GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE APARTMENT DEVELOPMENT
TAIX – SUNSET SITE
1911 TO 1931 W. SUNSET BOULEVARD
LOS ANGELES, CALIFORNIA

1911, 1915, 1921, 1925, 1927, 1929, and 1931 W. Sunset Boulevard
2000, 2008, 2010, 2016, and 2016 W. Reservoir Street
Tract: Lake Side Tract, Lot: 1 to 5A, Arb: 1 to 2

Prepared for:
Holland Acquisition Co., LLC
5000 E. Spring Street, Suite 500
Long Beach, CA 90815

Prepared by:
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Project No. 2914.I

February 1, 2019

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Holland Acquisition Co., LLC
5000 E. Spring Street, Suite 500
Long Beach, California 90815

Attention: Mr. Jacob Stone

Subject: Report of Geotechnical Investigation
Proposed Mixed-Use Apartment Development
Taix – Sunset Site
1911 to 1931 W. Sunset Boulevard
Los Angeles, California
GPI Project No. 2914.I

Dear Mr. Stone:

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

We are providing this report in an electronic format. Further copies of the report can be provided when required for City submittal.

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

Very truly yours,
Geotechnical Professionals Inc.

James E. Harris, 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 mixed-use apartment development located at 1911 to 1931 W. Sunset Boulevard in Los Angeles, California. The site location is shown on the Site Location Map, Figure 1.

1.2 PROJECT DESCRIPTION

The proposed project will consist of a 7-story building including 1 level of below grade parking. The upper 5 stories will be apartment buildings and the ground floor will include a lobby, retail space, and parking. The building covers a footprint of approximately 32,100 square feet (sf) at the lower level parking and a footprint of approximately 36,000 sf at the ground level. At ground level, a portion of the existing restaurant and a retail space will be located within the building site at ground level. Above the retail portion, the building extends 6 levels. The below-grade portion of the building extends to near the property lines on W. Reservoir Street, the adjacent property to the west, and a portion of W. Sunset Boulevard.

The approximate proposed site configurations at street level and subterranean level (P1) are shown on the Site Plans, Figures 2 and 3, respectively. A building section is presented on Figure 4.

Based on information provided by the Project Structural Engineer, either spread footings with a slab-on-grade or a mat foundation will be used to support the building. The structure will be constructed of two levels of a concrete podium underlying 5 levels of a wood frame structure. Detailed structural loads are not known at this time. Based on our experience with similar projects, we anticipate that the maximum column loads will be on the on the order of 200 to 300 kips. If a mat foundation is to be used, we anticipated a bearing pressure ranging from approximately 300 to 600 pounds per square foot (psf). We anticipate that the foundations will be approximately 13 to 16 feet below existing site grades, based on the finish floor depths provided on the project architects plans.

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 shoring and foundation plans become available, we should be provided with a copy 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.

2.0 SCOPE OF WORK

Our scope of work for this field investigation consisted of a review of published information, subsurface exploration, laboratory testing, geologic evaluations, engineering analyses, and preparation of this report.

Our field investigation consisted of three exploratory borings. The borings were drilled to depths of 41 to 71 feet below the existing grades. A description of field procedures and logs of borings are presented in Appendix A. The approximate locations of the subsurface explorations are shown on the Site Plans, Figures 2 and 3.

Laboratory soil tests were performed on selected representative samples as an aid in soil classification and to evaluate the engineering properties of the soils. The geotechnical laboratory testing program included determination of moisture content and dry density, direct shear, expansion potential, and soil corrosivity. Laboratory testing procedures and results are summarized in Appendix B.

HDR performed corrosivity testing on select soil samples provided by GPI under subcontract to GPI. Their test results and report are presented in Appendix B.

Engineering evaluations were performed to provide earthwork criteria, foundation, wall-below-grade, and slab design parameters, preliminary pavement sections, and assessment of seismic hazards. The results of our evaluations are presented in the remainder of the report.

3.0 GEOLOGY

3.1 GEOLOGIC SETTING/REGIONAL GEOLOGY

The project site is located in a geomorphic area termed the Los Angeles Basin, a generally low-lying coastal plain that stretches from the Santa Monica and San Gabriel Mountains northerly of the site, to the Pacific Ocean south and southeast of the site. The basin is underlain by a structural trough which has been filled with primarily Tertiary age marine sedimentary rocks, and locally by Pleistocene and Recent non-marine alluvial deposits. On a more regional basis, the site area is within the Peninsular Ranges Geomorphic Province, a province that stretches from the Los Angeles area to the tip of Baja California, characterized by northwest trending mountain ranges and elongate valleys, as well as northwest trending active faults, including the Newport-Inglewood fault southwest of the site.

Locally, the project site is located within the Elysian Park Hills, a series of low-moderate relief hills underlain by Late Tertiary marine sedimentary rocks assigned variously as the Puente formation by Lamar (1970), and others, and more recently an un-named shale by Dibblee (1991). The site area has been mapped by both Lamar and Dibblee within the south flowing drainage that continues to Echo Park Lake, and is shown as being underlain by recent alluvial deposits. Our borings on the site, as well as an adjacent site immediately to the south encountered bedrock at the ground surface. Either the site was graded at an earlier date and the alluvium removed, or the site area was interpreted improperly due to a lack of subsurface data. In either case, the site is underlain by bedrock and potential issues related to unconsolidated alluvium, such as liquefaction potential, are not a geologic hazard at the site. The local geologic conditions are shown on the enclosed Site Geology Map, Figure 5.

3.2 GEOLOGIC CONDITIONS

As explained above, bedrock was encountered essentially at ground surface so our geologic data from the site is not consistent with the geologic maps of Lamar (1970) and Dibblee (1991). Although the orientation of bedding was not measured in the small diameter borings, bedding was measured in a cut slope across W. Reservoir Street as discussed in our geotechnical report for the adjacent site (GPI, 2019). The measured bedding strikes slightly northwesterly and dips at low inclinations (13 to 16 degrees) to the west-southwest. As such, walls facing southwest and west may expose an out-of-slope bedding component.

4.0 SITE CONDITIONS

4.1 SURFACE CONDITIONS

The subject site is about 1.1 acre in plan, and bounded by W. Reservoir Street to the northeast, a two-story public library building and parking lot to the west, W. Sunset Boulevard to the south, and a single-story restaurant and large billboard to the west. The restaurant building and billboard is directly adjacent to the property line. The library building is about 10 feet from the property line.

The existing site is occupied by asphalt paving, a smaller single-story retail building, and the existing Taix Restaurant. The pavement at our boring locations consisted of 3 to 4 inches of asphalt concrete. There is no underlying aggregate base at the locations of 2 borings and 1.5 inches of sand over 5 inches of concrete pavement in the southern boring. The pavement is in fair to poor condition. Ground surface elevations across the existing parking lot vary from about +421 feet at the north corner to approximately +414 feet at the southern portion of the site along Sunset Boulevard based on a design survey by KPFF.

4.2 SUBSURFACE SOIL CONDITIONS

Our field investigation disclosed a subsurface profile consisting of minor amounts of undocumented fill soils overlying sedimentary bedrock. Detailed descriptions of the conditions encountered are shown on the Logs of Borings in Appendix A.

We encountered shallow undocumented fill soils to depths of approximately 5 feet or less in our exploratory borings. The fill soils consisted of silts and clays. Moisture contents of the fills were observed to generally be moist. Documentation regarding the placement of the fill soils is not available.

The underlying natural materials encountered consist of shale bedrock. The shale is very stiff to hard and very moist to wet. The bedrock materials anticipated to occur below the mat foundation exhibit very low compressibility and high strength characteristics.

4.3 GROUNDWATER AND CAVING

Groundwater was encountered in one of our borings at depths of 29 and 32 feet immediately after drilling. Historical high groundwater is not well defined in this area by the State of California (CGS, 1998) but the nearest historic high groundwater contours show a depth of 20 feet below existing grades. Based on the above information, a design groundwater depth of 20 feet below existing grade along Reservoir Street or Elev. +401 feet is appropriate for this project.

Caving was not noted in the small diameter borings performed, and is not expected to be a constraint during construction.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 OVERVIEW

Based on the results of our investigation, it is our opinion that from a geotechnical viewpoint it is feasible to develop the site as proposed. The proposed structure can be supported on a spread footings with a slab-on-grade or a mat foundation provided the geotechnical constraints discussed below are mitigated. The most significant geotechnical issues that will affect the design and construction of the proposed structure are as follows:

- The planned excavation for the subterranean parking levels will remove the undocumented fills and low density upper soils across the site. Details are presented in the “Earthwork” sections of this report.

- Based on groundwater encountered in the explorations at the site and directly across the street, as well as the nearest historical high groundwater depths, a design groundwater elevation of +401 feet (20 feet below existing grade along Reservoir Street) is appropriate for the project. We anticipate that the lower level of the subterranean parking will be water-proofed and designed to resist the hydrostatic pressures imposed by the design groundwater level if the lower level extend below this elevation. Detailed recommendations are given in the “Subsurface Drainage” section of this report.

