Appendix E
Geotechnical Investigation

GEOTECHNICAL INVESTIGATION
PROPOSED INDUSTRIAL FACILITIES
2720 S. WILLOW AVENUE
RIALTO, CALIFORNIA

Prepared for:
Scannell Properties
jay.tanjuan@scannellproperties.com

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

Project No. 3118.I

June 17, 2022

GEOTECHNICAL PROFESSIONALS INC.

June 17, 2022

Scannell Properties
Jay Tanjuan, Director of Development
jay.tanjuan@scannellproperties.com

Subject: Report of Geotechnical Investigation
Proposed Industrial Facilities
2720 S. Willow Avenue
Rialto, California
GPI Project No. 3118.I

Dear Jay:

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
patrickm@gpi-ca.com

Paul R. Schade, G.E.
Principal
pauls@gpi-ca.com

Distribution: Addressee (PDF)
Adam Frankenberg, Scannell Properties
Rafik Gerges, S.E., HSA Associates
Brian Thienes, P.E., Thienes Engineering, Inc.

TABLE OF CONTENTS

LIST OF FIGURES

1.0 INTRODUCTION

1.1 GENERAL

This report presents the results of a geotechnical investigation performed by Geotechnical Professionals Inc. (GPI) for the proposed industrial facility at the subject site in Rialto, California. The site location is shown on the Site Location Map, Figure 1.

1.2 PROJECT DESCRIPTION

The proposed project will consist of a one-story industrial/distribution building about 120,210 square feet in plan. The building will be of concrete tilt-up construction. The floor slab will be supported on-grade with dock-high construction. The project will also include pavements, retaining walls, site walls, storm water infiltration systems, and landscaping on the remainder of the 5.6-acre site. The Project Civil Engineer, Thienes Engineering, provided the planned infiltration locations and indicated that infiltration depths will be about 12 feet below the existing grades.

Proposed finish floor elevations and structural loads were not finalized at the time this report was prepared. We have assumed proposed grades for the building will be up to 4 feet above existing grades for the dock-high portion of the building, the proposed truck dock will be cut up to 4 feet below existing grades, and the remainder of the parking and landscape areas similar to existing grades with cuts and fills up to 2 feet. The western end of the site currently consists of a 10- to 15-foot-high descending slope down to the site to the west. We understand that it is planned to construct a retaining wall at the toe of the slope and use the wall backfill to extend the level portion of the site to the west.

Based on similar past projects, we assume that maximum column and wall loads will be on the order of 80 kips and 8 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 subsurface exploration, field infiltration testing, laboratory testing, engineering analysis and the preparation of this report.

Our subsurface exploration for this site consisted of four hollow stem auger borings, three exploratory test pits, and two infiltration test wells. The exploratory borings were drilled to depths of 16 to 51 feet below existing grades, exploratory test pits were excavated to depths of 10 feet below existing grades, and the infiltration wells were installed at depths of about 12 feet below existing grades. A description of field procedures and logs of the explorations are presented in the attached Appendices A and B. The procedures and results of the infiltration tests are discussed in this report. The approximate locations of the subsurface explorations are shown on the Existing Site Plan and Proposed Site Plan, 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, hydro-consolidation, expansion, corrosivity, R-value, and maximum density/optimum moisture content. 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.

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 contains two one-story industrial buildings about 24,500 and 18,500 square feet with surface parking surrounding them in the western half of the site. The eastern half of the site contains a vacant lot with short grasses on the surface and an asphalt road leading to the industrial buildings along the northern portion of the site. Along the westernmost portion of the site is a terraced slope from the existing building and parking at about Elevation +990 down towards the adjacent property to the west to an elevation of about +975 at about a 4:1 (horizonal: vertical) slope. The ground surface surrounding the existing industrial buildings on the west side of the site is predominately asphalt concrete with limited amounts of portland concrete cement.

The site is bounded on the north and south by single-story industrial facilities with at grade parking, to the east by South Willow Avenue, and to the west by a vacant lot adjacent to Lilac Avenue. Based on historic aerial photographs (Historic Aerials), the site was occupied by a citrus tree grove up until around 1980, then was developed to near current conditions by1994.

In general, the overall site is relatively flat and slopes gently downward from west to east, with a change in ground surface elevation from about Elevation +990 feet to +988 feet across the site.

3.2 SUBSURFACE SOIL AND GROUNDWATER CONDITIONS

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

The depths of undocumented fill/disturbed soils were variable across the site. We encountered undocumented fills to approximately 3 to 6 feet below existing grade in the explorations. In general, the fill materials encountered consisted of loose to medium dense, dry to slightly moist silty sands and dry to slightly moist sandy silts.

Based on our observations of the surficial soils near the surface and at the base of the terraced slope at the westernmost portion of the property, the slope is predominately comprised of soils consistent with those encountered in our explorations.

