being required. Although not encountered in our explorations, expansive clayey soils (E.I. greater than 50) were encountered in prior nearby investigations at the site and should not be used as fill within the upper 2 feet below the proposed building pad, or within the upper 1 foot below concrete flatwork subgrade.

Imported fill material should be predominately granular (contain no more than 40 percent fines - portion passing No. 200 sieve) and non-expansive (E.I. of 20 or less). 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 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 on-site soils prior to use.

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).

4.3.5 Placement and Compaction of Fills

Fill soils should be placed in horizontal lifts, moisture-conditioned, and mechanically compacted to densities equal to at least 90 percent of the maximum dry density, determined in accordance with ASTM D1557. Fills within one foot of the subgrade pavement areasggregate base material should be compacted to a relative compaction of 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	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 predominately slightly below optimum moisture content. Some moisture conditioning (wetting) will 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 will be required. A representative of GPI should observe and test the finished subgrade within 24 hours of concrete placement for floor slabs and hardscape.

4.3.6 Shrinkage and Subsidence

Shrinkage is the loss of soil volume caused by compaction of fills to a higher density than before grading. Subsidence is the settlement of in-place subgrade soils caused by loads generated by large earthmoving equipment. For earthwork volume estimating purposes, an average shrinkage value of 2 to 7 percent may be assumed for the surficial soils. Subsidence is expected to be less than 0.1 feet. These values are estimates only and exclude losses due to removal of vegetation or debris. Actual shrinkage and subsidence will depend on the types of earthmoving equipment used and should be determined during grading.

4.3.7 Trench/Wall Backfill

Utility trench backfill consisting of the on-site materials or imported soil, or wall backfill consisting of granular material should be mechanically compacted in lifts. Lift thickness should not exceed those values given in the "Placement and Compaction of Fills" section of this report. Moisture conditioning (wetting) 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.

4.4 FOUNDATIONS

4.4.1 Foundation Type

As discussed previously, the proposed structures can be supported on conventional spread footings founded in the properly compacted fill.

4.4.2 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 2,500 pounds per square foot (psf) may be used for both continuous footings and isolated column footings for the proposed building. 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 trash enclosures, we recommend a maximum allowable bearing capacity of 1,500 pounds per square foot be used with minimum footing widths and depths of 18 inches.

4.4.3 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)
2,500	24	24
2,000	24	18
1,500	18	18

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

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

4.4.4 Estimated Settlements

Total static settlement of continuous wall footings (up to 2 kips per lineal foot) is expected to be on the order of ½ to ¾-inch. Differential static settlement between similarly loaded column footings or along a 60-foot span of a continuous footing is expected to be on the order of ½-inch or less. The majority of the settlement will occur immediately upon load application.

The potential for seismic settlement was addressed in a previous section of this report and should be referred to in evaluating the potential total settlements.

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

4.4.5 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 0.35 may be used for design. In addition, an allowable lateral bearing pressure equal to an equivalent fluid weight of 300 pounds per cubic foot may be used, provided the footings are poured tight against compacted fill. These values may be used in combination without reduction.

4.4.6 Foundation Inspection

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

4.5 BUILDING FLOOR SLABS

Slab-on-grade floors should be supported on granular, non-expansive (EI $\leq$ 20), compacted soils as discussed in the "Placement and Compaction of Fills" section. There is not a geotechnical requirement for slab reinforcing based on the non-expansive characteristics of the on-site soils.

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. The need for a sand layer with the vapor barrier is not a geotechnical issue and is a decision for the Project Architect.

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 pipe penetrations), and 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.

4.6 RETAINING STRUCTURES

Based on information available to us at this time, retaining walls are not planned at the site, however we have included the following recommendations for walls or shoring less than 6 feet in height. We recommend that walls be backfilled with granular soils (less than 40 percent passing the No. 200 sieve), which are readily available on site.

Active earth pressures can be used for designing cantilevered walls or shoring that can yield laterally at least ½-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 35 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 60 pounds per cubic foot can be used.

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.