- Based on limited site access, shoring will be required during excavation of the basement level. Shoring may consist of steel soldier piles placed in drilled holes and backfilled with concrete. Driven or vibrated soldier piles may not be a feasible as alternative than drilling due to the bedrock strength at depths greater than 15 feet below grade. Based on the planned depth of the excavation, the shoring will not likely need to be tied-back using earth anchors or require rakers.

- Shale is expected to be exposed in the sidewalls of the excavation for the shoring. As discussed in the geology section of this report, the bedding is expected to be dipping to the west-southwest, resulting in a small component of adversely oriented bedding. Excavations for the shoring should be observed by a GPI Geologist to determine if significant out of slope bedding is present. Our recommended lateral pressures will need to be modified if adverse bedding exists that would impose a greater lateral pressure than those provided herein for the west and southwest facing walls.

- Chemical testing of the near surface soils has been performed by HDR, and the results are presented in Appendix B. The site soils are severely corrosive to buried metal elements. If corrosion recommendations are required, a corrosion engineer such as HDR should be consulted.

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

5.2 SEISMIC DESIGN

5.2.1 General

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

We assume the seismic design of the proposed development will be in accordance with the Los Angeles Building Code (CLABC), 2017 edition. For the 2017 CLABC, a Soil Class C may be used. The seismic code values can be obtained directly from the tables in the building code using the above values and appropriate SEAOC/OSHPD web site (seismicmaps.org). The Project Structural Engineer should determine the seismic design method.

5.2.2 Strong Ground Motion Potential

During the life of the project, the site will likely be subject to strong ground motions due to earthquakes on nearby faults. Based on the SEAOC/OSHPD website (seismicmaps.org), the site could be subjected to a peak ground acceleration (PGA_M) of 1.01g for a magnitude 6.7 earthquake. This acceleration has been computed using the mapped Maximum Considered Geometric Mean peak ground acceleration from ASCE 7-10 (ASCE, 2010) 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.

5.2.3 Potential for Ground Rupture

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

5.2.4 Liquefaction

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

The site is located within an area mapped as having a potential for soil liquefaction, as defined by the Seismic Hazards Mapping Act (Act). As defined by the Act, characteristics of the site require investigation for the potential hazard and, if a hazard exists, that its effects be mitigated. Specifically, the site is mapped within the Seismic Hazards Zone, Hollywood Quadrangle (CGS, 1999). Inclusion of a site on the hazard map does not mean that a hazard actually exists at the site. It simply means that the characteristics of the site (shallow groundwater and alluvial soil deposits) require investigation of the hazard.

As discussed in the Section 3.2, “Geologic Conditions”, of this report, alluvium at the site does not exist or has been removed during past grading. Sedimentary bedrock (shale) was encountered at depths of 5 feet or less below the existing grade. Historic high ground water is at a depth of 20 feet below existing grade. The hard shale encountered below historic groundwater is not susceptible to liquefaction.

5.2.5 Seismic Ground Subsidence

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

5.3 SUBSURFACE DRAINAGE

For the long-term operation of the building, the prevalent approach for similar projects is to waterproof and design the lower slabs and the basement walls to resist hydrostatic pressure. The reasons for this approach involve reducing exposure to environmental issues resulting in permanently pumping the groundwater. The design of the subterranean level to resist hydrostatic pressure would require a thorough waterproofing installation and a mat foundation to counteract buoyancy.

A wall backdrain system will be required for the basement walls at the base of the portion of the wall not designed to resist hydrostatic pressure. That is, if the walls are designed to resist hydrostatic pressure up to Elev. +401 feet (20 feet below grade), a perimeter drain should be installed just above this depth to collect groundwater infiltrating into the backfill from above. If a drain is not installed, all of the walls-below-grade should be designed to resist hydrostatic pressure extending up to the ground surface. Since shoring is anticipated, a rock pocket placed behind the shoring lagging may be used with the water being collected at the design groundwater level and directed to a suitable outlet.

Recommendations are presented in other sections of this report regarding the design of the spread footings with slab-on-grade, mat foundation and basement walls to resist hydrostatic pressures.

5.4 EARTHWORK

The earthwork anticipated at the project site will consist of demolition of existing improvements and pavements, clearing and grubbing, excavation for the subterranean parking, excavation of undocumented fills not removed by the excavation, subgrade preparation, and the placement and compaction of fill.

5.4.1 Clearing and Grubbing

Prior to grading, performing excavations, or constructing the proposed improvements, the areas to be developed should be stripped of vegetation and cleared of existing structures, 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 the clearing operation, including organic topsoil, should be removed from the site. If approved by the owner and regulatory agency, inert demolition debris, such as concrete and asphalt may be crushed for reuse in engineered fills outside the planned building areas in accordance with the criteria presented in the "Materials for Fill" section of this report.

If cesspools or septic systems are encountered during grading, they should be removed in their entirety. The resulting excavation should be backfilled as recommended in the "Subgrade Preparation" and "Placement and Compaction of Fill" sections of this report. As an alternative, cesspools can be backfilled with lean sand-cement slurry in accordance with City of Los Angeles Information Bulletin 2014-121. At the conclusion of the clearing operations, a representative of GPI should observe and accept the site prior to further grading.

5.4.2 Excavations

Excavations at this site will include the subterranean parking excavation, removals of undocumented fills not removed by the excavation, footing excavations, and trenching for new utility lines.

The City of Los Angeles does not permit supporting new fills, pavements or foundations on undocumented fills. Prior to placing fills or construction of the building or pavements, undocumented fills within the proposed building area and under future pavements or fills should be removed and replaced as properly compacted fill. Based on the project plans, the fill soils within the building limits are expected to be removed during the planned excavation for the subterranean parking levels. Some undocumented fill soils may remain outside the building footprint after the basement excavation is completed. Remaining undocumented fill should be overexcavated and replaced with compacted fill as outlined below.

For minor at-grade supported structures, such as screen walls, canopies, or short retaining walls, the existing fills should be removed and the footings should be underlain by competent bedrock or properly compacted fill. For pavement and hardscape outside the building, the soils within 2 feet of the existing or finished grade, whichever is lower, should be overexcavated and replaced with properly compacted fill. Localized deeper excavations may be required. The actual depths of removals should be determined in the field during grading by a representative of GPI. Existing grades refer to the grades at our exploration locations.

Where space is available, the base of the overexcavation should extend laterally at least 5 feet beyond the footings for the at-grade structures. The Project Surveyor should accurately stake the corners of the areas to be overexcavated in the field.
Groundwater was encountered in one of our borings at depths of 29 and 32 feet below existing grade. Historical groundwater levels in the site vicinity appear to occur at a depth of 20 feet deep. The potential for wet soils and, possibly seepage, should be considered when planning the excavations required for foundations, vaults, and elevator equipment.

The sedimentary bedrock encountered in our explorations at the basement level are near or well above optimum moisture content. The earthwork contractor should evaluate the moisture content of the existing wet soils derived from the sedimentary bedrock when planning the required earthwork and soil export.

Where not removed by the aforementioned excavations, existing undocumented utility trench backfill remaining below new foundation areas should be removed and replaced as properly compacted fill. This is especially important for deeper fills such as existing sewers and storm drains. For planning purposes, removals over the utilities should extend to within 1-foot of the top of the pipe. The removal should extend laterally 1-foot beyond both sides of the pipe. The actual limits of removal will be confirmed in the field. We recommend that known utilities be shown on the grading plan.

Temporary construction excavations may be made vertically without shoring to a depth of 5 feet below the adjacent grade. For cuts up to 12 feet, the slopes should be properly shored or sloped back to at least 1:1 (horizontal to vertical) or flatter. For cuts up to 25 feet, the slopes should be properly shored or sloped back to at least 1½:1 or flatter. The inclination is measured from the top to toe of slope, and we do not recommend incorporating a vertical cut at the base of the slope. 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 the 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 most current State of California Occupational Safety and Health Standards.

As discussed in the “Geology” section, the bedding in the bedrock is dipping. The anticipated excavations facing southwest and west are expected to expose shallow adverse bedding resulting in the need to design for out of slope bedding plane surcharging. As discussed in the “Retaining Structures and Shoring” section of this report, our recommended earth pressures for the northeast and east side of the excavation have been increased to take into account some contribution from adverse bedding. During the excavation, our geologist should observe the sidewalls for evidence of adverse bedding.

5.4.3 Subgrade Preparation

After removals are complete and prior to placing fills or constructing of proposed at-grade structures, the subgrade soils should be scarified to a depth of 8 inches, moisture-conditioned, and compacted to dry densities equal to at least 90 percent of the maximum dry density (95 percent for granular soils), determined in accordance with ASTM D 1557. In areas where very moist to wet soils are encountered, scarification of the subgrade may be omitted when permitted by a representative of GPI.