The natural soils consist predominately of sands with varying amounts of silt and gravel. We encountered trace amounts of cobbles (up to 12 inches in diameter) in the explorations at depths as shallow as 7 feet below existing grade. The natural soils have moderate to high strength and low compressibility characteristics. Hydro-consolidation testing indicates that the upper natural soils have a low potential for collapse (loss of volume when saturated). Expansion testing of the upper natural soils indicate they have a low potential for expansion (EI = 0).

The moisture content was relatively uniform and predominately dry to slightly moist within the upper 10 feet, with an average of 6 percent (about 3 percent below optimum).

Groundwater was not encountered in our current explorations to a maximum depth of 50 feet below ground surface. Published data by the California Department of Water Resources indicates historical high groundwater deeper than 50 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/disturbed soils were reported to depths of up to 6 feet below existing grade. The fill soils are not considered to be suitable for direct support of foundations or floor slabs without remedial earthwork. In addition, the upper natural soils have relatively low densities. 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.

- Current moisture contents of the upper soils are generally below the optimum moisture content; therefore, moisture conditioning (wetting) will be required.

- The upper on-site soils are predominantly dry to slightly moist, loose to 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 2019 California Building Code (CBC) criteria. For the 2019 CBC, a Site Class D may be used. Using the Site Class, which is dependent on geotechnical issues, and the appropriate seismic design maps, the corresponding seismic design parameters from the CBC are as follows:

<u>2019 CBC:</u>

$S_S = 1.66g$	$S_{MS} = F_a * S_S = 1.66g$	$S_{DS} = 2/3 * S_{MS} = 1.11g$
$S_1 = 0.65g$	$S_{M1} = F_V * S_1 = 1.11g$	$S_{D1} = 2/3 * S_{M1} = 0.74g$

In accordance with the 2019 CBC, site-specific response spectra are required for structures located in a Site Class D (with S1 greater than or equal to 0.2) unless, per the exceptions detailed in Section 11.4 8 of ASCE 7-16, the structure is designed using seismic response coefficient (Cs) determined by either:

- Equation 12.8-2 for values of T ≤ 1.5 TS,
- 1.5 times the value computed by Equation 12.8-3 for values of TL ≥ T > 1.5 TS, or
- 1.5 times the value computed by Equation 12.8-4 for values of T > TL.

If this exception is not taken and the structure will still be designed in accordance with the 2019 CBC, GPI should be notified that site-specific response spectra is requested.

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.

4.2.2 Strong Ground Motion Potential

Based on published information (geohazards.usgs.gov), the most significant fault in the proximity of the site is the San Jacinto (San Bernardino) fault, which is located about 4.5 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 0.77g for a mean magnitude 7.2 earthquake. This acceleration has been computed using the mapped Maximum Considered Geometric Mean peak ground acceleration from the ASCE 7-16 (for 2019 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.

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

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

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 (San Bernardino 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 1 inch. The differential seismic settlement is estimated to be less than ½-inch across a span of 60 feet (angular distortion of 1/1440). The differential settlement across other spans can be extrapolated from the information provided.

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 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. Inert demolition debris, such as concrete and asphalt, may be crushed for reuse in engineered fills in accordance with the criteria presented in the “Materials for Fill” section of this report.

Although not encountered in our explorations, cesspools or septic systems are likely to be encountered during grading due to the existing structures 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.

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 and Minor Structures

To provide uniform support for the planned building and retaining wall along the western property line, 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 6 feet below existing grades and at least 3 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 3 feet below existing grade or 1 foot below the base of the foundation, whichever is deeper. For perimeter site walls, removals should extend at least 3 feet below the existing grade or to the level of the bottom of the planned 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.

<u>Slot Cuts</u>

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 4 feet and 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. Moistening of the dry sandy soils anticipated at the site can usually be accomplished by deep ripping and liberal watering prior to compaction.

4.3.4 Material for Fill

The on-site soils are suitable for use as compacted fill. Imported fill material should be predominately granular (contain between 10 and 40 percent fines-portion passing No. 200 sieve), and relatively non-expansive (an Expansion Index of less than 20). GPI should be provided with a sample (at least 50 pounds) and notified at least 72 hours in advance of the location of soils proposed for import. 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. On-site materials greater than 8 inches in diameter can be exported, crushed, or disposed of in windrows outside of the building pad. If windrows are used, the oversized particles should be placed so that voids around the particles can be filled with sandy soils, which should be jetted or flooded after placement. At least 3 feet of properly compacted fill without oversized materials should cover the windrowed materials. Windrows should be located at least 4 feet below the finished grade and outside of planned building footprints, utility alignments, or other proposed below grade improvements.

If on-site concrete or asphalt are crushed/pulverized to re-use as aggregate base or general fill, it should be crushed to a maximum particle size of 3 inches. If used as general fill, the materials may need to be mixed with on-site soils before placement if significant voids are present in the crushed materials. If used to support pavements, it should be crushed to meet the specifications of Caltrans Class II or Greenbook crushed miscellaneous base (CMB).

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. Soils within 1-foot of the subgrade for building floor slabs and pavement areas, and the aggregate 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.