4.7 PAVEMENTS

A test on the near-surface soils resulted in an R-value of 56. To account for variability of the on-site soils, an R-value of 40 was used for the preliminary design. Based on the subgrade soils anticipated, we recommend the following pavement sections for the various levels of traffic (traffic indices) anticipated:

ASPHALT CONCRETE PAVEMENT

PAVEMENT AREA	TRAFFIC INDEX	SECTION THICKNESS (inches)	
		ASPHALT CONCRETE	AGGREGATE BASE COURSE
Auto Parking/Drives	4/5	3	4

PORTLAND CEMENT CONCRETE PAVEMENT

PAVEMENT AREA	TRAFFIC INDEX	SECTION THICKNESS (inches)	
		f'c = 3,500 psi PCC	f'c = 4,000 psi PCC
Auto Parking/Drives	4/5	5.5	5.0
Truck Areas	6	6.0	5.5
	7	6.5	6.0
	8	6.5	6.5

The Project Civil Engineer should select the appropriate traffic index for the pavement based on the anticipated traffic usage. For design purposes, the following traffic indices correspond to the following number of heavy (five axle) truck trips per day for a 20-year design life:

Traffic Index	Heavy Truck Trips/Day
4	0
5	1
6	3
7	11
8	35

The concrete used for paving should have a compressive strength at least equivalent to the design compressive strength at the time pavement is subjected to traffic. We do not recommend using concrete with a compressive strength of less than 3,500 psi. Based on the soils encountered in our explorations, reinforcing of the concrete pavements is not required from a geotechnical standpoint. Joint patterns and details should be determined by the Project Civil Engineer. Aggregate base is not considered to be required beneath portland cement concrete.

The pavement subgrade and aggregate 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 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.

4.8 CORROSION

Laboratory testing performed by others (CHJ, 2016) for the adjacent site development indicates that the near surface soils exhibit a soluble sulfate content of 241 mg/kg. For the 2022 CBC, foundation concrete should conform to the requirements outlined in ACI 318, Section 4.3 for Category S0 levels of soluble sulfate exposure from the on-site soils. Chloride levels in the on-site soils are found to be 246 mg/kg. For concrete exposed to soil moisture, such as footings and floor slabs, we recommend a chloride Category C1.

Resistivity testing indicates that they are severely corrosive to buried ferrous metals. Soil corrosion with regards to foundation concrete was addressed in a prior section of this report. GPI does not practice corrosion protection engineering. If corrosion protection recommendations are required, a corrosion engineer such as HDR should be consulted to provide recommendations to protect these elements from corrosion.

4.9 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.

4.10 INFILTRATION TESTING

Test wells P-1 and P-2 were installed in boreholes drilled using truck-mounted hollow-stem auger drill equipment at preliminary infiltration basin locations provided by the Project Civil Engineer. The locations of the test wells are shown on Figure 2. The wells consisted of 2-inch diameter PVC casing installed in an 8-inch diameter borehole. The casing was perforated in the lower 2 feet of the wells. Packing material around the slotted sections of the well casing consisted of #3 sand. The test wells were constructed to depths of approximately 10 to 12 feet below existing grade in order to test the soils near the bottom of the proposed infiltration basin being considered at the time our field work was conducted. The infiltration testing was performed in general accordance with the San Bernardino County guidelines for borehole infiltration tests.

The measured infiltration rates were calculated using the drop in water level over the test increment time. The final measured rates for each well, corrected as indicated above, are presented in the following table and should be used with an appropriate factor of safety.

Infiltration Test Results Summary

TEST WELL	APPROXIMATE DEPTH OF TEST WELL (feet)	CORRECTED INFILTRATION RATE (in./hr.)
P-1	10	3.0
P-2	12	1.9

The Civil Engineer should evaluate the feasibility of surface infiltration using the rates provided above. Additional factors of safety in computing the design infiltration rate of the proposed infiltration BMP should be determined by the project Civil Engineer.

It should also be noted that the infiltration rates are for clean, clear water and do not include effects of sediment, fines, dissolved solids or other debris, as these materials will significantly reduce the infiltration rates of the subsurface soils. Prior to infiltration, water should be cleaned of sediment or other deleterious materials to help reduce the potential for clogging and reduced percolation rates. Should fines or suspended solids be permitted to enter the basin, reduced infiltration rates will result.