We recommend that the subgrade consisting of the shale below the proposed floor slab be left undisturbed in order to minimize the potential for swell.

In areas to receive pavements (outside of the structure), the top 12 inches below the pavement base should be scarified, moisture-conditioned, and compacted to a minimum of 95 percent (90 percent for cohesive soils) of the maximum dry density.

5.4.4 Material for Fill

Soils available from on-site excavations, less debris or organic matter, will be suitable for re-use in fills with the exception of fills behind retaining walls or directly beneath exterior flatwork. Retaining wall backfill and soils within 1-foot of finished grade for exterior hardscape and flatwork should consist of on-site or imported granular (containing no more than 40 percent fines – portion passing the No. 200 sieve) and relatively non-expansive (Expansion Index of 20 or less) soils.

Imported fill material should be predominately granular (containing no more than 40 percent fines - portion passing No. 200 sieve) and non-expansive (E.I. of 20 or less). Import or on-site materials used in compacted fills should not contain particles larger than 6 inches in diameter. GPI should be provided with a sample (at least 50 pounds) and notified of the location of soils proposed for import at least 72 hours in advance of importing. Each proposed import source should be sampled, tested and accepted for use prior to delivery of the soils to the site. Soils imported prior to acceptance by GPI may be rejected if not suitable.

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 at least one sack of cement per cubic yard and have a maximum slump of 5 inches. When set, such a mix typically has the consistency of compacted soil. The use of sand cement slurry should comply with the appropriate City of Los Angeles Information Bulletin.

From a geotechnical engineering standpoint, asphalt concrete or portland cement concrete can be incorporated into fills placed outside the building areas provided that they are crushed to the consistency of aggregate base and thoroughly blended with enough soil to form a well-graded mixture (typically a 3:1 soil to debris ratio). Such material should not be placed within landscape areas. Approval from the owner and LADBS should be obtained prior to use of the inert materials.

In areas where open-graded gravel, such as pea gravel or ¾-inch crush rock, is placed, the gravel should be separated from the on-site soils with a suitable non-woven filter fabric, such as Mirafi 140N. The purpose of the filter fabric is to reduce the potential for soil particles to migrate into the void spacing of the gravel.

5.4.5 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. Imported granular fill 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	6-8 inches
Heavy loaders or vibratory rollers	8-12 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 consisting of the on-site silts derived from the bedrock should be placed at a moisture content of 1 to 3 percent over the optimum moisture content in order to achieve the required compaction and reduce the potential for future swelling. Imported granular fills should be placed at a moisture content of 0 to 2 percent over the optimum moisture content.

Once moisture conditioned and properly compacted, the exposed soils should not be allowed to dry out prior to covering. If exposed soils are allowed to dry out, processing and moisture conditioning will be required. A representative of GPI should confirm the moisture content of the subgrade soils immediately prior to placement of concrete or additional fill.

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

5.4.6 Shrinkage and Subsidence

Shrinkage is the loss of soil volume caused by compaction of fills to a higher density than before grading. Subsidence is the settlement of in-place subgrade soils caused by loads generated by large earthmoving equipment. Neither shrinkage nor subsidence is anticipated to be a major factor on the project because of the significant soil export. Actual shrinkage and subsidence will depend on the types of earthmoving equipment used and should be verified during grading.

5.4.7 Trench/Wall Backfill

Utility trench and wall backfill, consisting of the on-site materials (trenches only) or imported sand, should be mechanically compacted in lifts. Lift thickness should not exceed those values given in the "Compacted Fill" section of this report. Moisture conditioning of the on-site soils will be required prior to re-use as backfill. Jetting or flooding of backfill materials should not be permitted. GPI should observe and test trench and wall backfills as they are placed.

In backfill areas where mechanical compaction of soil backfill is impractical due to space constraints, sand-cement slurry may be substituted for compacted backfill. The slurry should contain one sack of cement per cubic yard and have a maximum slump of 5 inches. Within building areas, the slurry should contain two sacks of cement per cubic yard.

5.4.8 Observation and Testing

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

5.5 FOUNDATIONS

5.5.1 Foundation Type

The anticipated column loads can be supported on conventional spread footings or a mat foundation founded on the bedrock. We have provided recommendations for both type of foundations.

Footings for at-grade structures, such as screen walls or small retaining walls, should be supported on properly compacted fill or competent bedrock.

Depending on the finished floor level as compared to design groundwater level (Elev. +401 feet), buoyant forces should be considered for the mat foundation that extends below the water.

5.5.2 Mat Foundation

The proposed structure may be supported on a mat bearing on the undisturbed bedrock occurring at depths of approximately 5 feet or less below existing grades. Based on information provided by the project team, we understand that the base of the mat will be established at depths of approximately 13 to 16 feet below existing site grades.

For design of the mat foundation using a spring constant or modulus of subgrade reaction (k-value), a value of 200 pounds per cubic inch (pounds per square inch per inch of deflection) may be assumed for the bedrock and a 1-foot square loaded area. For the larger area of the mat foundation, we recommend that a reduced k-value of 50 pci be used for design.

The mat will be irregularly shaped with the longest width and length of approximately 450 feet and 500 feet in plan dimension. Based our experience with similar projects, the bearing pressure across the mat will vary from approximately 300 psf to 600 psf. The allowable bearing capacity of the mat is far greater than the anticipated design pressures.

Based on an average mat pressure of 450 psf, we estimate that the ground surface under the center portions of the loaded area having the above dimensions and the aforementioned applied pressure will settle approximately ½ inches. The outside edge of this area under the same loading conditions is expected to settle approximately ¼-inch. The outside corner of this area under the same loading conditions is expected to settle less than ¼-inch.

The static settlements assume a uniformly applied pressure and do not include the effects (stiffness) of the mat. The actual settlement of the mat will depend on the stiffness of the mat and its ability to distribute the loads. The majority of the settlements will occur as the loads are applied.

5.5.3 Spread Footings

Spread footings for the building should be supported on competent bedrock. Footings adjacent to basement walls should be deepened in order to avoid surcharging the wall. Footings should be deepened to below the 1:1 project from the bottom of the adjacent basement wall.

Footings for at-grade structures, such as screen or retaining walls, should be supported on properly compacted fill or competent bedrock.

Allowable Bearing Pressures

Based on the shear strength and elastic settlement characteristics of the on-site soils, a static allowable net bearing pressure of up to 5,000 pounds per square foot (psf) may be used for both continuous footings and/or isolated column footings. A static allowable net bearing pressure of up to 2,000 pounds per square foot (psf) may be used for both continuous footings and isolated column footings for minor structures supported at-grade on properly compacted fill.

The actual bearing pressure used may be less, such that economics and structural loads will determine the minimum width for footings as discussed below. These bearing pressures are for dead-plus-live loads, and may be increased one-third for short-term, transient, wind and seismic loading. The maximum edge pressures induced by eccentric loading or overturning moments should not be allowed to exceed these recommended values.

The actual bearing pressure used may be less, such that economics and structural loads will determine the minimum width for footings as discussed below. The maximum edge pressures induced by eccentric loading or overturning moments should not be allowed to exceed these recommended values.

Minimum Footing Width and Embedment

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

STATIC BEARING PRESSURE (psf)	MINIMUM FOOTING WIDTH (inches)	MINIMUM FOOTING* EMBEDMENT (inches)
Building Foundations on Bedrock		
5,000	72	24
4,000	60	24
3,500	48	24
3,000	36	24
2,000	24	24
Minor At-Grade Structures on Engineered Fill or Competent Fill		
2,000	24	18
1,500	18	18
1,000	15	15

* Depth to bottom of footing below lowest adjacent finish grade.

A minimum footing width and depth of 24 inches should be used even if the actual bearing pressure is less than 2,000 psf for the building footings. A minimum footing width and depth of 15 inches should be used even if the actual bearing pressure is less than 1,000 psf for minor structures.

Estimated Settlements

For the anticipated loads for buildings supported on competent bedrock, static settlement is expected to be less than 1-inch. Maximum differential static settlements between similarly loaded footings are expected to be less than ½-inch across a distance of 40 feet.

For the anticipated loads for minor structures supported on 2 feet of compacted fill, static settlement is expected to be less than ½-inch. Maximum differential static settlements are expected to be less than ¼-inch across a distance of 40 feet.

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

5.5.4 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 0.30 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 the compacted fill or undisturbed natural soils. These values may be used in combination without reduction.

5.5.5 Foundation Concrete

Based on laboratory testing by HDR (Appendix B) soluble sulfate contents of the on-site soils were found to be 426 mg/kg (0.043 percent by weight). Based on the test results, foundation concrete should conform to the requirements outlined by ACI 318, Section 4.3 and the 2017 CLABC for negligible sulfate content.