Fills should be placed at moisture contents of 0 to 2 percent over the optimum moisture content in order to readily achieve the required compaction. Current moisture contents of the upper soils are generally well below the optimum moisture content so that 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, additional moisture conditioning and processing will be required.

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 17 to 22 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.

4.3.8 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. Soils that do not meet minimum compaction requirements should be reworked and tested prior to placement of additional fill.

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 3,000 pounds per square foot (psf) may be used for both continuous footings and isolated column footings for the building. These bearing pressures are for dead-plus-live-loads, and may be increased one-half 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 and lateral limits of the overexcavation may be limited, we recommend a maximum allowable bearing capacity of 1,500 pounds per square foot be used.

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)
3,000	36	24
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. If interior footings are not fully loaded before the slab is in-place, the depth of interior footings may be taken from the top of the floor slab.

A minimum footing width and 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 isolated pad or continuous wall footings (up to 100 kips for columns and 8 kips per lineal foot for walls) is expected to be on the order of ½-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 ¼-inch or less (angular distortion of 1/1440). The differential settlement across other spans can be extrapolated from the information provided. 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 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 0.33 may be used for design. In addition, an allowable lateral bearing pressure equal to an equivalent fluid weight of 315 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.

4.4.7 Foundation Concrete

Laboratory testing by HDR (Appendix C) on a selected sample indicates that the near surface soils exhibit a soluble sulfate content of 38 mg/kg. For the 2019 CBC, foundation concrete should conform to the requirements outlined in ACI 318, Section 4.3 for negligible levels of soluble sulfate exposure from the on-site soils, (Category S0). Chloride levels in the sample of the upper soils tested were found to be 9.1 mg/kg, which is considered to be low (Category C1).

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. Based on our explorations, granular, non-expansive soils are readily available on-site. We do not anticipate swell pressures to negatively impact the building floor slab based on the non-expansive characteristics of the on-site soils. There is not a geotechnical requirement for slab thickness or reinforcing based on the non-expansive characteristics of the on-site soils.

For elastic design of slabs-on-grade supporting sustained concentrated loads, a modulus of subgrade reaction (k) of 250 pounds per cubic inch (pounds per square inch per inch of deflection) and 150 pounds per cubic inch may be used for on-site soils for short and long term loads.

Although not anticipated under most of the building, 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 or 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

The following recommendations are provided for walls or shoring less than 15 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 38 pounds per cubic foot may be used. This value can also be used for design of temporary cantilevered shoring. If the walls are designed with sloping backfill (up to 2:1), a lateral pressure of an equivalent fluid weighing 57 pounds per cubic foot may be used.

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 65 pounds per cubic foot can be used. If the walls are designed with sloping backfill (up to 2:1), a lateral pressure of an equivalent fluid weighing 98 pounds per cubic foot may 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 25 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.

4.7 EXTERIOR CONCRETE FLATWORK (PEDESTRIAN HARDSCAPE)

Exterior concrete pads and pedestrian hardscape should be supported on non-expansive, compacted fill, which appears readily available onsite. This includes exterior sidewalks, stamped concrete, non-traffic pavement, and pavers. Prior to placement of concrete, the subgrade should be prepared as recommended in the "Subgrade Preparation" section of this report.

4.8 PAVEMENTS

A test on the near-surface soils resulted in an R-value of 61. To account for variability of the on-site soils, an R-value of 50 was used for the preliminary design. We understand that asphalt concrete paving will only be considered for automobile-only pavement areas, and that pavement accessed by trucks will be portland cement concrete. 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 = 3,000 psi PCC	f’c = 3,500 psi PCC	f’c = 4,000 psi PCC
Auto Parking/Drives	4/5	6.0	5.5	5.0
Truck Areas	6	6.0	6.0	5.5
	7	6.5	6.5	6.0
	8	7.0	6.5	6.0

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

4.9 CORROSION

Resistivity testing of representative samples of the on-site soils indicates that they are moderately 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.

4.10 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.

4.11 INFILTRATION TESTING

Test wells P-1and P-2 were installed in boreholes drilled using truck-mounted hollow-stem auger drill equipment at preliminary infiltration basin locations provided by Thienes Engineering. The locations of the test wells are shown on Figures 2 and 3. 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 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 County guidelines (San Bernardino County) for borehole infiltration tests.

The measured infiltration rates were calculated using the drop in water level over the test increment time and corrected using the Porchet Method. 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	12	26
P-2	13	22

After completion of the infiltration testing, the well casings were removed, and the holes were backfilled with the on-site soils.

The subsurface conditions encountered in the explorations across the site appear to be relatively uniform and comparable infiltration rates are anticipated in other areas of the site.

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.

4.12 GEOTECHNICAL OBSERVATION AND TESTING

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

5.0 LIMITATIONS

This report, exploration logs, and other materials resulting from GPI's efforts were prepared exclusively for Scannell Properties 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 expressed or implied, is included or intended in our report.

Respectfully submitted,
Geotechnical Professionals Inc.