4.11 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. Sufficient in-place field density tests should be performed during fill placement and in-place compaction to evaluate the overall compaction of the soils. Soils that do not meet minimum compaction requirements should be reworked and tested prior to placement of additional fill. 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

This report, exploration logs, and other materials resulting from GPI's efforts were prepared exclusively for BRE Space Paxbello 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.

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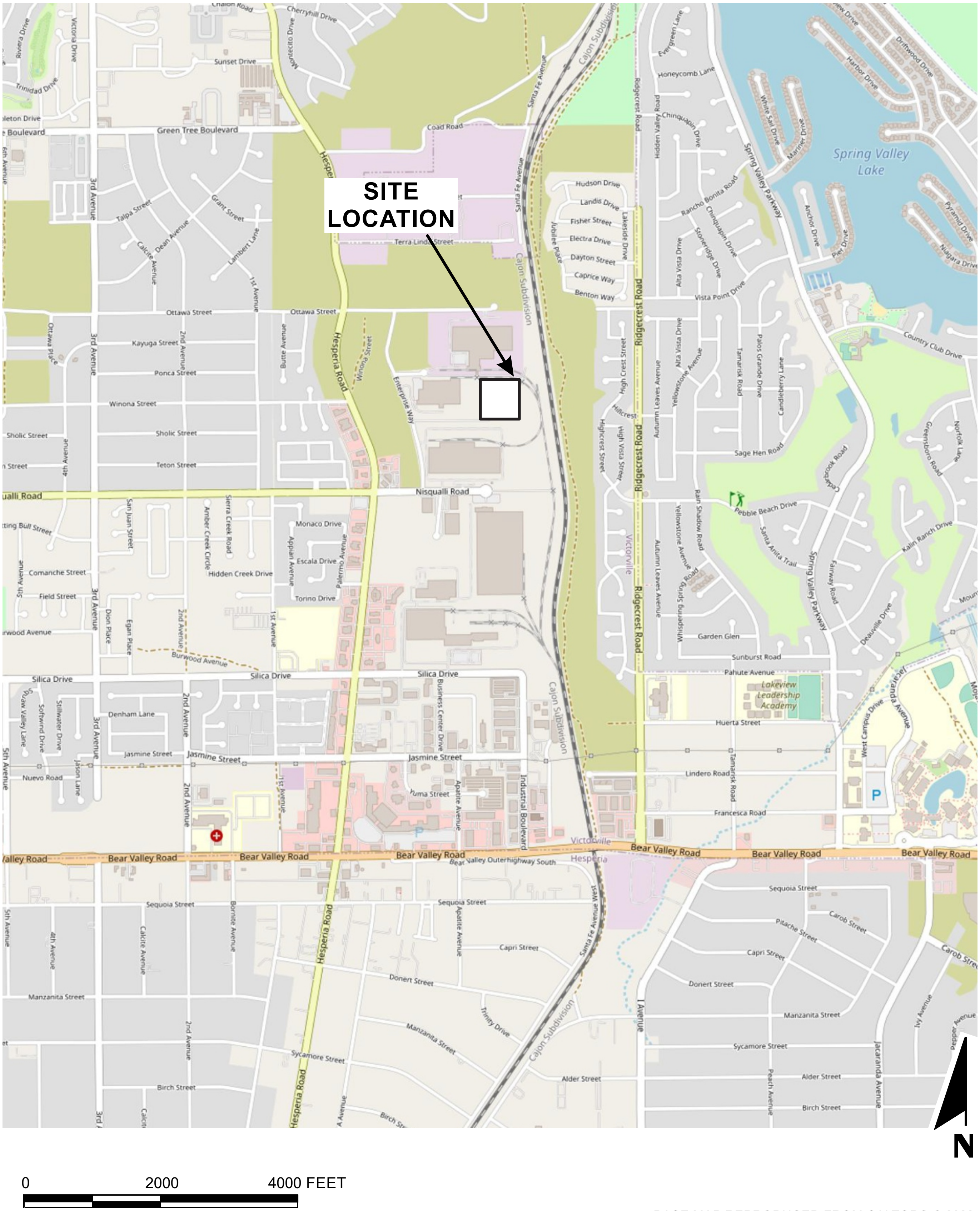


BASE MAP REPRODUCED FROM CALTOPO © 2022

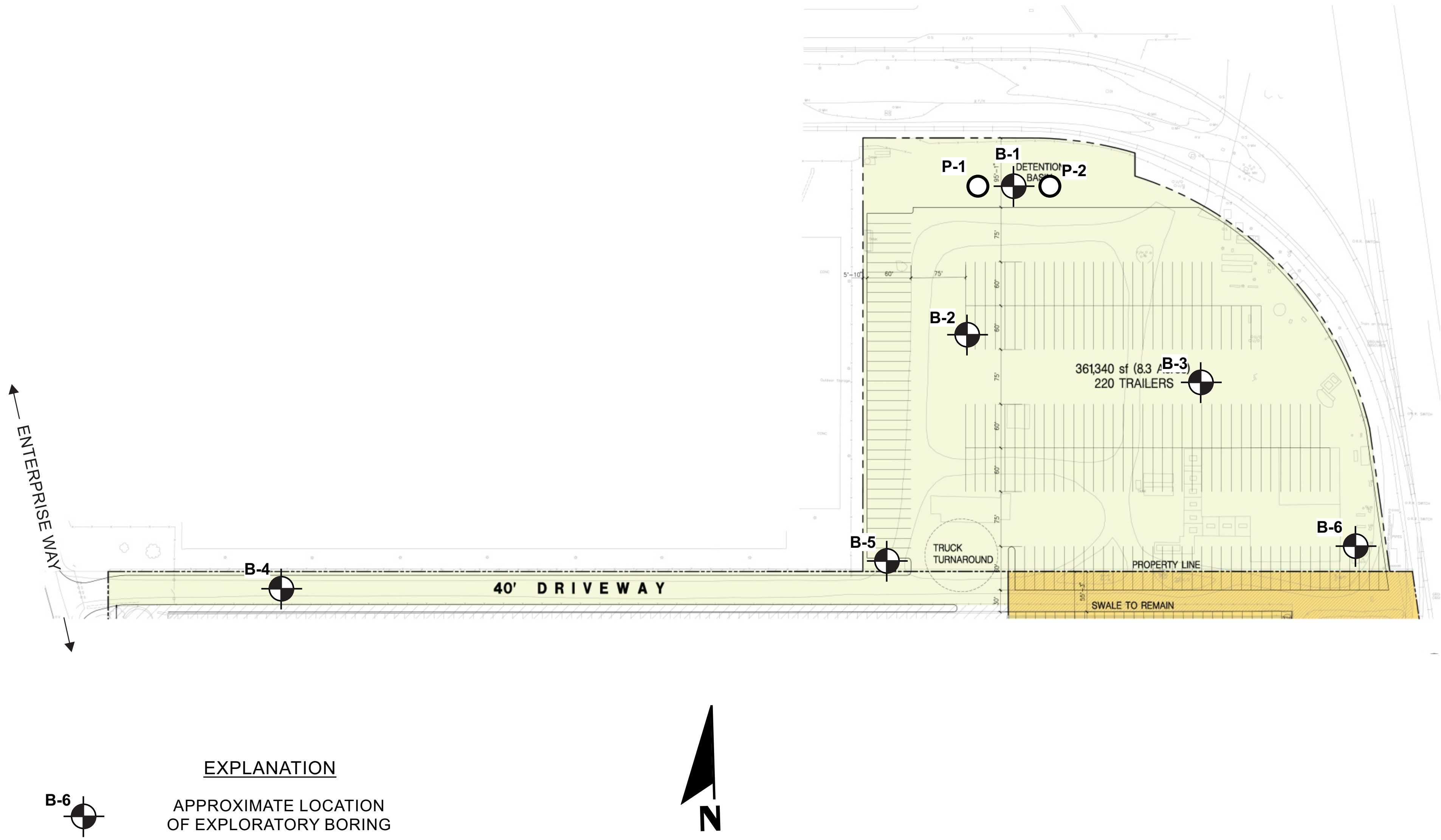
GEOTECHNICAL PROFESSIONALS, INC.

BRE SPACE VICTORVILLE

GPI PROJECT NO.: 3149.I	SCALE: 1" = 2000'

SITE LOCATION MAP

FIGURE 1



EXPLANATION

B-6 — APPROXIMATE LOCATION OF EXPLORATORY BORING

P-2 — APPROXIMATE LOCATION OF INFILTRATION TEST

0 200 400 FEET

BASE MAP REPRODUCED FROM OVERALL SITE PLAN PROVIDED BY HPA ARCHITECTURE : DATED 08-16-2021

GPI GEOTECHNICAL PROFESSIONALS, INC.

BRE SPACE - TRAILER LOT EXPANSION

GPI PROJECT NO.: 3149.I	SCALE: 1" = 200'

SITE PLAN

FIGURE 2

APPENDIX A

EXPLORATORY BORINGS

The subsurface conditions for the site were investigated by drilling and sampling 6 exploratory borings. The borings were advanced to depths of 4 to 26 feet below the existing ground surface. The approximate locations of the explorations are shown on the Site Plan, Figure 2.

The exploratory borings were drilled using truck-mounted hollow-stem auger drill equipment. Relatively undisturbed samples were obtained using a brass-ring lined sampler (ASTM D3550). 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 the “free-fall” hammer described above. 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 blow counts.

The field explorations for the investigation were performed under the continuous technical supervision of GPI's representative, who visually inspected the site, maintained detailed logs of the borings, classified the soils encountered, and obtained relatively undisturbed samples for examination and laboratory testing. The soils encountered in the borings were classified in the field and through further examination in the laboratory in accordance with the Unified Soils Classification System. Detailed logs of the borings are presented in Figures A-1 through A-6 in this appendix. Upon completion of the sampling of hollow-stem auger borings, the holes were backfilled with the excavated soils.

The boring locations were laid out in the field by measuring from existing site features. Ground surface elevations at the exploration locations were estimated from the ALTA Land Title Survey by David Evans and Associates dated December 13, 2022.

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					0	Fill: **GRAVELY SAND (GP)** brown, moist, trace silt	2900
	8.2	112	67	D		Natural: **GRAVELY SAND (GP)** brown, moist, dense, trace silt	
	7.1	116	50	D	5		2895
	3.6	113	74	D		@ 7 feet, slightly moist	
	2.9	117	80	D	10	**SANDY SILT (ML)** brown, dry, hard	2890
	7.3		33	S		**SILTY SAND (SM)** brown, slightly moist, dense, with gravel	
	6.2	114	72	D	15	**CLAYEY SAND (SC)** brown, moist, dense, with gravel @ 15 feet, trace cobbles	2885
	14.0		19	S		@ 17.5 feet, dark brown, very moist, medium dense	
	3.4	113	50/3"	D	20	**SILTY SAND (SM)** brown, dry to slightly moist, very dense, with gravel	2880
	3.8		49	S		**GRAVELY SAND (GP)** brown, slightly moist, dense, with silt	
	10.6	102	60	D	25	**SILTY SAND (SM)** brown, moist, dense	2875
						Total Depth 26 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.

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

DATE DRILLED: 12-8-22
EQUIPMENT USED: 8 " HOLLOW STEM AUGER
GROUNDWATER LEVEL (ft): NOT ENCOUNTERED

GPI

PROJECT NO.: 3149.I
BRE VICTORVILLE

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* 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)** brown, slightly moist, with gravel	
4.6	111	37	D		Natural: **SILTY SAND (SM)** brown, moist, medium dense	2900
2.9	117	77	D	5	@ 5 feet, dry	
					Total depth 6 feet	

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

DATE DRILLED:
12-8-22

EQUIPMENT USED:
8 " HOLLOW STEM AUGER

GROUNDWATER LEVEL (ft):
NOT ENCOUNTERED

GPI

PROJECT NO.: 3149.I

BRE VICTORVILLE

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.	
					0	Fill: **SILTY SAND (SM)** brown, moist, with gravel	2910
	3.2	114	47	D		Natural: **SILTY SAND (SM)** brown, dry to slightly moist, dense	
	6.2	106	65	D	5	@ 5 feet, slightly moist	2905
						Total Depth 6 feet	

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

DATE DRILLED:
12-8-22

EQUIPMENT USED:
8 " HOLLOW STEM AUGER

GROUNDWATER LEVEL (ft):
NOT ENCOUNTERED

GPI

PROJECT NO.: 3149.I

BRE VICTORVILLE

LOG OF BORING NO. B-3

FIGURE A-3

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	*DESCRIPTION OF SUBSURFACE MATERIALS* 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: **SAND (SP)** brown, slightly moist, with gravel	
	3.6	99	27	D		@ 2 feet, medium dense	
							2910
	3.0	99	39	D	5	Natural: **SILTY SAND (SM)** brown, dry to slightly moist, medium dense, with gravel	
						Total Depth 6 feet	

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

DATE DRILLED:
12-8-22

EQUIPMENT USED:
8 " HOLLOW STEM AUGER

GROUNDWATER LEVEL (ft):
NOT ENCOUNTERED

GPI

PROJECT NO.: 3149.I

BRE VICTORVILLE

LOG OF BORING NO. B-4

FIGURE A-4

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	*DESCRIPTION OF SUBSURFACE MATERIALS* 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)** brown, moist, with gravel	
7.9	118	70	D		Natural: **SILTY SAND (SM)** brown, moist, dense	
						2900
2.8	108	50/5"	D	5	@ 5 feet, dry, very dense	
4.4	117	50/6"	D		@ 7 feet, dry to slightly moist	
						2895
6.9	116	79	D	10	@ 10 feet, slightly moist	
						2890
6.4	96	62	D	15	**CLAYEY SAND (SC)** brown, slightly moist, dense, with gravel	
						2885
5.8	102	30	D	20	**SILTY SAND (SM)** brown, slightly moist, medium dense	
					CLAYEY SAND (SC) brown, slightly moist, medium dense, with gravel	
					Total Depth 21 feet	

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

DATE DRILLED:
12-8-22
EQUIPMENT USED:
8 " HOLLOW STEM AUGER
GROUNDWATER LEVEL (ft):
NOT ENCOUNTERED

PROJECT NO.: 3149.I
BRE VICTORVILLE

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 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: **SILTY SAND (SM)** brown, moist, with gravel	2910
						Total Depth 4 feet	

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

DATE DRILLED:
12-8-22
EQUIPMENT USED:
8 " HOLLOW STEM AUGER
GROUNDWATER LEVEL (ft):
NOT ENCOUNTERED

GPI

PROJECT NO.: 3149.I
BRE VICTORVILLE

LOG OF BORING NO. B-6

FIGURE A-6

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 from the borings. The samples were first trimmed to obtain volume and wet weight and then were 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.

PERCENTAGE PASSING NO. 200 SIEVE

Select 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. For select samples, the retained material was then run through a standard set of sieves in accordance with ASTM D6913 to classify the coarse fraction of representative sample. A summary of the percentages passing the No. 200 sieve is presented below. The grain size distribution data obtained from the full sieve analyses are presented in Figure B-1.

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	PERCENT PASSING No. 200 SIEVE
B-1	10	Sandy Silt (ML)	59
B-1	15	Clayey Sand (SC)	14
B-3	0-5	Silty Sand (SM)	17

COMPACTION TEST

Maximum dry density/optimum moisture tests were performed in accordance with ASTM D1557 on select representative bulk samples of the site soils. The samples were first screened through the No. 4 sieve and the sample retained was weighed to determine the material retained on the No. 4 sieve. The amount retained was used to determine the rock corrected maximum dry density in accordance with ASTM D 1557 specifications. The test results for the screened (passing No. 4 sieve) and rock-corrected sample are as follows:

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)
B-10	0-5	Silty Sand (SM)	132	8.0
		Silty Sand (SM) with rock correction	135	8.0

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 (GLA) under subcontract to GPI. The result of the test is as follows:

TEST PIT NO.	DEPTH (ft)	SOIL DESCRIPTION	R-VALUE BY EXUDATION
B-3	0 – 5	Silty Sand (SM)	56