5.5.6 Foundation Inspection

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

5.6 RETAINING STRUCTURES AND SHORING

Basement walls, cantilever retaining walls, and temporary shoring are planned for the site. The following recommendations are provided for walls up to 15 feet tall and shoring that does not extend more than 20 feet in height. We recommend that conventionally backfilled walls be backfilled with sandy (granular) soils.

5.6.1 Basement and Retaining Walls

Active pressure may be used in the design of the subterranean walls if the total movement of the wall is sufficient to mobilize the active pressure (yielding at least ½-inch laterally in 10 feet of wall height). For cantilever walls with level, drained backfill comprised of granular soils, the magnitude of active pressures is equivalent to the pressures imposed by a fluid weighing 35 pounds per cubic foot (pcf). For cantilever walls retaining level, drained undisturbed native bedrock, the magnitude of active pressures is equivalent to the pressures imposed by a fluid weighing 40 pounds per cubic foot (pcf). For unrestrained walls supporting the northeast and east excavations (southwest and west facing cuts) with adverse bedding, the walls should be designed for an equivalent fluid pressure of 65 pcf.

At-rest pressures should be used for restrained walls that remain rigid enough to be essentially non-yielding. At-rest pressures imposed by a fluid weighing 56 pounds per cubic foot should be used for drained, existing native bedrock. For walls supporting the northeast and east excavations (southwest and west facing cut), the basement walls should be designed for an at-rest pressure of 83 pcf.

To account for seismic loads, an additional lateral earth pressure equal to 26 pcf (equivalent fluid pressure distribution) should be added to the above active pressure. If the wall is designed using the above at-rest pressure, the at-rest pressure with the seismic load may be limited to the value of active pressure with seismic load.

For undrained backfill, we recommend the above lateral pressures be increased by a hydrostatic pressure equivalent to a fluid with a density of 40 pounds per cubic foot. These undrained pressure increases reflect the potential for groundwater to rise or for infiltration of surface water. The City requires the design groundwater depth to be consistent with the historical high determined by the State of California. For this project, the design groundwater elevation is +401 feet below existing grades if subsurface drains are installed at that level. If subsurface drains are eliminated, the hydrostatic pressure should be taken from the ground surface.

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. In addition to the recommended earth pressure, the upper 10 feet of the walls adjacent to the streets should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pound per square foot surcharge behind the shoring due to normal street traffic. If traffic is kept at least 10 feet from the walls, the traffic surcharge may be neglected.

Construction equipment, such as cranes, concrete trucks, or loaders supported on the ground adjacent to the walls can impose lateral surcharge loads if they are supported adjacent to the basement walls (or shoring). Therefore, surcharge effects from such equipment will need to be evaluated on a case-by-case basis and, if needed, the walls locally reinforced to support the surcharge from such loads.

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 gravel wrapped in filter fabric should be used. One cubic foot of rock should be used for each lineal foot of pipe. The fabric (non-woven filter fabric, Mirafi 140N or equivalent) should be lapped at the top. We prefer pipe and gravel drains to weep holes to avoid potential for constant flow of surface water in front of the wall. For retaining walls constructed adjacent to temporary shoring, a composite geotextile drain may be used with a manifold-type collection drain at the design groundwater level. In addition, “rock-pockets” should be installed at the design groundwater level with a collection pipe extending from the “rock-pocket” to the collection system. A representative of GPI should observe and approve wall drains prior to placement of wall backfill.

As a minimum, if the walls below grade are drained, we recommend that they be damp-proofed to reduce the adverse effects of moisture intrusion into the structure. As added protection, the walls below grade should be water-proofed.

The Structural Engineer should specify the use of select, granular wall backfill on the plans for walls that are to be conventionally backfilled. Wall footings should be designed as discussed in the "Foundations" section.

5.6.2 Temporary Shoring

Where there is not sufficient space for sloped embankments, such as along the property limits, shoring will be required. Based on current plans, cantilever shoring is anticipated along all sides of the project site. One method of shoring would consist of steel soldier piles placed in drilled holes, backfilled with concrete, and wood lagging. Tie-back anchors are not anticipated for the shoring with anticipated heights of less than 15 feet.

The shoring contractor should evaluate the subsurface conditions when planning the installation methods. Because of the hard layers of shale at depths of approximately 15 to 20 feet below existing grade, driven or vibrated soldier piles may not be a feasible and an economical alternative to drilled holes. The presence of hard shale should be considered when evaluating the alternatives for soldier piles.

A GPI Geologist should observe the bottom of excavation sidewalls to assess the presence of adverse bedding.

For cantilever shoring with level backfill consisting of the on-site soils, the magnitude of active pressure is equivalent to the pressures imposed by a fluid weighing 40 pounds per cubic foot (pcf). For northeast and east basement wall, the magnitude of active pressure equivalent to the pressures imposed by a fluid weighing 63 pounds per cubic foot (pcf) should be used to account for the likely presence of adverse bedding. It should be noted that the provided lateral earth pressures assume a fully drained condition and do not include hydrostatic pressures.

Shoring 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. Surcharge loads may include the adjacent buildings and the billboard foundation. In addition to the recommended earth pressure, the upper 10 feet of the shoring adjacent to streets should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pound per square foot surcharge behind the shoring due to normal street traffic. If traffic is kept at least 10 feet from the shoring, the traffic surcharge may be neglected.

For design of soldier piles spaced at least two diameters on centers, the allowable lateral bearing value (passive value) of the soils below the excavation may be taken to be 600 pounds per square foot at the excavated surface, up to a maximum of 6,000 psf. To develop the full lateral value, provisions should be made to assure firm contact between the soldier piles and the undisturbed soils. The concrete placed in the soldier pile excavation below the excavated level may be a lean mix, but it should be of adequate strength to transfer the imposed loads to the surrounding soils. While not anticipated due to hard bedrock conditions, if the soldier piles are driven or vibrated into place, the design width of the soldier piles (effective pile diameter) used in calculations should be equal to the actual width of the flange of the soldier piles.

While not anticipated to be feasible, driving of soldier piles to improve production or minimize ground vibration should only allow predrilling down to the design elevation of the excavation bottom provided that a continuous flight auger is utilized to enable reversing the auger to minimize the removal of soil during the process. If soil is removed during the predrilling process, the resulting void should be backfilled with 1½ sack sand-cement slurry. The diameter of the auger used for predrilling should not exceed 80 percent of the maximum depth of the soldier pile beam section.

The frictional resistance between the soldier piles and the retained earth may be used in resisting the downward component of the anchor load. The coefficient of friction between the soldier pile and the retained earth may be taken as 0.35. This value is based on the assumption that uniform full bearing will be developed between the steel soldier beam and the lean-mix concrete and between the lean mix concrete and the retained earth. In addition, provided the portion of the soldier piles below the excavated level is backfilled with structural concrete, the soldier piles below the excavated level may be used to resist downward loads. The frictional resistance between the concrete soldier piles and the soils below the excavated level may be taken as equal to 500 pounds per square foot.

Continuous lagging will be required between the soldier piles. Careful installation of the lagging will be necessary to achieve bearing against the retained earth. We recommend that the voids between the lagging and retained earth be backfilled with a lean-mix sand-cement slurry prior to continuing the excavation deeper. The soldier piles should be designed for the full anticipated lateral pressure. However, the pressure on the lagging will be less because of arching of the soils between piles. We recommend that the lagging be designed for the recommended earth pressure but limited to a maximum value of 400 pounds per square foot, provided the soldier beam spacing is 8 feet or less.

It is difficult to accurately predict the amount of deflection of the shored embankment. It should be realized, however, that some deflection will occur. Adjacent to city right-of-way, the shoring should be designed to limit deflection to 1-inch. If greater deflection occurs during construction, additional bracing may be necessary. In areas where less deflection is desired, such as adjacent to existing settlement sensitive improvements, the shoring should be designed for higher lateral earth pressures. We recommend limiting the lateral deflection of shoring adjacent to any buildings to ½-inch.

While not anticipated at this project, driven/vibrated soldier piles should be limited to areas beyond 20 feet from existing buildings, and to a greater distance where adjacent structures appear to be sensitive to vibration or settlement. Ground vibrations could be monitored when driving/vibrating soldier piles adjacent to sensitive structures. A seismograph should be used to measure peak particle velocities (PPV) at the ground surface of the structures of concern. We suggest a maximum allowable PPV of 0.5 inches per second be used as a threshold value unless a lower value is required by the adjacent property owners. Measures should be taken to reduce vibrations if PPV limits are exceeded. Such measures could include altering the predrilling methods or changing to the installation of the soldier piles in a drilled and grouted hole.

We recommend performing a detailed survey of the improvements to be supported above the planned shoring prior to and during the shoring installation. The survey should include topographic data and a video account of the condition of the existing improvements, including cracks or signs of distress. During construction, the monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of the soldier piles. We suggest weekly readings during the excavation and for the first three weeks after achieving the bottom of the excavation. After that time, the readings should be performed every other week until the completion of the basement walls.