REGISTERED PROFESSIONAL ENGINEER
PATRICK I.F. MCGERVEY
No. C 90770
CIVIL
STATE OF CALIFORNIA

Patrick I.F. McGervey, P.E.
Project Engineer

REGISTERED PROFESSIONAL ENGINEER
PAUL R. SCHADE
No. 2371
GEOTECHNICAL
STATE OF CALIFORNIA

Paul R. Schade, G.E.
Principal

REFERENCES

American Society of Civil Engineers (ASCE) (2017), "Minimum Design Loads and Associated Criteria for Buildings and Other Structures," ASCE/SEI 7-16

California Department of Water Resources, Water Data Library, http://www.water.ca.gov/waterdatalibrary/

San Bernardino County Land Use Plan "General Plan – Geologic Hazard Overlays" sheet FH2-C, dated March 9, 2010

County of San Bernardino, Department of Public Works (2011), "Appendix VII – Infiltration Rate
Evaluation Protocol and Factor of Safety Recommendations," dated May 19, 2011

California Office of Statewide Health Planning and Development (OSHPD), Seismic Design Maps Website, https://seismicmaps.org/

United States Geologic Survey, Earthquake Ground Motion Parameters Application, http://earthquake.usgs.gov/research/hazmaps/design

United States Geological Survey (2014), 2008 National Seismic Hazard Maps, Source Parameters, https://earthquake.usgs.gov/cfusion/hazfaults_2008_search/query_main.cfm

BASE MAP REPRODUCED FROM CALTOPO © 2022

GPI GEOTECHNICAL PROFESSIONALS, INC.

SCANNELL - WILLOW

GPI PROJECT NO.: 3118.I	SCALE: 1" = 2000'

SITE LOCATION MAP

FIGURE 1

EXPLANATION

B-4 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING

TP-5 APPROXIMATE LOCATION AND NUMBER OF CONE PENETRATION TEST

P-4 APPROXIMATE LOCATION AND NUMBER OF INFILTRATION TEST

N

0 100 200 FEET

BASE MAP REPRODUCED FROM GOOGLE EARTH © 2022

GEOTECHNICAL PROFESSIONALS, INC.

SCANNELL - WILLOW

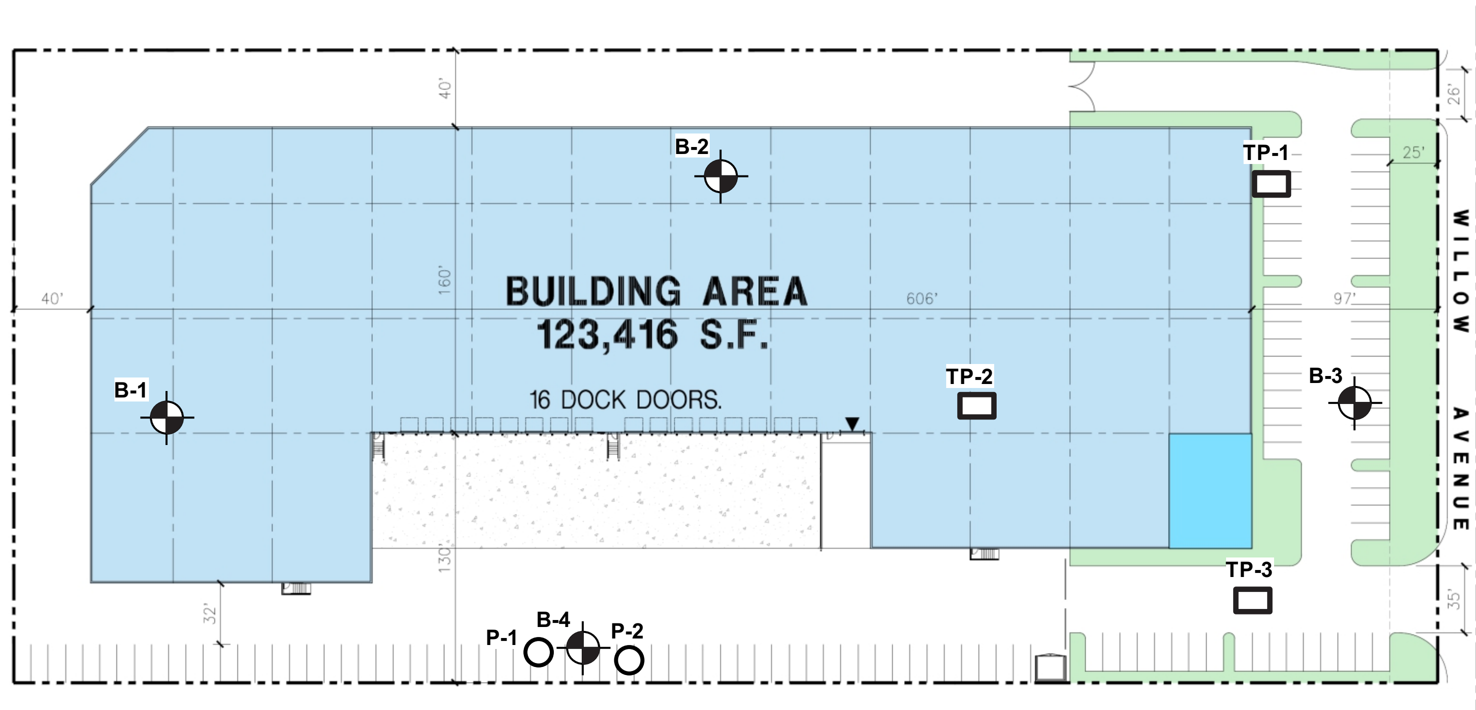
GPI PROJECT NO.: 3118.I

SCALE: 1” = 100’