5.7 CONCRETE FLOOR SLABS

Although not anticipated for the subterranean parking level, a moisture vapor retarder should be placed under slabs that are to be covered with moisture-sensitive floor coverings (parquet, vinyl, tile, etc.). Polyolefin in 15-mil thickness should be covered by a layer of clean sand (less than 5 percent by weight passing the No. 200 sieve) having a minimum thickness of 2 inches. Based on our explorations and laboratory testing, the soils at the site are not suitable for this purpose. The function of the sand layer is to protect the vapor retarder during construction and to aid in the uniform curing of the concrete. This layer should be nominally compacted using light equipment. The sand placed over the vapor retarder should only be slightly moist. If the sand gets wet (for example as a result of rainfall or excessive moistening) it must be allowed to dry prior to placing concrete. Care should be taken to avoid infiltration of water into the sand layer after placement of the concrete slab, such as at slab cut-outs and other exposures.

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-cement ratio for the concrete used for the floor slab, effective sealing of joints and edges (particularly at pipe penetrations) as well as excess moisture in the concrete. The manufacturer of the floor coverings should be consulted for establishing acceptable criteria for the condition of the floor surface prior to placing moisture-sensitive floor coverings.

5.8 CORROSION

Soil corrosivity testing was performed by HDR under subcontract to GPI. The corrosivity test results are presented in Appendix B. The on-site soils should be considered severely corrosive to buried metals. If additional corrosion consultation is required, a corrosion engineer such as HDR should be consulted.

5.9 EXTERIOR CONCRETE AND MASONRY FLATWORK

Exterior concrete and masonry flatwork should be supported on imported non-expansive compacted fill. The use of clayey soils or soils derived from the on-site shales within the upper 24 inches of exterior flatwork subgrade is not recommended. Prior to placement of concrete, the subgrade should be prepared as recommended in “Subgrade Preparation” section.

5.10 STORMWATER INFILTRATION

In accordance with the requirements of the City of Los Angeles, stormwater infiltration in soils retained by basements or retaining walls is not permitted. To achieve this requirement, infiltration below the bottom of the finish floor or adjacent to basement walls would be required. The materials occurring at this depth consist of fine-grained bedrock, not suitable for infiltration. Therefore, we recommend that stormwater infiltration at the site be avoided.

5.11 PAVED AREAS

Preliminary pavement design has been based on an assumed R-value of 10. The California Division of Highways Design Method was used for design of the recommended preliminary pavement sections. These recommendations are based on the assumption that the pavement subgrades will consist of the existing soils. The subgrade soil conditions will need to be confirmed at the conclusion of rough grading.

PAVEMENT AREA	TRAFFIC INDEX	SECTION THICKNESS (inches)	
		ASPHALT/PORTLAND CONCRETE	AGGREGATE BASE COURSE
Asphalt Concrete			
Automobile Parking	4.0	3.0	6
Automobile Drives	5.0	3.0	9
Truck Drives	6.0	3.5	13
Portland Cement Concrete			
Automobile Parking	4.0	6.5	4
Automobile Drives	5.0	6.5	4
Truck Drives	6.0	7.0	4

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

The pavement base course should be compacted to at least 95 percent of the maximum dry density (ASTM D 1557). Aggregate base should conform to the requirements of Section 26 of the California Department of Transportation Standard Specifications for Class II aggregate base (three-quarter-inch maximum) or Section 200-2 of the Standard Specifications for Public Works Construction (Green Book) for untreated base materials (except Processed Miscellaneous Base).

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

5.12 SURFACE 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.

5.13 GEOTECHNICAL OBSERVATION AND TESTING

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

6.0 LIMITATIONS

This report, exploration logs, and other materials resulting from GPI's efforts were prepared exclusively for use by Holland Acquisition Co., 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 project other than the currently proposed development as it may not contain sufficient or appropriate information for such uses.

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 construction phase services, the client and new geotechnical firm 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,
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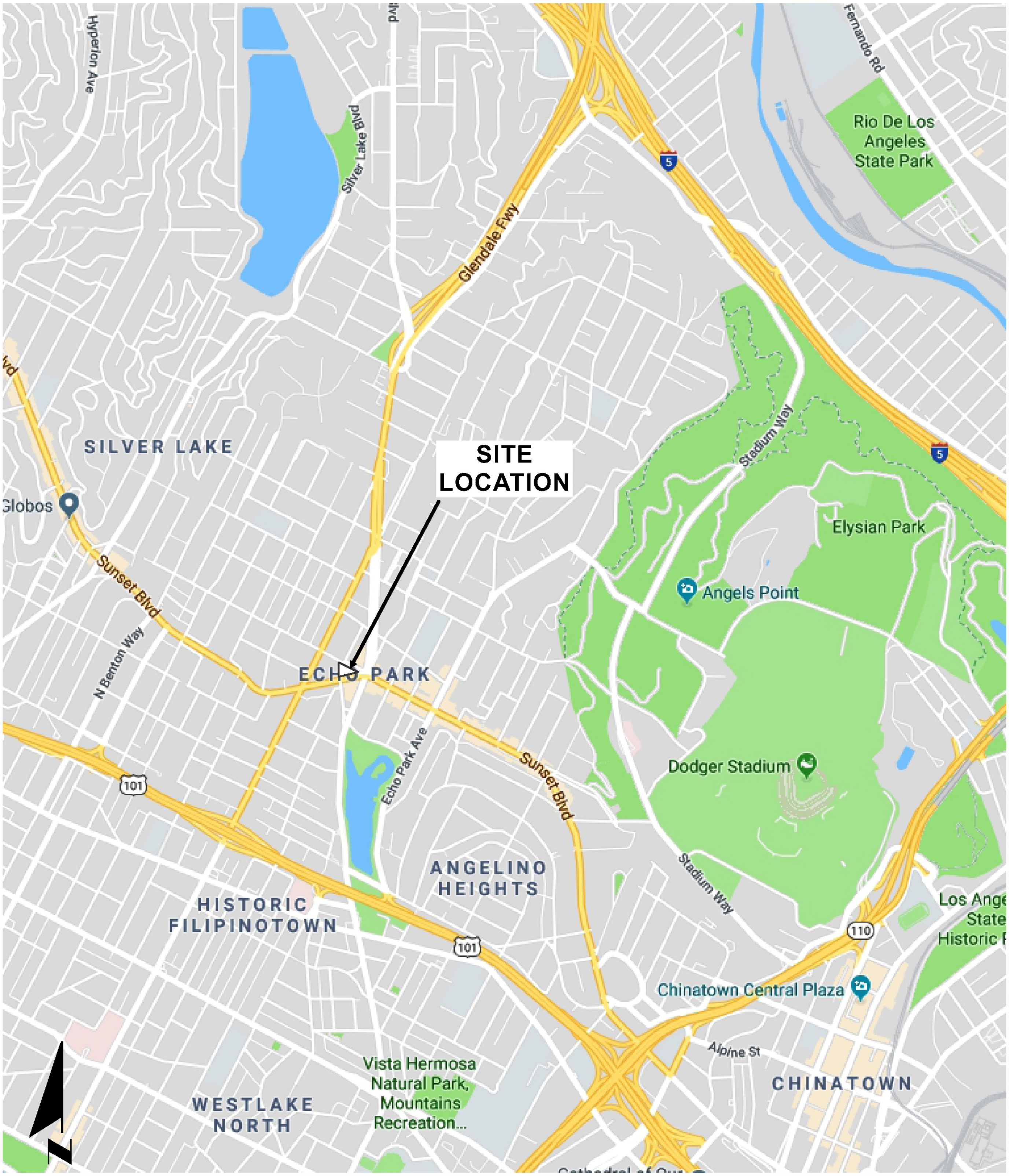