SITE PLAN

(EXISTING)

FIGURE 2



EXPLANATION

B-4 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING

TP-3 APPROXIMATE LOCATION AND NUMBER OF CONE PENETRATION TEST

P-4 APPROXIMATE LOCATION AND NUMBER OF INFILTRATION TEST

N

0 100 200 FEET

BASE MAP REPRODUCED FROM CONCEPTRUAL SITE PLAN
PROVIDED BY HPA ARCHITECTURE: DATED 02-1-2022

GPI GEOTECHNICAL PROFESSIONALS, INC.

SCANNELL - WILLOW

GPI PROJECT NO.: 3118.I	SCALE: 1" = 100'

SITE PLAN

(PROPOSED)

FIGURE 3

APPENDIX A

APPENDIX A

EXPLORATORY BORINGS

The subsurface conditions at the site were investigated by drilling and sampling four exploratory borings. The borings were advanced to depths ranging from 16 to 51-feet below the existing ground surface. The locations of the explorations are shown on the Exploration Location Plans, Figures 2 and 3.

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 B-1 and B-4 in this appendix.

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 internet sources and should be considered very approximate.

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
			B	0	3-Inch AC over 4-Inch BASE	
					Fill/Disturbed: **SANDY SILT (ML)** light brown, dry, medium dense, trace gravel	
4.1	110	20	D			
2.8	98	16	D	5	Natural: **SILTY SAND (SM)** light brown, dry, medium dense	985
		30	D		@ 7 feet, no recovery, trace gravel, possible cobbles	
		52	D	10	@ 20 feet, no recovery	980
1.4		65	D	15	@ 15 feet, dense	975
4.1	103	33	D	20	@ 20 feet, medium dense	970
					Total Depth 21 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: 4-1-22

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 3118.I

SCANNELL WILLOW RIALTO

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)
				B	0	4-Inch AC over 4-Inch BASE	
						Fill/Disturbed: **SILTY SAND (SM)** light brown, moist, loose	990
	9.6	102	11	D			
	10.4	104	11	D	5		
	4.7		8	D		Natural: **SAND (SP)** brown, slightly moist, medium dense, trace gravel and cobbles	985
	2.5	93	20	D	10	@ 9 feet, dry	
							980
	2.5	114	29	D	15		
						Total Depth 16 feet	

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

DATE DRILLED:
4-1-22

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 3118.I

SCANNELL WILLOW RIALTO

LOG OF BORING NO. B-2

FIGURE A-2

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
					0	Fill/Disturbed: **SANDY SILT (ML)** light brown, dry	985
	3.6	92	16	D		@ 1 foot, medium dense	
	4.0	90	20	D			
	4.3	109	24	D	5	Natural: **SILTY SAND (SM)** light brown, dry, medium dense, trace gravel	980
	6.0	94	15	D		@ 7 feet, slightly moist	
	1.2		25	D	10	**SAND (SP)** light brown, dry, medium dense	975
	1.7		40	S	15		970
	2.2	92	71	D	20		965
	3.0		27	S	25	**SILTY SAND (SM)** brown, dry, medium dense	960
	3.1	111	83/10"	D	30	**SAND WITH SILT (SP-SM)** brown, dry, very dense, trace gravel	955
	1.8		39	S	35	@ 35 feet, trace cobbles	950

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

DATE DRILLED:
4-1-22

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 3118.I

SCANNELL WILLOW RIALTO

LOG OF BORING NO. B-3

FIGURE A-3

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
						This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
	1.8	66	84	D	40	**SAND WITH SILT (SP-SM)** brown, dry, very dense, trace gravel	945
	3.4		67	S	45	**SILTY SAND (SM)** brown, dry, very dense	940
	1.8		90/11"	D	50	**SAND WITH SILT (SP-SM)** brown, dry, very dense, trace gravel and cobbles	935
						Total Depth 51 feet	

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

DATE DRILLED: 4-1-22

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 3118.I

SCANNELL WILLOW RIALTO

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)
			B	0	2.5-Inch AC over 4-Inch BASE	
9.2	110	32	D		Fill/Disturbed: **SILTY SAND (SM)** brown, slightly moist, medium dense @ 2 feet, moist	990
13.5	110	14	D	5	Natural: **SAND (SP)** brown, very moist, loose, trace gravel	
9.4	109	9	D			985
3.1	108	22	D	10	@ 10 feet, dry, medium dense	
						980
3.4		38	S	15	**SAND WITH SILT (SP-SM)** brown, dry, medium dense	
		50/5"	S			975
2.7					@ 17.5 feet, dense, trace cobbles	
3.7		23	S	20	**SAND (SP)** light brown, slightly moist, medium dense, trace gravel	
						970
2.0	114	51	D	25	**SAND WITH SILT (SP-SM)** brown, dry, dense, trace gravel and cobbles	
					Total depth 26 feet	