0 2000 4000 FEET

BASE MAP REPRODUCED FROM GOOGLE EARTH © 2019

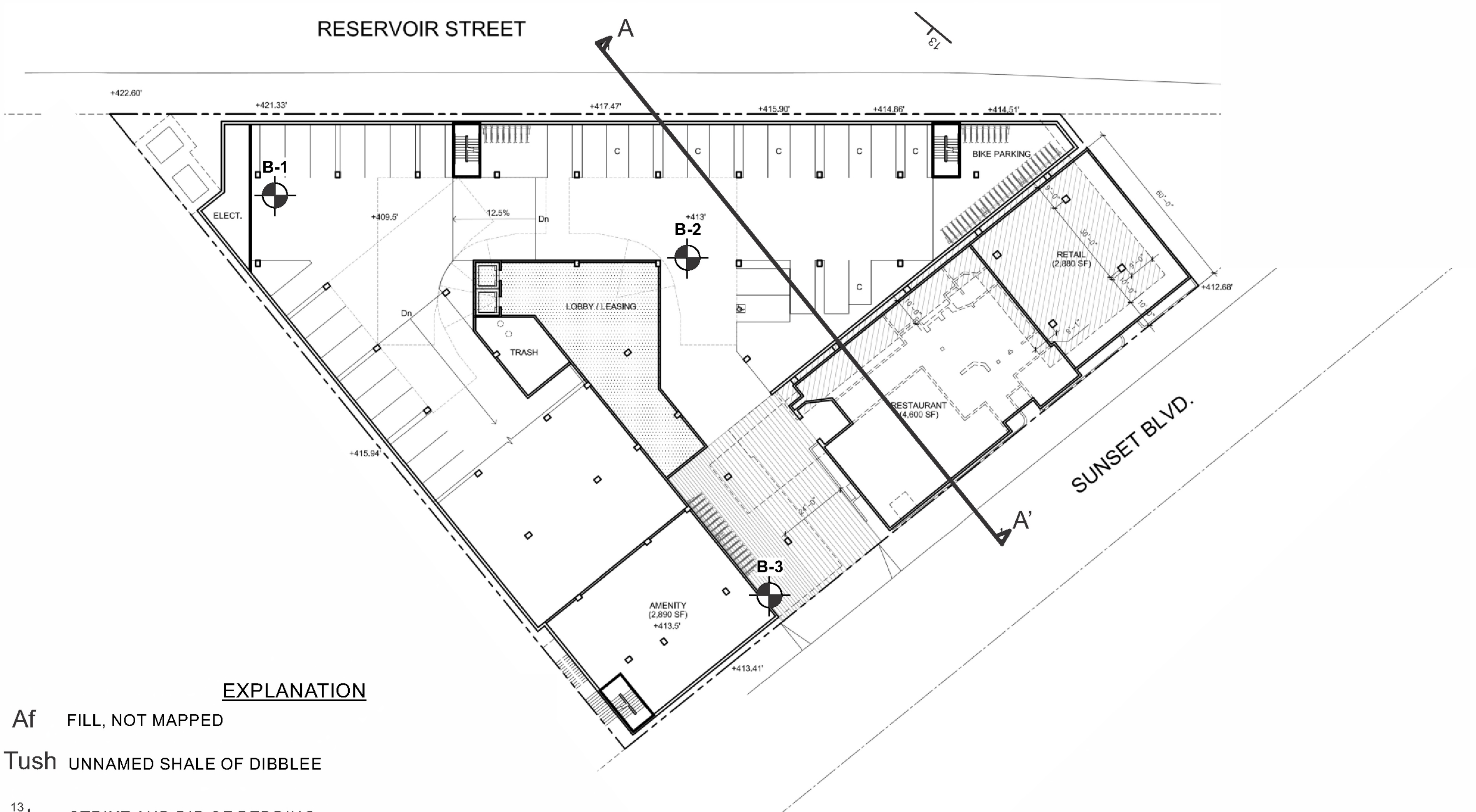
GEOTECHNICAL
PROFESSIONALS, INC.

TAIX SUNSET

GPI PROJECT NO. 2914.I | SCALE: 1" = 2000'

SITE LOCATION MAP

FIGURE 1



EXPLANATION

Af FILL, NOT MAPPED

Tush UNNAMED SHALE OF DIBBLEE

13 STRIKE AND DIP OF BEDDING

A A CROSS SECTION

B-3 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING

N

0 50 100 FEET

BASE MAP PRODUCED FROM SITE PLAN BY TSM INCORPORATED, DATED JANUARY 14, 2019

GPI GEOTECHNICAL PROFESSIONALS, INC.

TAIX SUNSET

GPI PROJECT NO.: 2914.I

SCALE: 1" = 50'

SITE PLAN (STREET LEVEL)

FIGURE 2

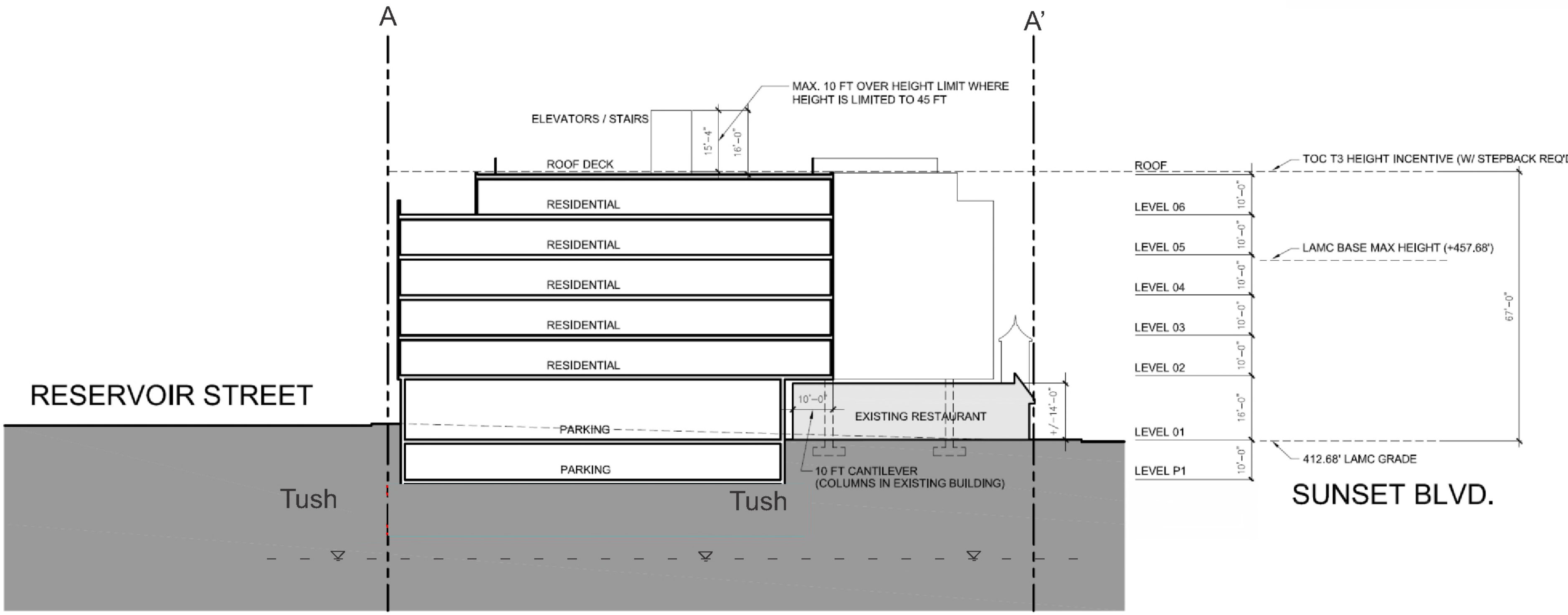
GPI GEOTECHNICAL PROFESSIONALS, INC.

TAIX SUNSET

GPI PROJECT NO.: 2914.I	SCALE: 1" = 50'

SITE PLAN
(P1 LEVEL)

FIGURE 3



BASE MAP PRODUCED FROM SECTION PLAN BY TSM INCORPORATED, DATED JANUARY 8, 2019

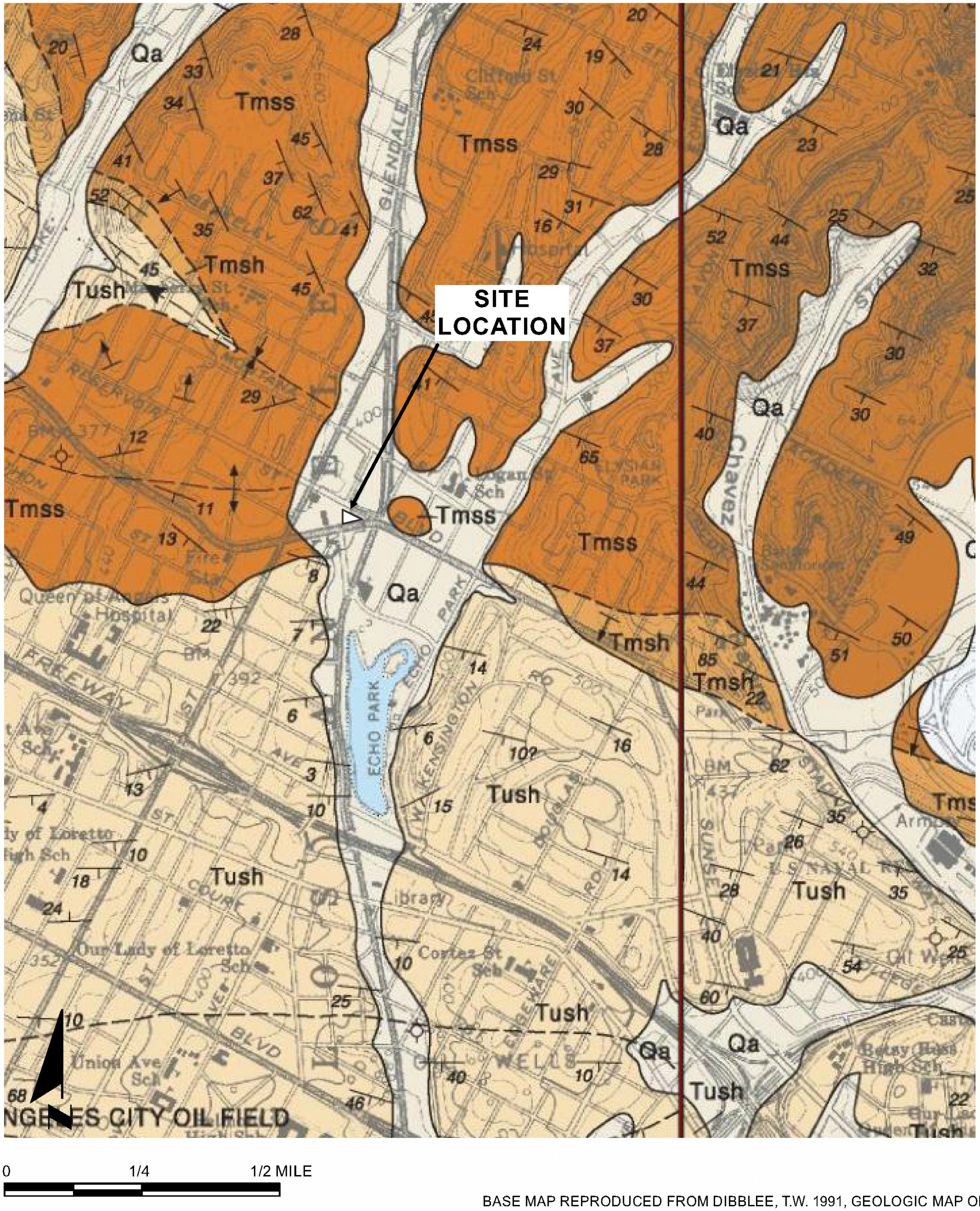
GPI GEOTECHNICAL PROFESSIONALS, INC.

TAIX SUNSET

GPI PROJECT NO.: 2914.I

SITE SECTION

FIGURE 4



BASE MAP REPRODUCED FROM DIBBLEE, T.W. 1991, GEOLOGIC MAP OF THE HOLLYWOOD AND BURBANK (SOUTH 1/2) QUADRANGLES

GPI GEOTECHNICAL PROFESSIONALS, INC.	**SITE GEOLOGIC MAP**
TAIX SUNSET	
GPI PROJECT NO. 2914.I / SCALE: 1" = 1/4 MILE	FIGURE 5

APPENDIX A

APPENDIX A

EXPLORATORY BORINGS

The subsurface conditions at the site were investigated by drilling and sampling three exploratory borings. The borings were advanced to depths of 41 to 71 feet below the existing ground surface. The exploration locations are shown on the Site Plans, Figures 2 and 3.

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

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

The field exploration for the investigation was performed under the continuous technical supervision of GPI's representative, who visually inspected the site, maintained detailed logs of the borings, classified the soils encountered, and obtained relatively undisturbed samples for examination and laboratory testing. The soils encountered in the boring were classified in the field and through further examination in the laboratory in accordance with the Unified Soils Classification System. Detailed logs of the borings are presented in Figures A-1 to A-3 in this appendix.

The boring and test pit locations were laid out in the field by measuring from existing features at the site. Upon completion, the borings were backfilled with the excavated soil cuttings. The ground surface elevations at the boring locations were estimated from a design survey plan prepared by KPFF and should be considered approximate.

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
						This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
				B	0	3-Inch AC	
						Fill: **CLAY (CL)** brown, moist	420
	29.0	80	11	D	5	@ 5 feet, dark brown, firm	
						Natural: **UNNAMED SHALE** light brown, very moist to wet, firm	415
	22.6	92	27	D	10	@ 10 feet, very stiff	
							410
	24.4	97	62	D	15	@ 15 feet, hard	
							405
	23.3	100	61	D	20		
							400
	26.1		22	S	25	@ 25 feet, stiff to very stiff, with sandstone layers	
							395
	26.7	96	93/9"	D	30	@ 30 feet, hard	
							390
	27.7		32	S	35		
							385

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

DATE DRILLED:
12-19-18

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
29

PROJECT NO.: 2914.I

TAIX - SUNSET

LOG OF BORING NO. B-1

FIGURE A-1

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)		DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
							This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
	24.0	96	75/8"	D	40		**UNNAMED SHALE** light brown, very moist to wet, hard	
								380
	36.8		20	S	45			
								375
	24.3	97	50/5"	D	50			
								370
	28.8		65/11"	S	55			
								365
	22.3	101	70/6"	D	60		@ 60 feet, with sandstone	
								360
	21.8		50/4"	S	65			
								355
	20.1	100	70/5"	D	70			
							Total Depth 71 feet	

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

DATE DRILLED:
12-19-18
EQUIPMENT USED:
8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft):
29

GPI

PROJECT NO.: 2914.I
TAIX - SUNSET

LOG OF BORING NO. B-1

FIGURE A-1

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
				B	0	4-Inch AC	
						Fill: **SILT (ML)** orange brown, moist	415
	26.9	97	69	D	5	Natural: **UNNAMED SHALE** brown, very moist to wet, hard	
							410
	23.2	99	56/6"	D	10		
							405
	22.7	98	56	S	15	@ 15 feet, orange brown and light grey	
							400
	24.7	92	56/6"	D	20		
							395
	25.0	93	98/9"	D	25		
							390
	26.6	94	53/6"	D	30		
							385
	24.5	94	53/6"	D	35	@ 35 feet, layers of sandstone	
							380

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:
12-19-18

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2914.I

TAIX - SUNSET

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)
	26.2	94	82/11"	D	40		**UNNAMED SHALE** brown, very moist to wet, hard	
							Total Depth 41 feet	

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

DATE DRILLED:
12-19-18

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2914.I

TAIX - SUNSET

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.	
				B	0	3-Inch AC over 1.5-Inch SAND over 5" PCC	
						Fill: **SILT (ML)** light brown, slightly moist to moist	
							410
	26.2	95	47	D	5	Natural: **UNNAMED SHALE** brown, very moist to wet, hard	
							405
	21.5	100	52	D	10		
							400
	24.1	98	80/10"	D	15		
							395
	22.8	94	74/10"	D	20		
							390
	21.9	97	87/9"	D	25		
							385
	26.8	94	82/10"	D	30		
							380
	24.7	96	77/11"	D	35		
							375

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

DATE DRILLED:
12-20-18
EQUIPMENT USED:
8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft):
32

GPI

PROJECT NO.: 2914.I
TAIX - SUNSET

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)
27.4	91	73/11"	D	40	**UNNAMED SHALE** brown, very moist to wet, hard	
					Total Depth 41 feet	

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

DATE DRILLED:
12-20-18

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
32

GPI

PROJECT NO.: 2914.I

TAIX - SUNSET

LOG OF BORING NO. B-3

FIGURE A-3

APPENDIX B

APPENDIX B

LABORATORY TESTS

INTRODUCTION

Representative undisturbed soil samples, tube samples and bulk samples were carefully packaged in the field and sealed to prevent moisture loss. The samples were then transported to our Cypress office for examination and testing assignments. Laboratory tests were performed on selected representative samples as an aid in classifying the soils and to evaluate the physical properties of the soils affecting foundation design and construction procedures. Detailed descriptions of the laboratory tests are presented below under the appropriate test headings. Test results are presented in the figures that follow.