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

DATE DRILLED:
4-1-22

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 3118.I

SCANNELL WILLOW RIALTO

LOG OF BORING NO. B-4

FIGURE A-4

APPENDIX B

APPENDIX B

EXPLORATORY TEST PITS

The subsurface conditions at the site were investigated by excavating and sampling three exploratory test pits. The excavations were advanced to depths of approximately 10-feet below the existing ground surface using a backhoe with a 24-inch-wide bucket. The approximate location of the exploration is shown on the Exploration Location 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 pit, and classified the soils encountered. Bulk samples of the subsurface soils were obtained and bagged at select locations within the excavation. The soils encountered in the test pit were classified in the field and through further examination in the laboratory in accordance with the Unified Soils Classification System. Detailed logs of the exploration are presented in Figures B-1 to B-3 in this Appendix.

The test pit locations were laid out in the field by measuring from existing site features. Ground surface elevations at the exploration locations were estimated from internet sources and should be considered very approximate.

PROBING (INCHES)	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)
1					0	Fill: **SANDY SILT (ML)** light brown, dry to slightly moist	
12				B		Natural: **SANDY SILT (ML)** light brown, slightly moist to moist, porous	980
				B	5		
8				B		@ 9 feet, dark brown	
					10	Total Depth 10 feet	975

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

DATE DRILLED: 4-6-22
EQUIPMENT USED: 24Backhoe
GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 3118.I
SCANNELL WILLOW RIALTO

LOG OF TEST PIT NO. TP-1

FIGURE B-1

PROBING (INCHES)	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
2				B	0	Fill: **SANDY SILT (ML)** light brown, dry to slightly moist, trace gravel	985
18						Natural: **SANDY SILT (ML)** light brown, dry to slightly moist, trace gravel	
				B	5	@ 4.5 feet, brown, slightly moist to moist	
						@ 7 feet, trace clay	980
7				B	10	@ 9 feet, dark brown	

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

DATE DRILLED: 4-6-22
EQUIPMENT USED: 24Backhoe
GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 3118.I
SCANNELL WILLOW RIALTO

LOG OF TEST PIT NO. TP-2

FIGURE B-2

PROBING (INCHES)	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)
3				B	0	Fill: **SANDY SILT (ML)** light brown to brown, dry to slightly moist, trace gravel	980
				B	5	Natural: **SANDY SILT (ML)** light brown to brown, dry to slightly moist, trace gravel	
14						@ 7 feet, brown, slightly moist to moist	975
6				B		@ 9 feet, dark brown	
					10	Total Depth 10 feet	

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

DATE DRILLED:
4-6-22

EQUIPMENT USED:
24Backhoe

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 3118.I

SCANNELL WILLOW RIALTO

LOG OF TEST PIT NO. TP-3

FIGURE B-3

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. 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. Afterwards, the retained material was run through a standard set of sieves in accordance with ASTM D 422 for select samples. The percentages passing the No. 200 sieve are tabulated below. The grain size distribution obtained from the full sieve analysis is presented in Figure D-1.

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	PERCENT PASSING No. 200 SIEVE
B-1	5	Silty Sand (SM)	35
B-2	0-5	Silty Sand (SM)	49
B-2	6	Silty Sand (SM)	17
B-2	9	Sand (SP)	3
B-4	10	Sand (SP)	4
B-4	0-5	Silty Sand (SM)	46
B-4	15	Sand with Silt (SP-SM)	5

DIRECT SHEAR

Direct shear tests were performed on an undisturbed and a remolded bulk sample in accordance with ASTM D 3080. The bulk samples were remolded to approximately 90 percent of maximum density (ASTM D1557). The samples were placed in the shear machine, and a normal load comparable to the in-situ overburden stress was applied. The samples were inundated, allowed to consolidate, and then were sheared to failure. The tests were repeated on additional test specimens under increased normal loads. Shear stress and sample deformation were monitored throughout the test. The results of the direct shear tests are presented in Figures C-1 and C-2.

HYDROCONSOLIDATION

One-dimensional consolidation testing was performed on an undisturbed sample in accordance with ASTM D5333. After trimming the ends, the sample was placed in the consolidometer and loaded to 0.4ksf. Thereafter, the samples were incrementally loaded to a maximum load of 1.6 ksf at the in-situ moisture content and then saturated. Sample deformation was measured to 0.0001 inch. The amount of collapse is shown below as percent compression of the sample.

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	TOTAL COLLAPSE (%)
B-4	10	Sand (SP)	0.49

COMPACTION TEST

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

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)
B-4	0-5	Silty Sand (SM)	132	9

EXPANSION INDEX TEST

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

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	EXPANSION INDEX, EI	EXPANSION POTENTIAL
B-4	0-5	Silty Sand (SM)	0	Very Low

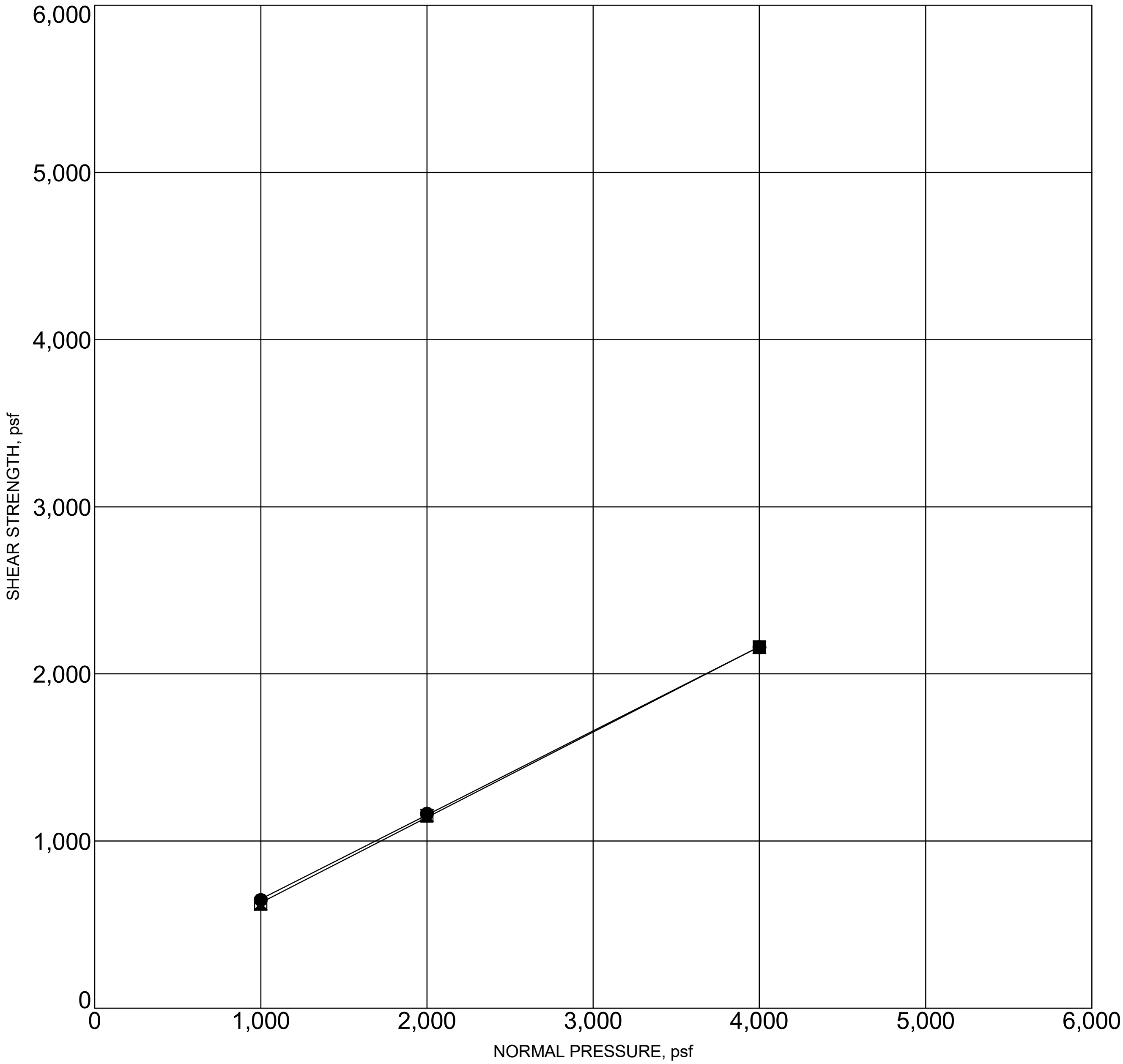
R-VALUE

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

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	R-VALUE BY EXPANSION
B-2	0-5	Silty Sand (SM)	61

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= 27 degrees
Cohesion= 150 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 27 degrees
Cohesion= 120 psf

Sample Location		Classification	DD,pcf	MC,%
B-1	5.0	SILTY SAND (SM)	98	2.8

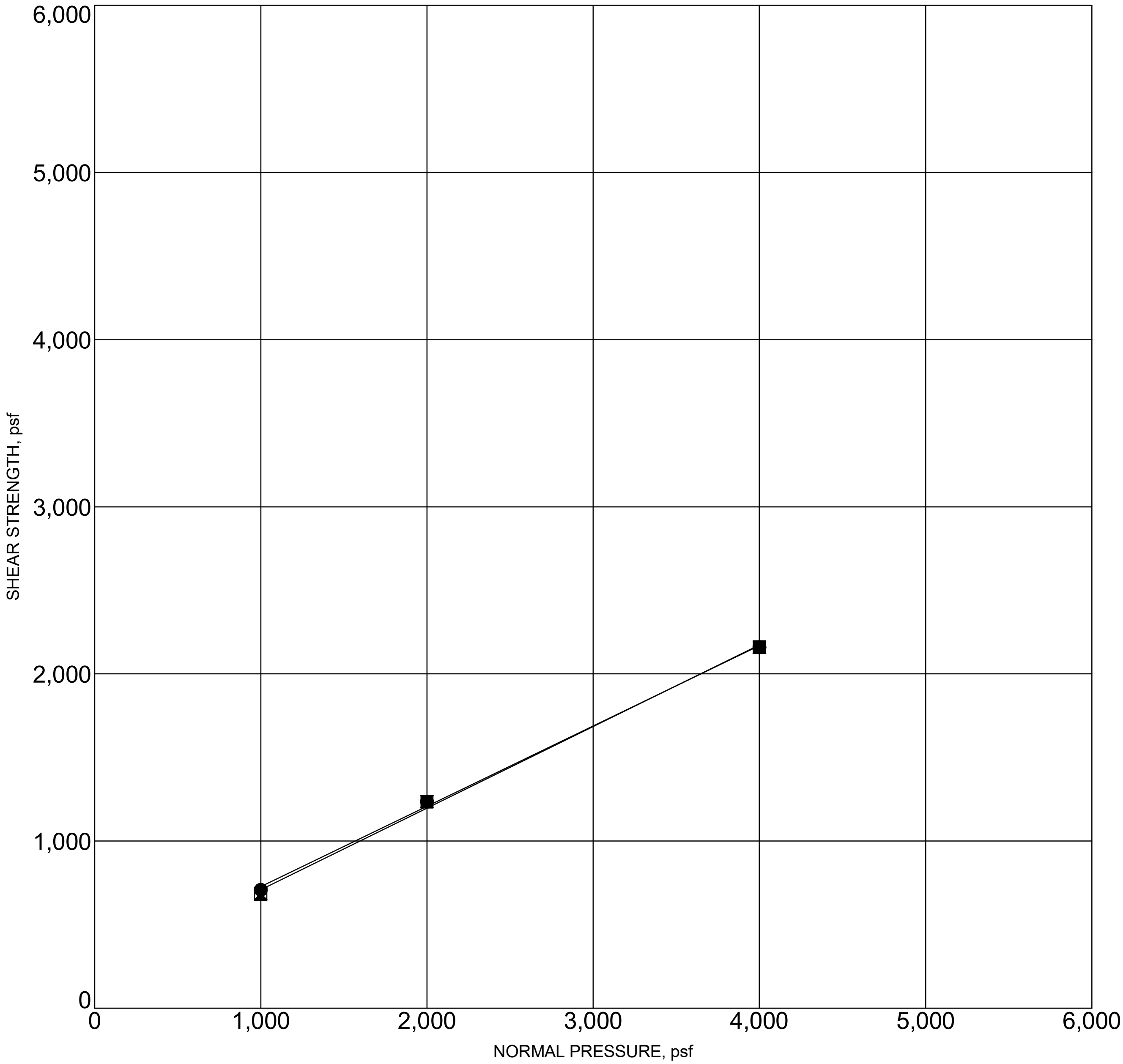
PROJECT: SCANNELL WILLOW RIALTO

PROJECT NO.: 3118.I

GPI

DIRECT SHEAR TEST RESULTS

FIGURE C-1



● **PEAK STRENGTH**
Friction Angle= 26 degrees
Cohesion= 246 psf

ULTIMATE STRENGTH
Friction Angle= 26 degrees
Cohesion= 222 psf

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

Sample Location		Classification	DD,pcf	MC,%
B-4	0-5	SILTY SAND (SM)	119	9.0

PROJECT: SCANNELL WILLOW RIALTO　　PROJECT NO.: 3118.I

GPI

DIRECT SHEAR TEST RESULTS

FIGURE C-2

HDR

Table 1 - Laboratory Tests on Soil Samples

Geotechnical Professionals, Inc.
Willow Rialto
Your #3118.I, HDR Lab #22-0405LAB
15-Apr-22

Sample ID			
			B-2 @ 0-5'
Resistivity		**Units**	
as-received		ohm-cm	31,200
saturated		ohm-cm	2,560
pH			7.5
Electrical Conductivity		mS/cm	0.09
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	75
magnesium	Mg^{2+}	mg/kg	ND
sodium	Na^{1+}	mg/kg	38
potassium	K^{1+}	mg/kg	ND
ammonium	NH_4^{1+}	mg/kg	ND
Anions			
carbonate	CO_3^{2-}	mg/kg	ND
bicarbonate	HCO_3^{1-}	mg/kg	162
fluoride	F^{1-}	mg/kg	4.7
chloride	Cl^{1-}	mg/kg	9.1
sulfate	SO_4^{2-}	mg/kg	38
nitrate	NO_3^{1-}	mg/kg	91
phosphate	PO_4^{3-}	mg/kg	3.2
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