MOISTURE CONTENT AND DRY DENSITY

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

DIRECT SHEAR

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

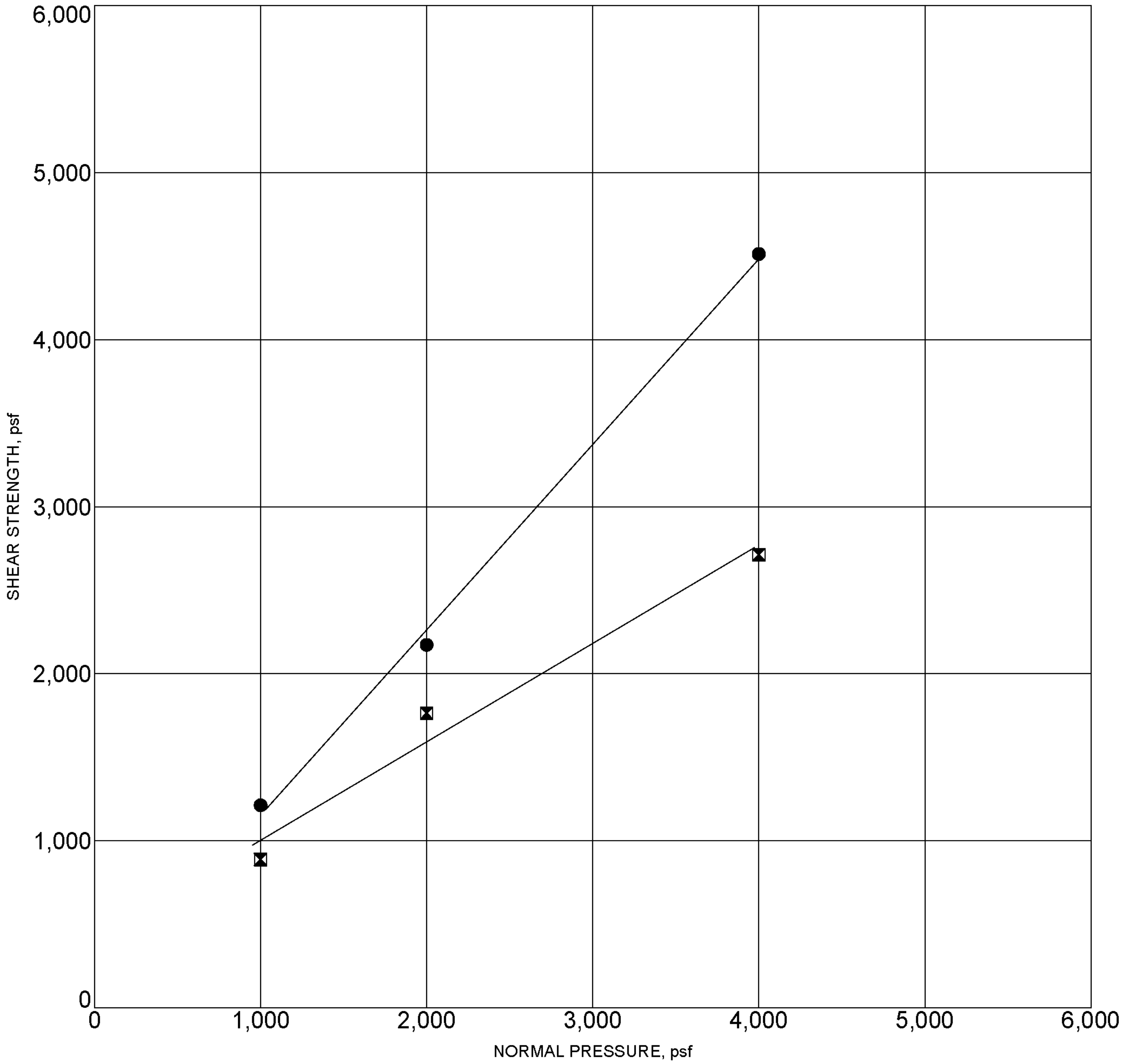
EXPANSION INDEX

An expansion test was performed in accordance with ASTM D 4829 on a sample to assess the expansion potential of the on-site soils. The results of the test are summarized below.

BORING N0.	DEPTH (ft)	SOIL DESCRIPTION	EXPANSION INDEX
B-2	0 - 5	Silt (ML)	90

CORROSIVITY

Soil corrosivity testing was performed by HDR on a soil sample provided by GPI. The test results and corrosion protection recommendations are summarized in this Appendix.



● **PEAK STRENGTH**
Friction Angle= 48 degrees
Cohesion= 42 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 30 degrees
Cohesion= 414 psf

Sample Location		Classification	DD,pcf	MC,%
B-1	10.0	Bedrock	92	22.6

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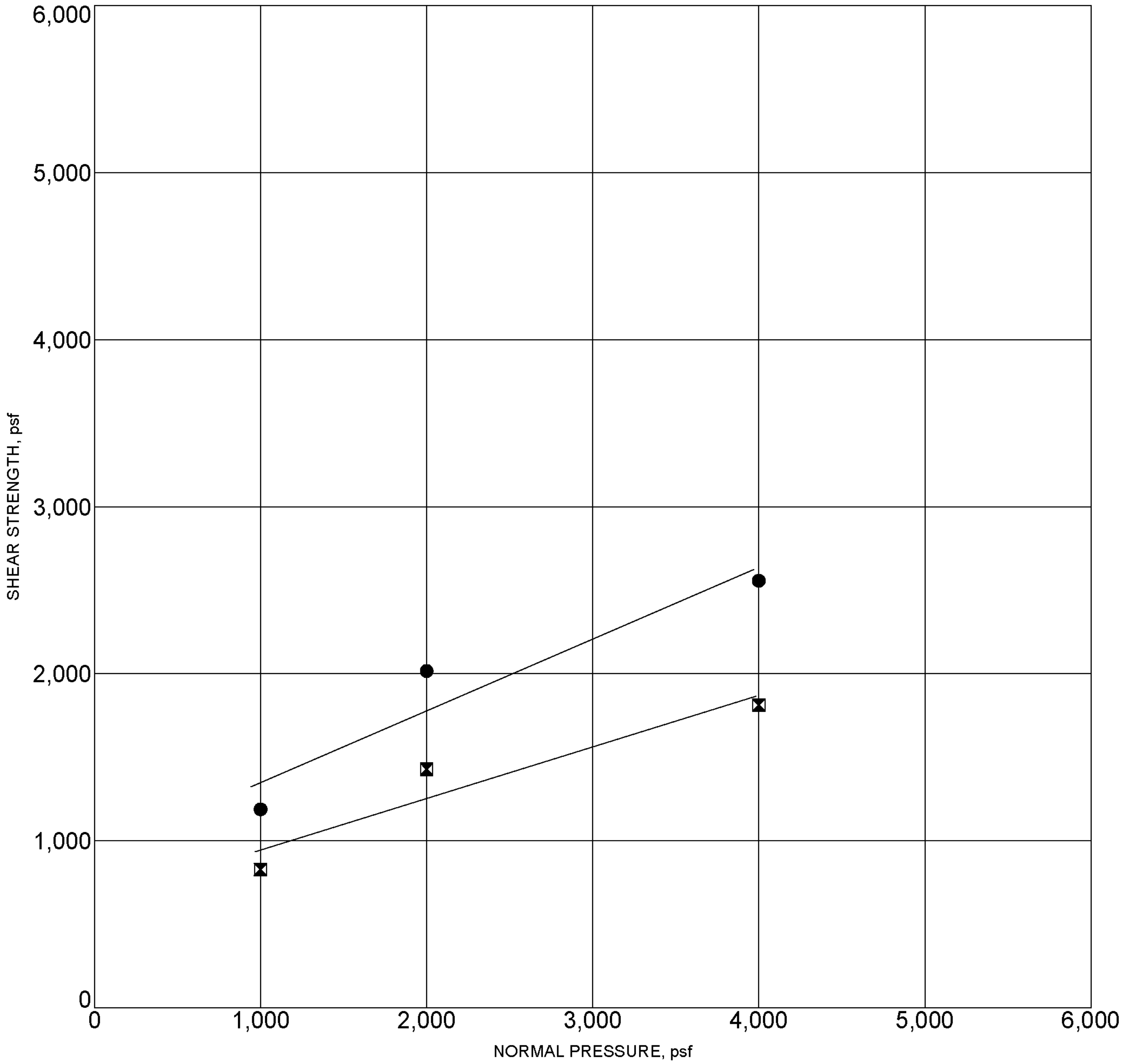
DIRECT SHEAR TEST RESULTS

FIGURE B-1



● **PEAK STRENGTH**
Friction Angle= 23 degrees
Cohesion= 918 psf

⊠ **ULTIMATE STRENGTH**
Friction Angle= 17 degrees
Cohesion= 636 psf

Sample Location		Classification	DD,pcf	MC,%
B-1	20.0	Bedrock	100	23.3

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DIRECT SHEAR TEST RESULTS

FIGURE B-2

Table 1 - Laboratory Tests on Soil Samples

Geotechnical Professionals, Inc.
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Your #2914.I, HDR Lab #18-0820LAB
9-Jan-19

Sample ID				
			B-1 @ 35'	B-2 @ 0-5'
Resistivity		**Units**		
as-received		ohm-cm	2,960	1,880
saturated		ohm-cm	880	920
pH			7.7	7.6
Electrical Conductivity		mS/cm	0.35	0.45
Chemical Analyses				
Cations				
calcium	Ca^{2+}	mg/kg	105	326
magnesium	Mg^{2+}	mg/kg	24	12
sodium	Na^{1+}	mg/kg	188	72
potassium	K^{1+}	mg/kg	50	80
Anions				
carbonate	CO_3^{2-}	mg/kg	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	372	275
fluoride	F^{1-}	mg/kg	11	30
chloride	Cl^{1-}	mg/kg	25	4.0
sulfate	SO_4^{2-}	mg/kg	426	880
phosphate	PO_4^{3-}	mg/kg	1.1	ND
Other Tests				
ammonium	NH_4^{1+}	mg/kg	1.6	2.2
nitrate	NO_3^{1-}	mg/kg	12	5.5
sulfide	S^{2-}	qual	na	na
Redox		mV	na	na

Resistivity per ASTM G187, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.
Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed