Appendix C
Geotechnical Investigation

GEOTECHNICAL
PROFESSIONALS INC.

GEOTECHNICAL INVESTIGATION
PROPOSED INDUSTRIAL DEVELOPMENT
14321 AND 14351 MYFORD ROAD (MYFORD II)
TUSTIN, CALIFORNIA

Prepared for:
B8 Myford II Industrial Owner LLC
c/o Panattoni Development Co. Inc.
2442 Dupont Drive
Irvine, California 92612

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Project No. 3078.I

August 10, 2022

GEOTECHNICAL PROFESSIONALS INC.

August 10, 2022

B8 Myford II Industrial Owner LLC
c/o Panattoni Development Co. Inc.
2442 Dupont Drive
Irvine, California 92612

Attention: Michael Sizemore
Development Manager

Subject: Report of Geotechnical Investigation
Proposed Industrial Development
14321 and 14351 Myford Road (Myford II)
Tustin, California
GPI Project No. 3078.I

Dear Michael:

Transmitted herewith is an electronic copy of 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.

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. With remedial earthwork to overexcavate and recompact the existing fill soils and a portion of the upper natural soils, the proposed building can be supported on conventional shallow footings. The upper clay soils are very moist to wet, compressible, have a high potential for expansion, and we discuss remedial measures in the report for support of the foundations, floor slab, and hardscape. The moisture content of the upper soils is high, and wet soils are anticipated that will require drying prior to reuse as compacted fill. Subgrade stabilization may be required locally or in wet weather periods, but our experience at the adjacent site indicates that the in-place soils can support smaller rubber tire scrapers without yielding. Based on the shallow groundwater and clay soils, stormwater infiltration will not be feasible.

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.

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3078-I-01L (08/22)

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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 industrial development project at 14321 and 14351 Myford Road in Tustin, California. The geographical site location is shown on the Site Location Map, Figure 1. GPI conducted a site investigation and is providing field observations and testing during construction for the industrial development to the south of the site at 14451 Myford Road (GPI, 2021).

1.2 PROJECT DESCRIPTION

The proposed development is about 7.18 acres and will include a one-story, approximately 148,437 square-foot industrial building (both office and warehouse space) with associated parking and drives. The building will likely be of tilt-up concrete construction with a concrete slab-on-grade floor and truck loading docks on the north and south sides of the building. Structural loads were not available at the time this report was prepared. Based on our experience with similar projects, structural loads are anticipated to be up to 100 kips (columns) and 8 kips per lineal foot (walls).

The site plan (Scheme 13, dated August 9, 2022 by HPA Architecture) shows the proposed warehouse building located at the center of the site, with truck loading docks located on the west side of the building. Vehicle surface parking will be located at the east side of the site adjacent to Myford Road.

Grading plans are not yet available. We anticipate grades to be up to 3 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 existing and proposed site configurations are shown on the Existing and Proposed Site Plans, Figures 2 and 3, respectively.

Our recommendations are based upon the above structural and grading information. We should be notified if the actual loads and/or grades change during the project design to either confirm or modify our recommendations.

1.3 PURPOSE OF INVESTIGATION

The primary purpose of this investigation and report is to provide an evaluation of the existing geotechnical and seismic 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 for this investigation consisted of a review of historical aerial photographs, geotechnical field exploration, laboratory testing, engineering analysis, and the preparation of this report.

Our geotechnical field exploration program consisted of five Cone Penetration Tests (CPT's) and six exploratory borings. The CPT's were advanced to depths ranging from 44 to 80 feet below existing site grades (CPT C-3 refused prematurely at a depth of 44 feet on dense subsurface soils, prior to the planned depth). The exploratory borings were drilled to depths of 21½ to 51½ feet below existing site grades. Details of the field procedures and logs of the CPT's and borings are presented in Appendices A and B, respectively. The locations of the subsurface explorations are shown on Figures 2 and 3.

Laboratory tests were performed on selected representative soil samples as an aid in soil classification and to evaluate the engineering properties of the soils. The geotechnical laboratory testing included determinations of moisture content and dry density, Atterberg Limits, shear strength (direct shear), consolidation, expansion index, compaction (maximum density/optimum moisture), and soil corrosivity. Laboratory testing procedures and results are summarized in Appendix C.

Soil corrosivity testing was performed by HDR under subcontract to GPI. Their test results are presented in Appendix C.

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

3.0 SITE CONDITIONS

3.1 SITE HISTORY

Our understanding of the development history of the site is based on a review of historical aerial photographs (Historic Aerials). Up to at least 1972, the subject site was undeveloped and likely used for agricultural purposes. By 1980, the buildings at each site are in-place, and the site remained relatively unchanged since.

3.2 SURFACE CONDITIONS

The site is bounded on the east by Myford Road, and existing industrial developments on the north, south, and west. The site to the south is under construction for Panattoni (Myford I). The existing site conditions are shown on the Existing Site Plan, Figure 2.

The existing site grades range from a high of about Elevation +70 feet at the northern property boundary to a low of about +64 feet at the southwestern property boundary. Overall, the site is relatively flat, with slopes typically less than or equal to 1 percent.

The pavement sections at our boring locations varied from about 4 to 6 inches of asphalt concrete over 4 to 6 inches of aggregate base, with two locations (west side of the existing northern building) consisting of 7 inches of portland cement concrete over 3 to 6 inches of aggregate base. The existing pavement is generally in good condition, with limited surficial cracking in the asphalt parking and drive areas. In the PCC pavement areas (north and west side of the northern building), we observed the pavement to be relatively new and in good condition.

3.3 SUBSURFACE SOILS

Our field investigation disclosed a subsurface profile consisting of undocumented fill overlying natural soils. Detailed descriptions of the subsurface conditions encountered are shown on the Logs of CPT’s and Borings in Appendices A and B, respectively.

Fill soils, to depths of approximately 2 to 3 feet, were encountered in our exploratory borings. The undocumented fills consisted predominantly of silty clays with trace amounts of sand. We anticipate deeper undocumented fills may be encountered during grading within the limits of the existing buildings. The clayey fills were very moist to wet, with moisture contents ranging from 19 to 39 percent (roughly 3 to 23 percent above optimum).

The underlying natural soils consisted predominantly of firm to very stiff clays and silts with few deposits of sandy silts and sands with thin lenses of silty sands and sands throughout the profiles. We encountered significant calcium carbonate deposits in the natural fine grained soils. The moisture content of the soils within 10 feet of the ground surface is predominantly very moist to wet, varying from 13 to 41 percent, with an average moisture content of about 28 percent, approximately 12 percent above the optimum moisture content. The upper natural materials exhibit low to moderate strength and moderate to high compressibility characteristics.

Our laboratory tests indicate the upper fill and natural soils to have medium to high potential for expansion (EIs ranging from 65 to 102). At an adjacent site, the expansion index of the upper soils was measured as high as 110 (very high). As such, the clays are anticipated to shrink and swell with changes in moisture content.

3.4 GROUNDWATER AND CAVING

Groundwater was encountered between depths of 20 to 25 feet in our recent explorations. We encountered groundwater at depths of 11 and 19 feet below the ground surface in our exploratory borings performed at the adjacent site to the south in 2019. Historical data provided by the California Geologic Survey (CGS) indicates that the shallowest depth to groundwater is approximately 12 to 13 feet below existing grades in the site vicinity (CGS, 1998).

Caving was not encountered in our 8-inch diameter hollow-stem borings. Based on the fines and moisture contents of the soils encountered, the caving potential of the upper soils is considered to be low, with the exception of the soils beneath the groundwater.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 GENERAL

Based on the results of our investigation, it is our opinion that from a geotechnical viewpoint it is feasible to develop the site as proposed. The proposed structure can be supported on shallow foundations following remedial grading to mitigate the geotechnical constraints discussed below. The most significant geotechnical issues that will affect the design and construction of the proposed structure are as follows:

- The undocumented fill and upper compressible natural soils are not considered to be suitable for uniform support of new foundations or floor slabs. We recommend the existing fill and upper compressible soils be removed and replaced as properly compacted fill. Details are presented in the "Earthwork" section of this report.

- Moisture contents of the near surface soils (immediately beneath the existing pavements to within 10 feet of the existing grades) are predominantly very moist to wet, averaging about 13 percent above the optimum moisture content. Therefore, active discing in favorable weather to dry these materials prior to placement as fill or backfill should be expected. In addition, the exposed subgrade soils will be well over the optimum moisture content. Although the in-place moisture contents would indicate that subgrade stabilization would be required to support rubber-tire compaction equipment, our experience at the adjacent site indicates that the in-place soils can support smaller rubber tire paddlewheel scrapers without stabilization. The earthwork contractor should evaluate the moisture content of the existing soils when planning the earthwork.

- The on-site soils consist predominantly of highly expansive clays. Retaining wall backfill and the soils placed within the upper 24 inches (18 inches if cement treated) of the finished subgrade in floor slab and within the upper 18 inches of finished subgrade in hardscape areas should consist of cement-treated on-site soils or granular, non-expansive import fills. If the site is designed to balance or has export, we anticipate cement-treatment of the building pad and hardscape subgrade surface is the more feasible mitigation option rather than importing non-expansive select fill to "cap" the building pad. Crushed aggregate base created from the site demolition may be used as select fill.

- Because of the elevated in-place moisture content of the upper site soils, earthwork operations in the rainy season will be difficult (high cost and low production). With favorable weather, we anticipate active mechanical drying using earthwork equipment such as a disc will be a feasible option to lower the soil moisture content. In the rainy season, we would anticipate significantly longer drying times or the need for drying with cement treatment.

- Static settlements of the proposed building were evaluated with respect to the anticipated foundation loads (maximum 100-kip column loads) and the placement of additional fill to raise the building pad above the existing grades (approximately 2 to 3 feet). For foundations, we estimate approximately 1-inch of static settlement under the more heavily loaded columns. When raising grades above the existing grade, we estimate an approximate settlement of ⅓-inch per foot of grade increase (i.e., raising grades by 3 feet will result in about 1 inch of areal settlement). Additional details are presented in the “Foundations” section of this report.

- Given the above settlement information, we recommend reinforcing the building floor slab. Also, we recommend that the placement of concrete within the pour strips along the wall footings and diamond block-outs around the columns be delayed as long as possible to allow settlement to occur before connecting the slab to walls or columns.

- The site is located in a seismic hazard zone for soil liquefaction. Based on our analyses, we computed potential seismic-induced liquefaction settlements of about ¼ to ½ inch. Dry seismic subsidence, unrelated to liquefaction, is estimated to be negligible due to the shallow groundwater depth and presence of clays.

- The upper very moist to wet, expansive clays will provide poor support for pavements, resulting in relatively thick sections and the need to underlay concrete with aggregate base. As an alternative, the pavement subgrade can be cement-treated to improve stability and allow thinner pavement sections.

- Corrosivity testing was performed on a representative sample obtained from our borings indicate a negligible level of soluble sulfate and chloride content with respect to concrete. Resistivity testing of the soils indicated they were considered to be corrosive to buried ferrous metals.

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 CONSIDERATIONS

4.2.1 General

The site is located 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 internet website (https://seismicmaps.org/), the corresponding seismic design parameters from the CBC are as follows:

$S_S = 1.268g$	$S_{MS} = F_a * S_S = 1.268g$	$S_{DS} = 2/3 * S_{MS} = 0.845g$
$S_1 = 0.454g$	$S_{M1} = F_V * S_1 = 0.840g$	$S_{D1} = 2/3 * S_{M1} = 0.560g$

In accordance with the 2019 CBC, site-specific response spectra are required for structures located in a Site Class D (with S_1 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 $T \leq 1.5\ T_{S,}$
- 1.5 times the value computed by Equation 12.8-3 for values of $T_L \geq T > 1.5\ T_S$, or
- 1.5 times the value computed by Equation 12.8-4 for values of $T > T_L$.

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. Based on the mapped seismic parameters, the T_S period is approximately 0.66 seconds (therefore $1.5{*}T_S$ is approximately 0.99 seconds).

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 (earthquake.usgs.gov), the most significant fault in the proximity of the site is the San Joaquin Hills Fault, which is located about 2¼ miles from the subject 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 OSHPD website (https://seismicmaps.org/), we computed that the site could be subjected to a peak ground acceleration (PGA_M) of 0.583g for a magnitude 7.2 earthquake (San Joaquin Hills). This acceleration has been computed using the mapped Maximum Considered Geometric Mean peak ground acceleration from ASCE 7-16 (ASCE, 2017) 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

The site is not located within an Alquist-Priolo Special Studies Zone and there are no known faults crossing or projecting toward the site. Therefore, ground rupture due to faulting is considered unlikely at this site.

4.2.4 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 by the State of California as having a potential for soil liquefaction (CGS, 1999). Groundwater was encountered as shallow as 20 feet in our most recent explorations, and as shallow as 11 feet in our recent exploratory borings for the adjacent site to the south. Historical groundwater levels at the site are estimated to be approximately 12 to 13 feet below the ground surface at the site. A groundwater depth of 11 feet was used in our evaluation.

Revisions to the 2019 California Building Code (CBC), ASCE 7-16, and Special Publication 117A (CGS, 2008) require that the ground motion used for this evaluation be based on the Peak Ground Acceleration (PGA_M) adjusted for site class effects. This value is computed using the mapped Maximum Considered Geometric Mean (MCE_G) peak ground acceleration for a Site Class D and a site coefficient, F_{PGA}. In accordance with the 2019 CBC, we considered a ground acceleration of 0.58g for a magnitude 7.2 earthquake (San Joaquin Hills) for our analyses, which corresponds to the PGA_M obtained using the methods described above.

The potential for liquefaction was evaluated using the methods presented by the NCEER (Youd, 1997) and Robertson (Robertson, 2009), with modifications provided in Special Publication 117A. Criterion for liquefaction susceptibility of the fine-grained soils was based on methods presented in Bray and Sancio (2006).

The materials encountered below the shallow groundwater depth of 11 feet consisted primarily of firm to very stiff clays with minor layers of medium dense to dense sands and silty sands.

Based on our evaluation, we estimate a potential for liquefaction induced settlement on the order of ¼ to ½ inch. The differential (across a 60-foot span) liquefaction-induced settlement is anticipated to be less than ¼ inch (angular distortion of 1/2880). The differential settlement across other spans can be extrapolated from the information provided.

4.2.5 Seismic Ground Subsidence

Seismic ground subsidence (not related to liquefaction induced settlements), occurs when loose, granular (sandy) soils above the groundwater are densified during strong earthquake shaking. The subsurface soils above the groundwater table were generally very moist to wet, firm to stiff clays and silts. If strong earthquake shaking occurs, the seismic ground subsidence (not related to liquefaction) is not anticipated to impact the site.

4.3 EARTHWORK

The earthwork anticipated at the project site will consist of clearing and grubbing, excavation of undocumented fills, loose natural soils and disturbed soils, subgrade preparation and stabilization, moisture conditioning of fill soils, and the placement and compaction of fill.

Earthwork operations in the rainy season will be difficult (high cost and low production) due to the elevated in-place moisture content of the upper site soils. With favorable weather, we anticipate active mechanical drying using earthwork equipment such as a disc may be a feasible option to lower the soil moisture content. In the rainy season, we would anticipate significantly longer drying times or the need for drying with cement treatment. Cement treatment may be considered as a potential alternative to mechanical drying regardless of weather conditions.

4.3.1 Clearing and Grubbing

Prior to grading, the areas to be developed should be stripped of vegetation and cleared of debris. Buried obstructions, such as footings, utilities, and tree roots should be removed. Deleterious material generated during the clearing operation should be removed from the site. The pavement sections across the site were relatively thick at our boring locations (average of 4¾ inches AC over 4½ inches AB) and demolition of the existing building will result in concrete debris. 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.

If deeper excavations are required for the demolition of the existing development, they should be left open, documented with respect to depth and location, or backfilled under the observation of a GPI representative.

Although not encountered during our investigation, leach lines, cesspools or septic systems encountered during grading 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.

At the conclusion of the clearing operations, a GPI representative should observe and accept the site prior to further grading.

4.3.2 Excavations

Excavations at this site will include removal of undocumented fill soils and upper compressible natural soils, footing excavations, and trenching for proposed utility lines.

Prior to placing fills or construction of the proposed building, undocumented fill and compressible natural soils occurring within the proposed building pad area should be removed and replaced as properly compacted fill. For planning purposes, removals for the building foundations should extend to depths of at least 5 feet below existing grades or 3 feet below the base of the planned foundations, whichever is deeper. Localized deeper removals may be required where deeper fills are encountered. Based on the provided preliminary plans, we anticipate that sufficient space is available for open-cut excavations.

For minor structures such as site walls, removals should extend at least 3 feet below the existing grade or 2 feet below the base of foundations, whichever is deeper. In proposed pavement and hardscape areas, the existing near-surface soils should be removed to a depth of 1 foot below existing grades or finished subgrade, whichever is deeper, moisture-conditioned (dried), and replaced as properly compacted fill. Deeper removals may be required where deeper disturbance of the soils is caused in demolishing the existing building.

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

The corners of the areas to be overexcavated should be accurately staked in the field by the Project Surveyor. The base of the excavations should extend laterally at least 5 feet beyond the outside edge of the perimeter foundations or a minimum distance equal to the depth of overexcavation/compaction below finish grade (i.e., a 1:1 projection below the top edge of footings), whichever is greater. This includes the footprint of the building and other foundation supported improvements, such as loading dock and site walls, trash enclosures, and canopies.

Where not removed by the aforementioned excavations, existing utility trench backfill within building areas should be removed and replaced as properly compacted fill. The limits of removal should 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 4 feet below adjacent grade. For cuts up to 12 feet deep, the entire cut should be properly shored or sloped back to at least 1:1 or flatter. For cuts deeper than 12 feet, dewatering will likely be required, and the entire slope should be properly shored or sloped back at least 1½:1 (horizontal to vertical) or flatter. Some raveling of the localized sandy deposits should be anticipated at the slope inclinations recommended. If raveling cannot be tolerated, flatter slope inclinations should be considered. The exposed slope face should be kept moist (but not saturated) during construction to reduce local sloughing.

In areas where removals are performed adjacent to property lines, existing streets, or other improvements where temporary slopes are not feasible, “ABC” slot cuts may be utilized instead of shoring. The slots should be no wider than 7 feet and no deeper than 8 feet, and should be backfilled immediately to finish grade prior to excavation of the adjacent two slots on each side. Where the localized sand and gravel deposits are encountered, narrower slots may be required. We should review the plans for excavation adjacent to property lines and existing improvements when they are developed.

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.

4.3.3 Subgrade Preparation

The subgrade soils exposed at the base of the overexcavation will consist of wet (over-optimum) clays. Although the in-place moisture contents would indicate that subgrade stabilization would be required to support rubber-tire compaction equipment, our experience at the adjacent site indicates that the in-place soils can support smaller rubber tire paddlewheel scrapers without stabilization. The earthwork contractor should evaluate the moisture content of the existing soils when planning the earthwork.

Although not anticipated based on our adjacent site experience, if stabilization is required to achieve a firm and unyielding bottom to support the fill placement, stabilization can consist of either 18 inches of cement treatment, or 18 inches of crushed miscellaneous base (CMB) over a geogrid such as Tensar BX1100, or equivalent. Within the building pad, the stabilization method used should be consistent to provide uniformity in support of the foundations and floor slab. Outside the building pad, such as in pavement areas, the stabilization can consist of cement treatment and/or placement of aggregate base/geogrid.

For clarification, the cement treatment or CMB/geogrid stabilization options described above are in addition to the overexcavation depths detailed in Section 4.3.2 and are not considered a replacement for the recommended depths of properly compacted fill.

4.3.4 Material for Fill

The on-site soils are, in general, suitable for use as compacted fill. As previously noted, the on-site clays are highly expansive (EI of up to 102) and are expected to shrink and swell with changes in moisture content. As such, the on-site clays are not suitable for placement beneath floor slabs or hardscape or used as retaining wall backfill. Based on the tested expansion index, we recommend either the soils within 24 inches of the finished subgrade in floor slab consist of select non-expansive soils as detailed below (including crushed inert debris generated from demolishing the existing building and pavements), or 18 inches of cement-treated on-site soils. Soils placed within 18 inches of hardscape subgrade should consist of either cement-treated on-site soils or select non-expansive soils. Retaining wall backfill should consist of imported select, granular soils. Based on our subsurface explorations, suitable granular material is not anticipated to be available within the anticipated over excavation depths.

The over-optimum clayey soils removed during overexcavation will need to be dried mechanically using active discing during warm, dry weather.

Imported fill material should be predominately granular (contain no more than 40 percent fines - portion passing No. 200 sieve) and non-expansive (E.I. less than 20). The import should also exhibit an R-value of at least 30 if used in proposed paved areas. GPI should be provided with a sample (at least 50 pounds) and notified of the location of soils proposed for import at least 72 hours prior to importing. Each proposed import source should be sampled, tested and accepted for use prior to delivery of the soils to the site. Soils imported prior to acceptance by GPI may be rejected if not suitable.

Both imported and existing on-site soils to be used as fill should be free of debris and pieces larger than 6 inches in greatest dimension.

If on-site concrete or asphalt concrete are crushed/pulverized to re-use as aggregate base for stabilizing wet, yielding subgrade, it should be crushed to maximum particle size of 3 inches. If used to support pavements, it should be crushed to meet the specifications of Class II or crushed miscellaneous base (CMB). If mixing with the on-site soils, we recommend the mixture consist of about 50 percent aggregate base and be well blended with the on-site soils using a disc or equivalent equipment.

4.3.5 Placement and Compaction of Fills

Fill soils should be placed in horizontal lifts, moisture-conditioned, and mechanically compacted to at least 90 percent of the maximum dry density in accordance with ASTM D 1557. Granular soils (if used) within the upper 1 foot of the pavement subgrade should be compacted to at least 95 percent. The optimum lift thickness will depend on the compaction equipment used and can best be determined in the field. The following uncompacted lift thickness can be used as preliminary guidelines.

Plate compactors	4-6 inches
Small vibratory or static rollers (5-ton)	6-8 inches
Scrapers, heavy loaders, and large 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.

The moisture content of the existing near surface soils is well above the optimum moisture content. As such, the soils will require mixing and moisture conditioning (drying) prior to placement as properly compacted fill. With clear and dry weather conditions, it may be feasible to use a disc to mechanically dry the over-optimum soils. During the rainy season, significantly longer drying times will be required (high cost/low production). The moisture content of the clayey fill materials should be at least 3 percent over optimum moisture conditions at the time of compaction (1 to 3 percent over optimum for granular soils where placed).

During backfill of excavations, the fill should be properly benched into the construction slopes as it is placed in lifts. The moisture content of properly compacted soils should be maintained prior to covering or reprocessed and moisture conditioned immediately prior to covering.

Because of the expansiveness of the on-site soils, it will be important to maintain the elevated moisture contents in the compacted fill prior to covering. Soils that are allowed to dry out will require moisture conditioning that is likely to include reprocessing to introduce the moisture.

4.3.6 Cement Treatment

In areas where cement treatment of the soils is performed (subgrade stabilization, building pad subgrade, or pavement areas), we recommend the use of 5 percent cement by unit weight of soil (assume 125 pcf). The cement should be distributed across the soil using an appropriate mechanical spreader equipped with a self-contained vacuum system capable of capturing dust fines to minimize the spread of the cement outside the treated area. The cement, soil, and water should be mixed a minimum of two times using a four-wheel drive rotary mixer that can introduce water during mixing through a metering device. We recommend the moisture content of the cement-treated subgrade be within one percentage point below or two percentage points above the optimum content to ensure proper chemical action.

For the building pad subgrade or in pavement areas, the initial compaction of the cement-treated subgrade should be performed using a RexPactor 3-70 or equivalent, followed by final compaction using steel drum rollers. The final compaction should be performed within 2½ hours after the initial application of water during mixing. The cement-treated soils should be compacted to densities of at least 90 percent of the maximum dry density determined in accordance with ASTM D 1557. Following cement-treatment and compaction, construction traffic should be kept off of the subgrade for at least 72 hours, with the exception of maintaining a wet surface during the curing period.

We recommend the treatment should be performed by a subcontractor experienced with in-place cement treatment. Based on prior experience with similar projects, the cement-treatment specialty contractor should have performed a minimum of five comparable cement stabilization projects within the last 2 years.

4.3.7 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 the upper soils, an average shrinkage value of 15 to 20 percent and a subsidence of 0.2 feet may be assumed. 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. The upper soils within the existing building pads may have been previously compacted, resulting in lower shrinkage values.

4.3.8 Trench/Wall Backfill

Utility trench and wall backfill should be mechanically compacted in lifts. The on-site clays are not suitable for use as retaining wall backfill. In addition, the moisture content of the upper clays is very moist to wet, and the soils will require drying before reusing as properly compacted fill. Lift thickness should not exceed those values given in the "Placement and Compaction of Fills" section of this report. Jetting or flooding of backfill materials should not be permitted. A representative of GPI should observe and test all 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. Slurry should also be used as backfill within the pipe zone for utilities that extend adjacent to and below building foundations. The slurry should contain 1½ sacks 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.

4.3.9 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 any additional fill.

4.4 FOUNDATIONS

4.4.1 Foundation Type

The proposed structures may be supported on conventional isolated and/or continuous shallow footings, provided the subsurface soils are prepared in accordance with the recommendations given in this report. Footings should be supported on properly compacted fill.

4.4.2 Allowable Bearing Pressures

Based on the shear strength and elastic settlement characteristics of the recompacted on-site soils, a static allowable net bearing pressure of 2,500 pounds per square foot (psf) may be used for both continuous footings and isolated column building footings. These bearing pressures are for dead-plus-live-loads, and may be increased one-third for short-term, transient, wind and seismic loading. The actual bearing pressure used may be less than the value presented above and can be based on economics and structural loads to determine the minimum width for footings as discussed below. The maximum edge pressures induced by eccentric loading or overturning moments should not be allowed to exceed these recommended values.

For minor structures, such as site walls and property line screen walls, where reduced excavation depths have been recommended 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)
2,500	48	24
2,250	36	24
2,000	24	24
1,500	15	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 of 15 inches should be used even if the actual bearing pressure is less than 1,500 psf. Because of the expansive nature of the on-site soils, we recommend a minimum footing depth of 18 inches.

4.4.4 Estimated Settlements

For the building foundation loads anticipated, total static settlement of the heavily loaded column footings (100 kips) is expected to be on the order of 1-inch. Maximum differential settlements between similarly loaded adjacent footings or along a 60-foot span of a continuous footing are expected to be about ½-inch (angular distortion of 1/1440). The differential settlement across other distances can be extrapolated based on the above recommendations.

Conceptual grading plans are not available at the time of this report. Due to soft, compressible soils at depth, if grades are going to be raised, we estimate an approximate settlement of ⅓-inch per foot of grade increase.

Potential seismic settlements presented in a previous section of this report should be added to these values when considering total settlements.

The above estimates assume that the recommended earthwork will be performed and that the footings will be sized in accordance with our recommendations.

Given the above anticipated settlements, we recommend that placement of concrete within the pour strips and diamond block-outs be delayed as long as possible (at least 30 days after loads are applied) to allow settlements to occur before connecting the floor slab to walls or columns.

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 0.27 may be used for design. In addition, an allowable lateral bearing pressure equal to an equivalent fluid weight of 250 pounds per cubic foot may be used, provided the footings are poured tight against compacted fill soils.

For structures where lateral limits of the overexcavation may be limited, we recommend an allowable lateral bearing pressure equal to an equivalent fluid weight of 200 pounds per cubic foot be used. These values may be used in combination without reduction.

4.4.6 Foundation Concrete

Based on laboratory testing by HDR (Appendix C) soluble sulfate contents of a representative sample of the on-site soils were 615 mg/kg. For the 2019 CBC, foundation concrete should conform to the requirements outlined by ACI 318, Section 19.3 for a negligible level of soluble sulfate exposure for soil (Category S0). The chloride content of the sample tested was 41 mg/kg, which is considered to be low (Category C1).

4.4.7 Footing Excavation Observation

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

4.5 BUILDING FLOOR SLABS

Slab-on-grade floors should be supported on either 24 inches of granular (sandy) non-expansive, compacted soils or 18 inches of cement treated on-site soils as discussed in the “Material for Fill” section. The moisture content of the upper 24 inches of the compacted subgrade should be maintained at or above the optimum moisture content until covering.

The finished pad subgrade will either be cement-treated or consist of imported select soils and, as such, the potential of expansion of the finished subgrade will be minimal. We understand the floor slab is currently proposed to be a 7- or 8-inch thick and constructed with 4,000 psi concrete reinforced with #3 rebar spaced at 16 inches on center each way. From a geotechnical perspective, we take no exception to the floor slab design.

For elastic design of slabs-on-grade supporting sustained concentrated loads, a modulus of subgrade reaction (k) of 150 pounds per cubic inch (pounds per square inch per inch of deflection) may be used for imported granular soils. If the upper soils within the building pad are cement treated and properly compacted, a modulus of subgrade reaction (k) of 300 pounds per cubic inch may be used. The structural design should consider both long-term loads related to building operations and short-term construction 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. We take no exception if a sand layer is used above or below the vapor barrier as it is not a geotechnical issue, and is a decision of 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 at pipe penetrations), as well as excess moisture in the concrete. The manufacturer of the floor coverings should be consulted for establishing acceptable criteria for the condition of floor surface prior to placing moisture-sensitive floor coverings.

4.6 RETAINING STRUCTURES AND SHORING

The following recommendations are provided for retaining walls less than 6 feet in retained height. We recommend that walls be backfilled with non-expansive (Expansion Index of 20 of less) granular (no more than 40 percent passing No. 200 U.S. standard sieve) soils. Based on our subsurface explorations, such material is not anticipated to be available within the upper 10 feet below existing grades. We anticipate import of select materials to be required, and the need for such materials should be noted on the project plans.

4.6.1 Retaining Walls

Active earth pressures can be used for designing walls that can yield at least ½-inch laterally in 10 feet of wall height under the imposed loads. For level backfill comprised of on-site or imported granular soils, the magnitude of active pressures is equivalent to the pressures imposed by a fluid weighing 35 pounds per cubic foot (pcf). This pressure may also be used for the design of temporary excavation support.

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 55 pounds per cubic foot should be used for drained granular backfill.

A seismic lateral earth pressure should be used for the design of retaining walls supporting 6 feet or more of backfill. We recommend a total lateral earth pressure of 55 pcf (active plus seismic). If walls are designed using the above at-rest pressure, total (static plus seismic) lateral earth pressure may be limited to that value.

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 retaining walls adjacent to streets should be designed to resist a uniform lateral pressure of 100 pounds per square foot (psf), acting as a result of an assumed 300 psf 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.

The wall backfill should be well-drained to relieve possible hydrostatic pressure or designed to withstand these pressures. 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 base. In addition, “rock pockets” can be installed at the base of the shoring 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.

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. Earthwork associated with the site walls should be performed as discussed in the “Earthwork” section.

4.6.2 Temporary Shoring

Where there is not sufficient space for sloped embankments, such as along the property limits or adjacent to existing structures, shoring will be required. One method of shoring would consist of steel soldier piles placed in drilled holes and backfilled with concrete. Driven or vibrated soldier piles may also be more economical alternative to drilled holes, and they can be used for supporting cuts that do not support existing structures. The presence of groundwater should be considered when evaluating the alternatives.

For cantilever shoring up to 10 feet in height with level backfill, the magnitude of active pressure is equivalent to the pressures imposed by a fluid weighing 35 pounds per cubic foot (pcf). The active pressure should be applied to the full embedment depth of the cantilever shoring.

In addition to the recommended earth pressure, the shoring should be designed for surcharge loads due the adjacent structures and construction traffic surcharge loads. 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. Existing adjacent structures will impart a surcharge load on shoring. The location and depth of the adjacent building footings, as well as the loading, will need to be determined to estimate the surcharge pressure on the shoring.

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 500 pounds per square foot at the excavated surface, up to a maximum of 5,000 psf. These values can also be used to design a pile supported retaining wall, which we understand is being considered along the northern property line. 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.

The shoring contractor should evaluate the potential drilling conditions when planning the installation methods.

Driven or vibrated soldier piles may be a feasible and more economical alternative. If soldier piles are vibrated or driven, predrilling should not be allowed below the planned excavation level. Predrilling should be performed with a continuous flight auger capable of reversing the auger to minimize the removal of soil during the process. The diameter used for predrilling should not exceed 80 percent of the maximum depth of the soldier pile section. For design, the width of the driven or vibrated pile should be taken as the width of the flange.

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.

4.7 CORROSIVITY

A representative sample of the site’s soils indicates that they are 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.

4.8 DRAINAGE

Positive surface gradients should be provided adjacent to structures 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. We recommend that landscape planters be avoided immediately adjacent to the building. If planters are required, they should be provided with surface drains and planted with drought tolerant plants to reduce the potential for the infiltration of surface water beneath the building foundations and floor slab.

4.9 EXTERIOR CONCRETE AND MASONRY FLATWORK

The on-site clays have a high expansion index and are anticipated to shrink and swell in changes in moisture content. Exterior concrete and masonry flatwork should be supported on at least 18 inches of non-expansive, compacted fill or cement-treated treated soils. Prior to placement of concrete, the subgrade should be prepared as recommended in "Subgrade Preparation" section. The use of untreated clayey soils in the slab-subgrade should not be permitted.

We recommend the Project Civil Engineer design the concrete hardscape and sidewalks, including determination of thickness and reinforcing. For exterior flatwork, hardscape, and sidewalks, we recommend a minimum slab thickness of 4 inches with minimum slab reinforcement of No. 3 rebar placed at 16 inches on-center, in both directions. Control joints to direct shrinkage cracking in exterior slabs and sidewalks should be provided at maximum spacing of 8 and 6 feet on center in two directions, respectively. Where flatwork is planned at building entries, we recommend it be structurally connected to the perimeter building foundation to reduce the potential differential settlement at this joint. These recommendations should be considered as minimums based on the geotechnical site conditions, and the Project Civil Engineer should confirm if more stringent recommendations are needed for other purposes.

4.10 PAVED AREAS

Based on the soils encountered in our explorations and R-value testing of the onsite soils at the adjacent site, we used an R-value of 10 for design to allow for some variability in the exposed conditions. The California Division of Highways Design Method was used for design of the recommended preliminary pavement sections. These recommendations assume that the pavement subgrades will consist of existing near surface soils. The following pavement sections are recommended for typical distribution center traffic uses and untreated, on-site clay subgrade:

UNTREATED SUBGRADE SOIL

PAVEMENT AREA	TRAFFIC INDEX	SECTION THICKNESS (inches)	
		ASPHALT/PORTLAND CONCRETE	AGGREGATE BASE COURSE
Asphalt Concrete			
Automobile Parking	4.0	3.0	7.0
Automobile Drives	5.5	3.5	10.0
Truck Drives	7.0	4.0	15.0
Heavy Truck Apron	8.0	5.0	16.0
Portland Cement Concrete			
Automobile Parking	4.0	7.0	4.0
Automobile Drives	5.5	7.5	4.0
Truck Drives/Loading Dock Apron	7.0	8.0	4.0
Heavy Truck Apron	8.0	8.5	4.0

To improve the pavement support characteristics of the on-site clays resulting in thinner pavement sections, cement treatment can be performed. We recommend that cement treatment in pavement areas consist of a treatment depth of at least 12 inches and 5 percent cement by unit-weight of soil (assumed 125 pcf). The following pavement sections are recommended for typical distribution center traffic uses and cement-treated subgrade soil:

CEMENT-TREATED SUBGRADE SOIL

PAVEMENT AREA	TRAFFIC INDEX	SECTION THICKNESS (inches)	
		ASPHALT/PORTLAND CONCRETE	AGGREGATE BASE COURSE
Asphalt Concrete			
Automobile Parking	4.0	3.0	4.0
Automobile Drives	5.5	3.0	6.0
Truck Drives	7.0	3.5	9.0
Heavy Truck Apron	8.0	4.0	10.0
Portland Cement Concrete			
Automobile Parking	4.0	6.0	---
Automobile Drives	5.5	6.5	---
Truck Drives/Loading Dock Apron	7.0	7.0	---
Heavy Truck Apron	8.0	7.5	---

The concrete used for paving should have a modulus of rupture of at least 490 psi (equivalent to an approximate compressive strength of 3,000 psi at the time the pavement is subjected to traffic).

The pavement subgrade underlying the aggregate base should be properly prepared and compacted in accordance with the recommendations outlined under "Subgrade Preparation". If desired, we can evaluate the use of cement treatment of pavement subgrade to provide thinner pavement sections and more moisture resistant subgrade conditions beneath the pavement.

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).

Cement-treatment of the subgrade will create a rigid section that is prone to developing shrinkage cracks. The cracks can cause stress concentrations in the asphalt surface and the development of reflective cracking in the asphalt. Placing a flexible material, like an aggregate base course, between the treated subgrade and asphalt pavement will provide stress relief and help minimize reflective cracking. We also recommend “micro-cracking” the cement-treated subgrade to provide a more flexible subgrade section and reduce the potential for reflective cracking. Micro-cracking is the process by which minute, subsurface cracks are generated in the cement-treated subgrade and is achieved by loading the cement-treated subgrade within 30 to 72 hours after initial cure and completion of compaction. The implementation of micro-cracking will help to minimize the development of shrinkage cracks while not impacting the pavement’s overall structural capacity. Additional details on the micro-cracking process can be found in Section 301-3.4.13 of the 2018 Standard Specifications or Public Works Construction (Green Book).

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

4.11 SUBSURFACE INFILTRATION

Current regulations require that storm water be infiltrated into the site soils of new developments, when possible. The soil types present at the site control the ability of water to infiltrate into the subgrade. Based upon our subsurface investigation, the subsurface natural soils underlying the site consist predominately of very moist to wet clays, which typically are inadequate to accept infiltration.

Due to the presence of clayey soils and shallow groundwater, infiltration of stormwater is not considered feasible for the site.

4.12 GEOTECHNICAL OBSERVATION AND TESTING

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

5.0 LIMITATIONS

The report, exploration logs, and other materials resulting from GPI's efforts were prepared exclusively for use by B8 Myford II Industrial Owner LLC and Panattoni Development Company, Inc. and their consultants in designing the proposed development. The report is not intended to be suitable for reuse on extensions or modifications of the project or for use on any project other than the currently proposed development, as it may not contain sufficient or appropriate information for such uses. If this report or portions of this report are provided to contractors or included in specifications, it should be understood that they are provided for information only. This report cannot be utilized by another entity without the express written permission of GPI. This report is an instrument of our services and remains the property 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 during grading, excavation, and foundation construction by GPI. 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 construction phase services are performed by others, they must accept full responsibility (as Project Geotechnical Engineer) 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
als Inc.

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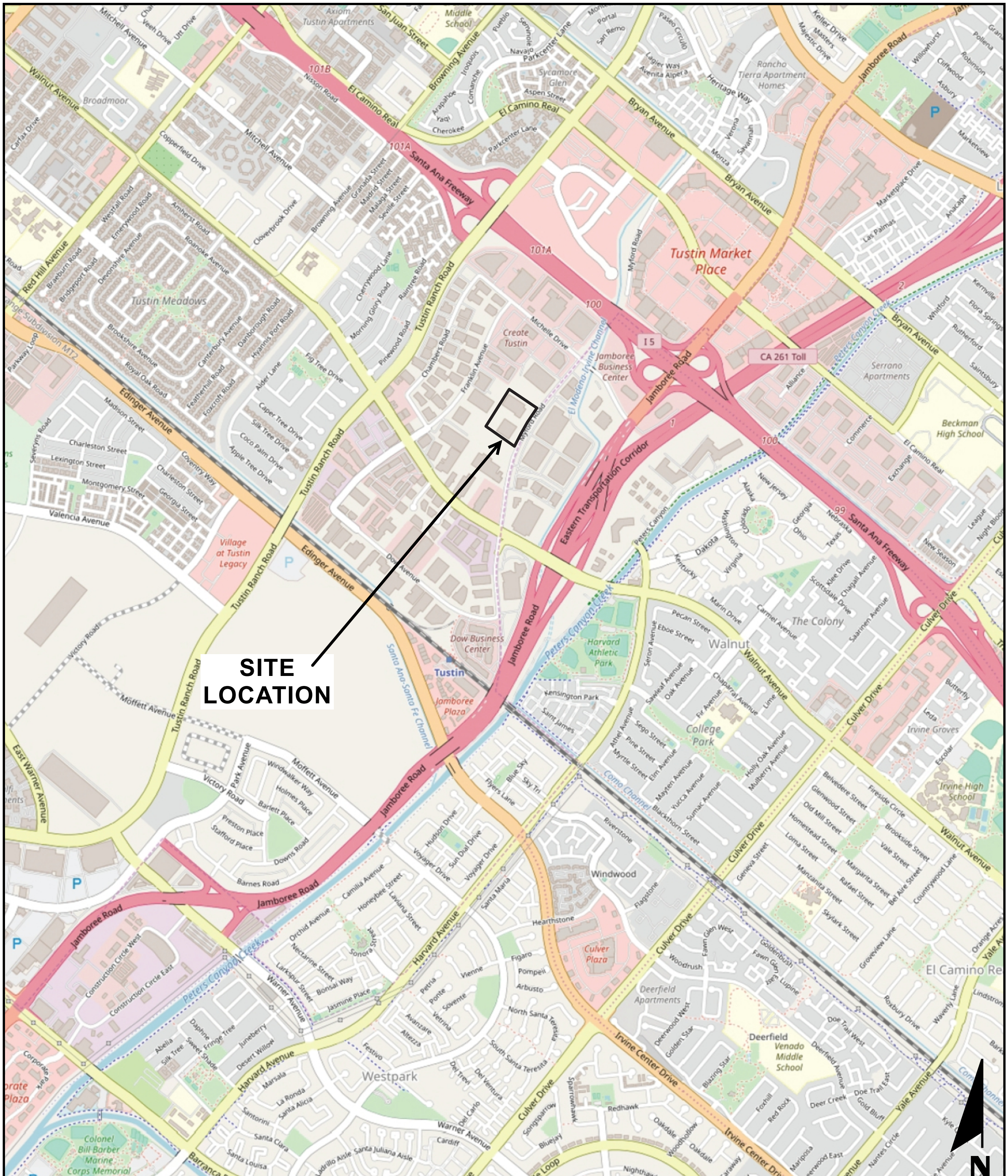


0 2000 4000 FEET

BASE MAP REPRODUCED FROM USGS 7.5' TOPO MAPS (WGS84 USNG ZONE 11SMT) © CALTOPO

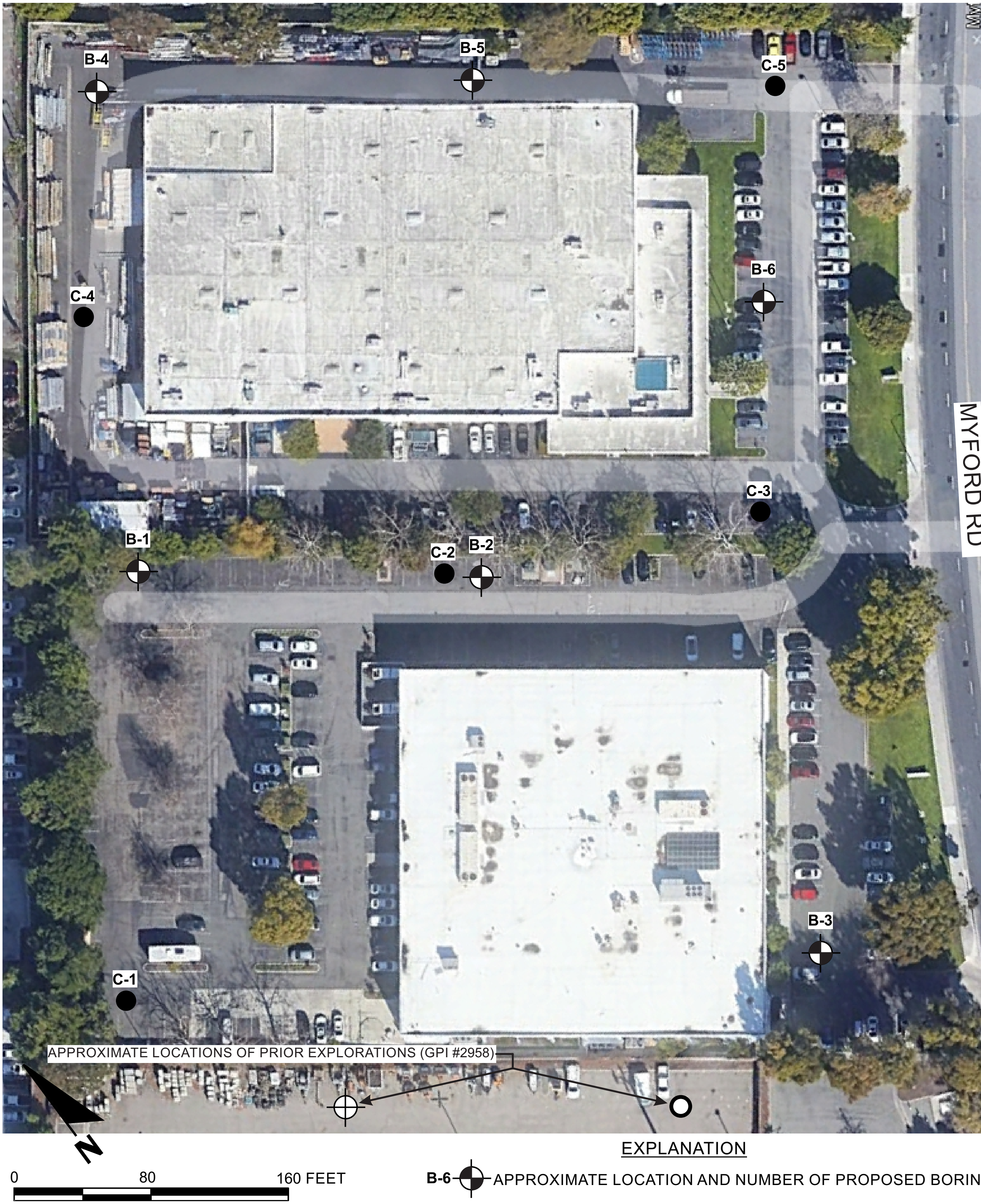
GEOTECHNICAL PROFESSIONALS, INC.

PANATTONI MYFORD II

GPI PROJECT NO.: 3078.I | SCALE: 1" = 2000'

SITE LOCATION MAP

FIGURE 1



EXPLANATION

B-6 APPROXIMATE LOCATION AND NUMBER OF PROPOSED BORING

C-5 APPROXIMATE LOCATION AND NUMBER OF PROPOSED CPT

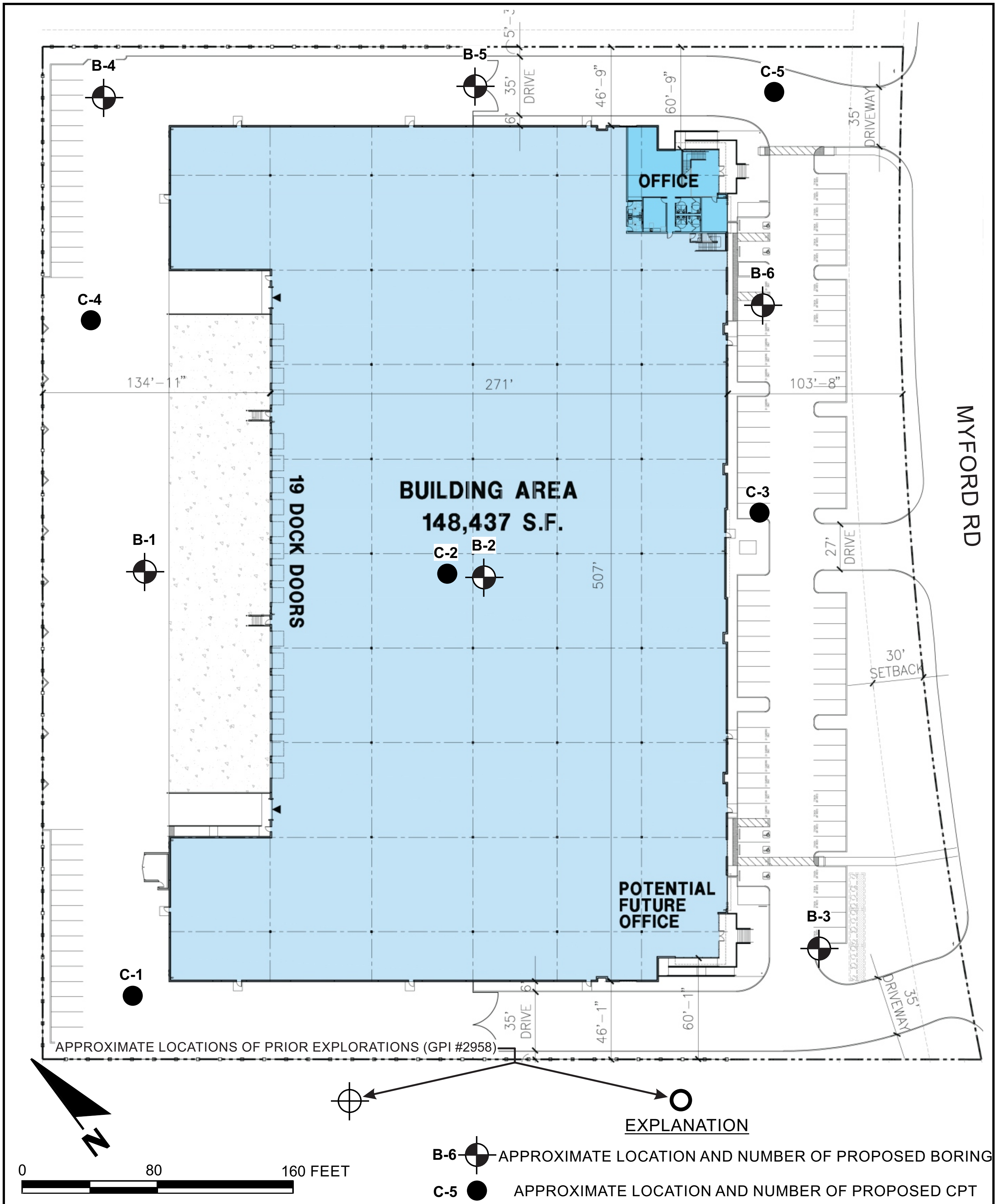
GPI GEOTECHNICAL PROFESSIONALS, INC.

MYFORD II

GPI PROJECT NO.: 3078.I | SCALE: 1" = 80'

SITE PLAN (EXISTING CONDITIONS)

FIGURE 2



BASE PLAN REPRODUCED FROM CONCEPTUAL SITE PLAN BY HPA ARCHITECTURE 8/9/22

GPI GEOTECHNICAL PROFESSIONALS, INC.

MYFORD II

GPI PROJECT NO.: 3078.I | SCALE: 1" = 80'

SITE PLAN (PROPOSED CONDITIONS)

FIGURE 3

APPENDIX A

APPENDIX A

CONE PENETRATION TESTS

The subsurface conditions were investigated by performing five Cone Penetration Tests (CPT’s) at the site. The soundings were advanced to depths of 44 to 80 feet below existing grades, with one CPT refusing at a depth of 44 feet on dense subsurface soils prior to the planned depth. The locations of the CPT’s are shown on the Existing and Proposed Site Plans, Figures 2 and 3, respectively.

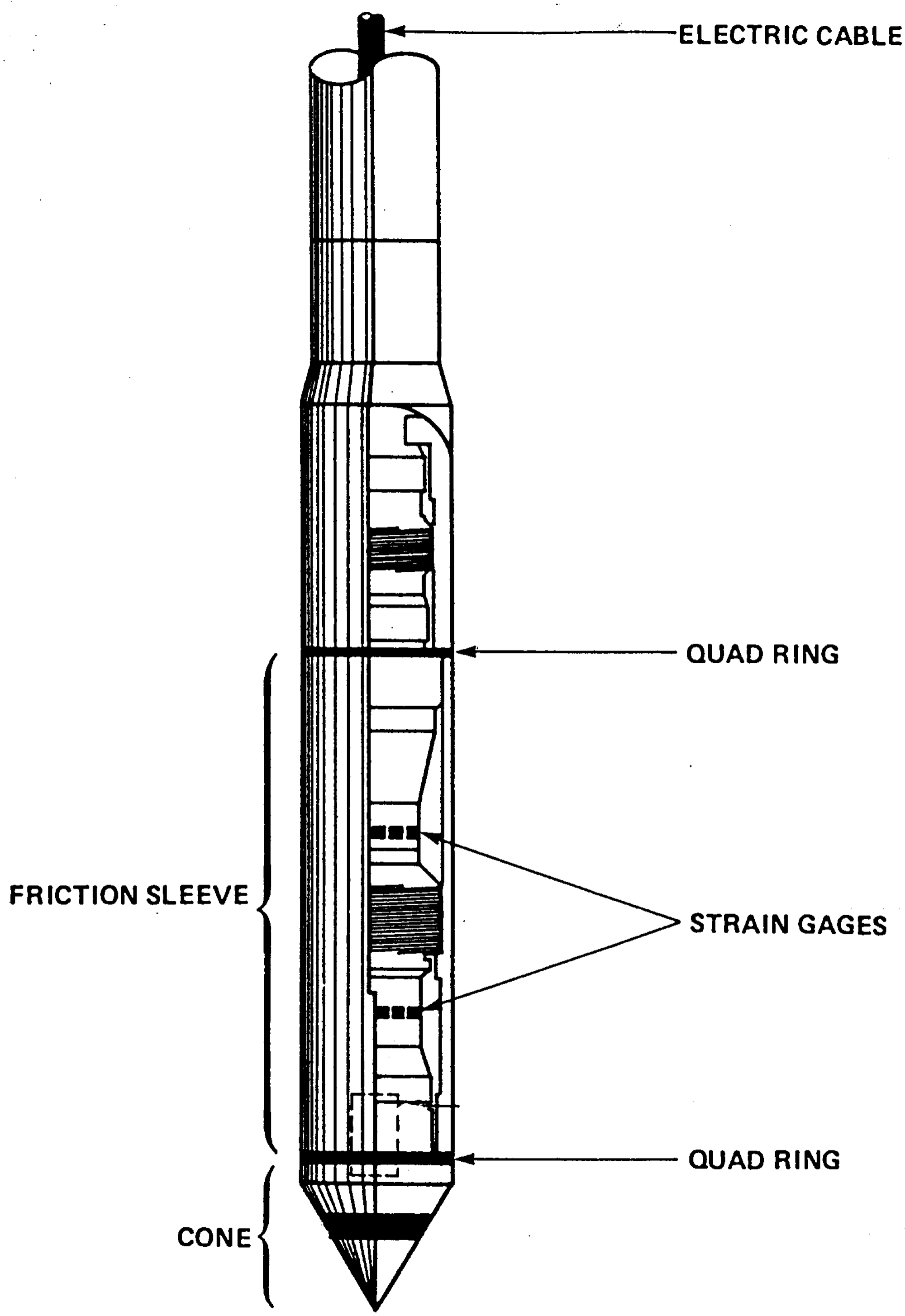
The Cone Penetration Test consists of pushing a cone-tipped probe into the soil deposit while simultaneously recording the cone tip resistance and side friction resistance of the soil to penetration (refer to Figure A-1). The CPT's described in this report were conducted in general accordance with ASTM specifications (ASTM D 5778) using an electric cone penetrometer.

The CPT equipment consists of a cone assembly mounted at the end of a series of hollow sounding rods. A set of hydraulic rams is used to push the cone and rods into the soil while a continuous record of cone and friction resistance versus depth is obtained in both analog and digital form at the ground surface. A specially designed truck is used to transport and house the test equipment and to provide a 30-ton reaction to the thrust of the hydraulic rams.

Data obtained during a CPT consists of continuous stratigraphic information with close vertical resolution. Stratigraphic interpretation is based on relationships between cone tip resistance and friction resistance. The calculated friction ratio (CPT friction sleeve resistance divided by cone tip resistance) is used as an indicator of soil type. Granular soils typically have low friction ratios and high cone resistance, while cohesive or organic soils have high friction ratios and low cone resistance. These stratigraphic material categories form the basis for all subsequent calculations, which utilize the CPT data.

Computer plots of the reduced CPT data acquired for this investigation are presented in Figures A-2 to A-6 of this appendix. The field testing and computer processing for the current investigation was performed by Kehoe Testing under subcontract to Geotechnical Professionals Inc. (GPI). The interpreted soil descriptions were prepared by GPI.

The CPT locations were laid out in the field by measuring from existing features at the site. The ground surface elevations at the CPT locations were estimated from Google Earth and should be considered approximate.



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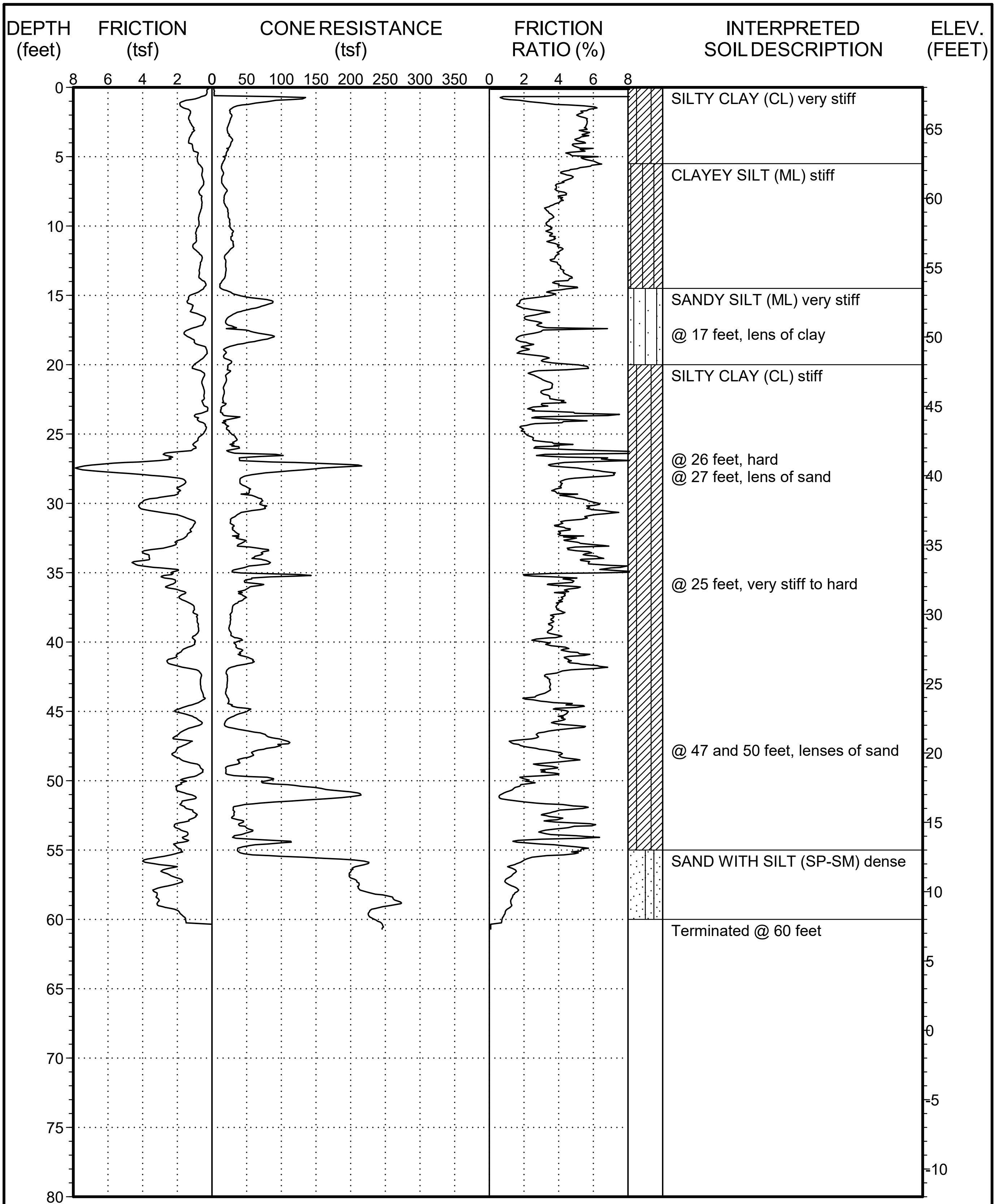
CONE PENET O ETER

FIGURE



Date performed:1-26-22

This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance and is a simplification of actual conditions encountered.

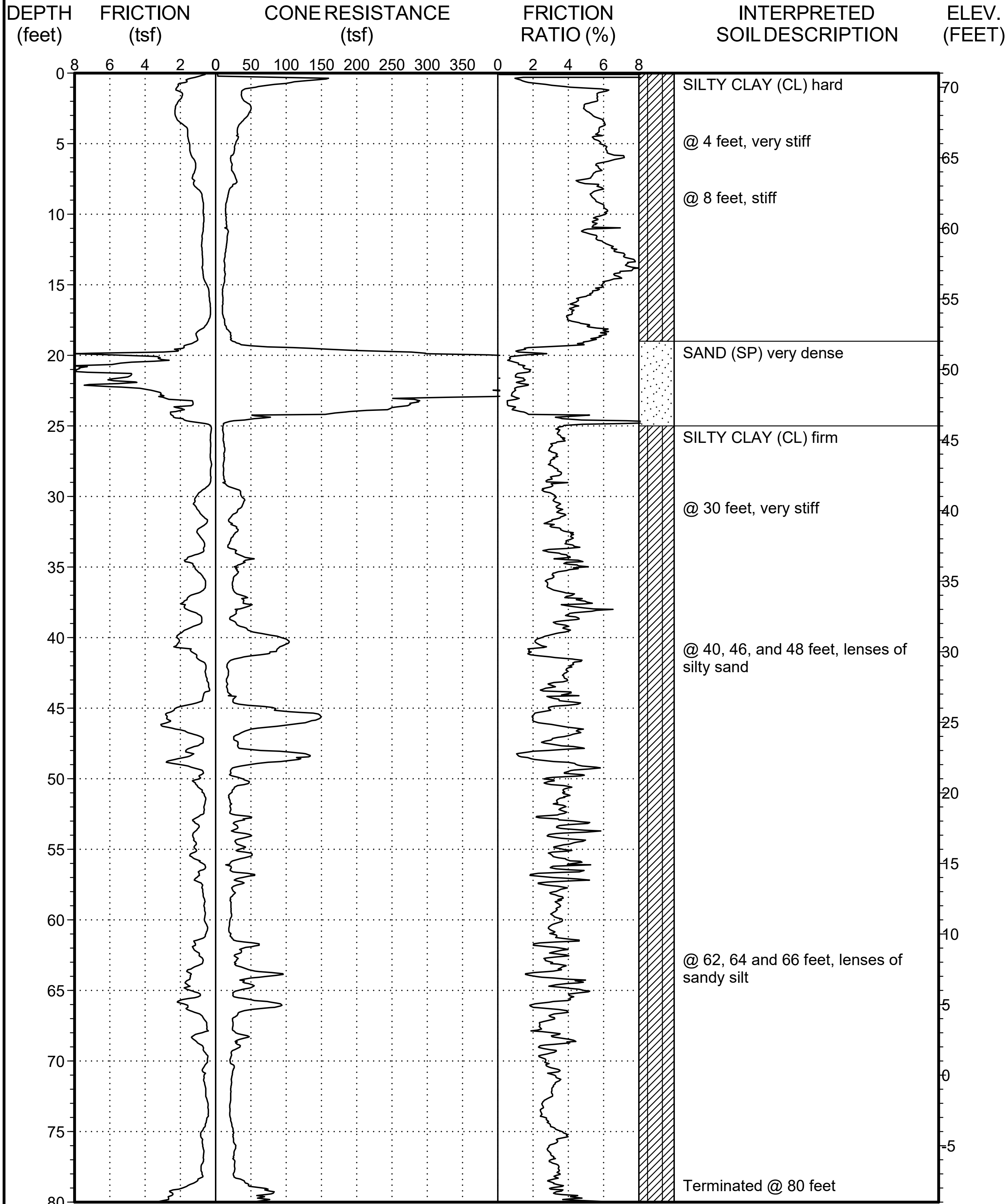
GPI

PROJECT NO.: 3078.I

MYFORD II

LOG OF CPT NO. C-1

FIGURE A-2



Date performed: 1-26-22

This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance and is a simplification of actual conditions encountered.

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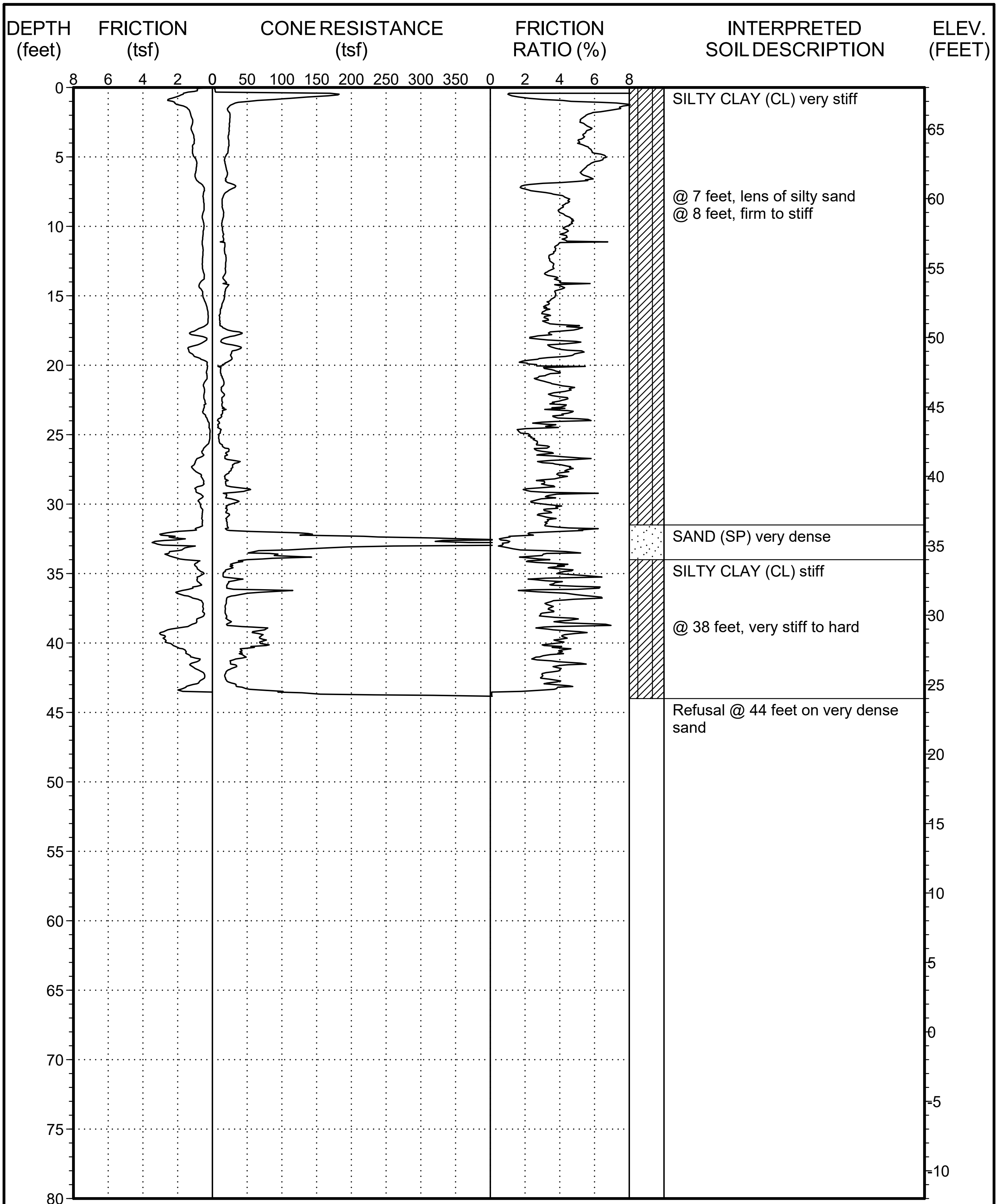
PROJECT NO.: 3078.I
MYFORD II

LOG OF CPT NO. C-2

FIGURE A-3



Date performed: 1-26-22

This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance and is a simplification of actual conditions encountered.

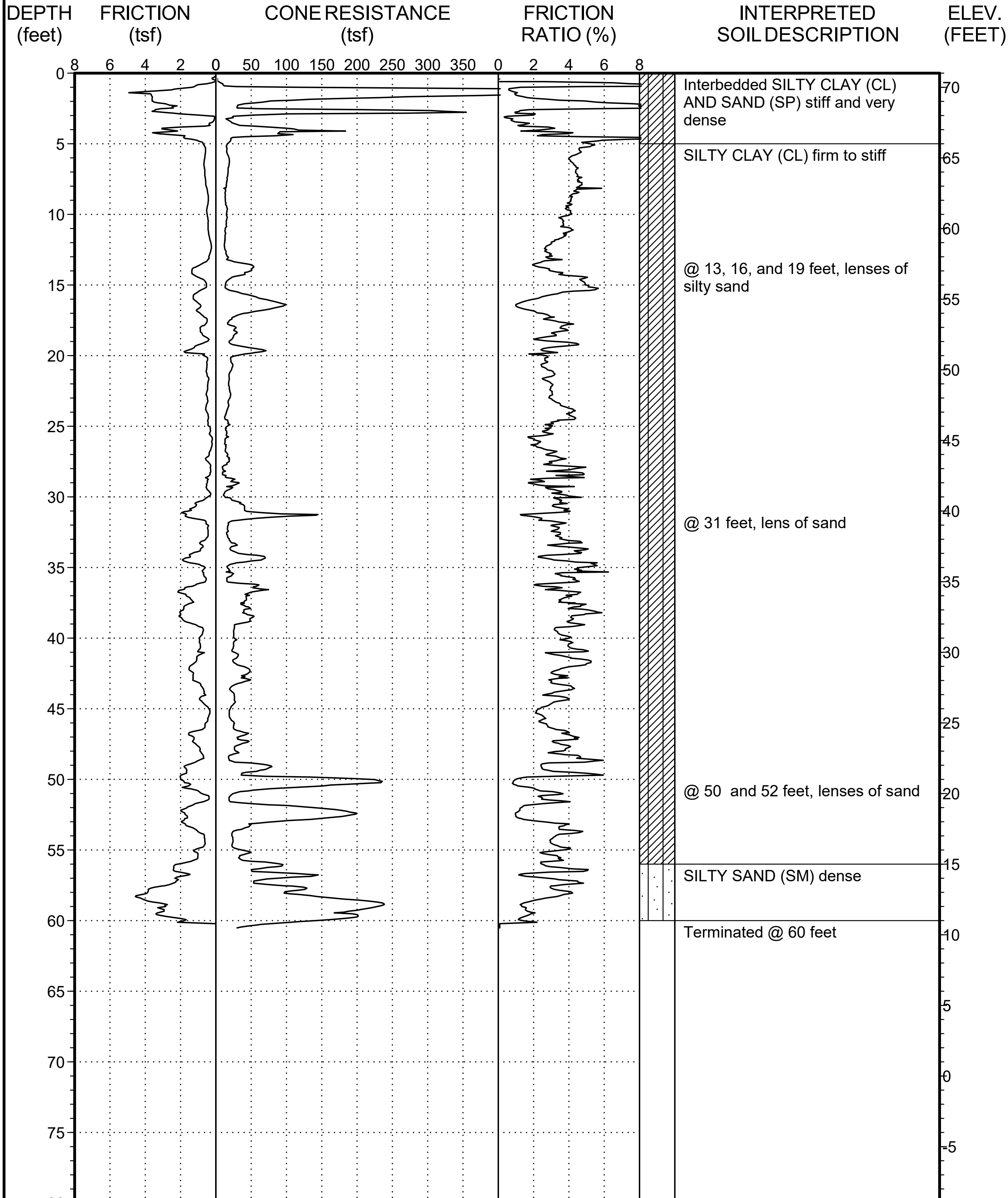
GPI

PROJECT NO.: 3078.I

MYFORD II

LOG OF CPT NO. C-3

FIGURE A-4



Date performed: 1-26-22

This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance and is a simplification of actual conditions encountered.

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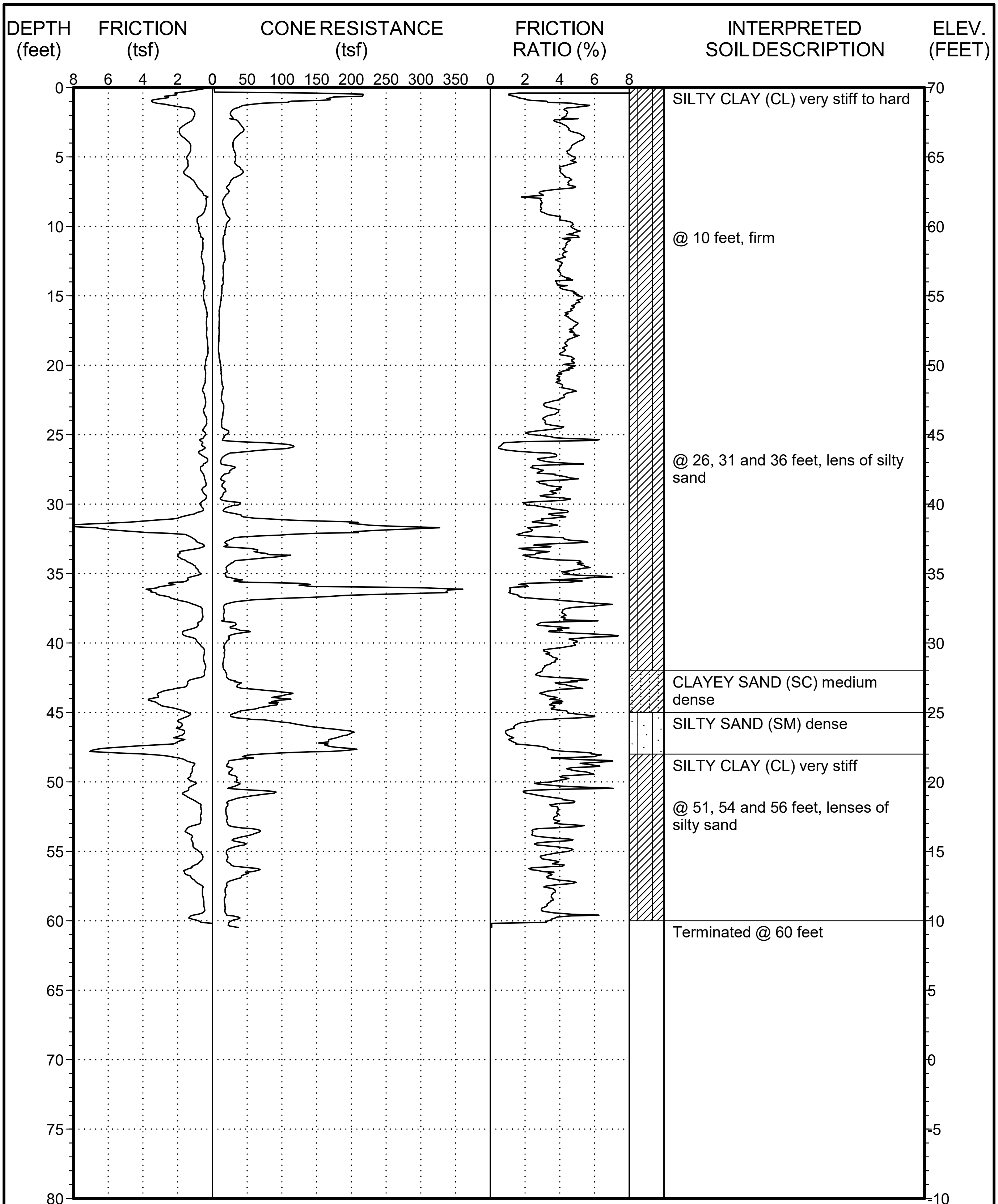
LOG OF CPT NO. C-4

FIGURE A-5



Date performed:1-26-22

This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance and is a simplification of actual conditions encountered.

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FIGURE A-6

APPENDIX B

APPENDIX B

EXPLORATORY BORINGS

The subsurface conditions at the site were investigated by drilling and sampling six exploratory borings. The borings were advanced to depths of about 21½ to 51½ feet below the existing ground surface. The exploration location is shown on the Existing and Proposed Site Plans, Figures 2 and 3, respectively.

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 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 to B-6 in this appendix.

The boring location was laid out in the field by measuring from existing features at the site. Upon completion, the boring was backfilled with the excavated soil cuttings. Excess soil cuttings were drummed, tested, and disposed of offsite. The ground surface elevations at the boring locations were estimated from Google Earth and should be considered approximate.

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	*DESCRIPTION OF SUBSURFACE MATERIALS* 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	4-Inch AC over 5-Inch BASE	
18.8	91	18	D		Fill: **SILTY CLAY (CH)** dark brown, very moist, very stiff, trace gravel	
27.9	86	18	D		**SILTY CLAY (CL)** brown, wet, stiff, trace sand, with calcium carbonate	65
17.6	101	20	D	5	@ 5 feet, very moist	
					SANDY SILT (ML) brown, very moist, stiff	
					SILTY CLAY (CH) dark brown, moist, stiff, trace gravel, with calcium carbonate	60
14.4	106	13	D	10		
21.8	101	19	D		@ 13 feet, brown, very moist, trace sand	55
13.2	120	27	D	15	**SILTY SAND (SM)** brown, very moist, medium dense	
						50
18.5		9	S	20		
					Total Depth 21.5 feet	

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

DATE DRILLED:
1-28-22

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

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MYFORD II

LOG OF BORING NO. B-1

FIGURE B-1

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
15.5	97			0	5-Inch AC over 4-Inch BASE	
15.5	97				Fill: **SILTY CLAY (CH)** dark brown, moist, very stiff, trace sand	
33.5	88	34	D			
					Natural: **SILTY CLAY (CH)** dark brown, wet, very stiff, with calcium carbonate	
29.6	92	34	D	5		65
27.1	94	17	D		**CLAYEY SILT (MH)** grey brown, wet, stiff, trace sand, with calcium carbonate	
27.5	93	15	D	10		60
22.0	102	11	D		**SILTY CLAY (CL)** grey brown, very moist, firm, trace sand	
15.8	115	15	D	15	**CLAYEY SAND (SC)** light brown, very moist, loose to medium dense	55
18.7		6	S	20	@ 20 feet, wet, loose	50
21.0					**SANDY CLAY (CL)** light brown, very moist, firm	
					SILTY CLAY (CL) brown, very moist, stiff, with calcium carbonate	
24.4	100	17	D	25		45
24.3		13	S	30		40
20.2	112	31	D	35		35
						30

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

DATE DRILLED:
1-20-22

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
25

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MYFORD II

LOG OF BORING NO. B-2

FIGURE B-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)
	21.7		11	S	40	**CLAYEY SAND (SC)** light grey, wet, stiff	
	24.2	104	25	D	45	**CLAYEY SILT (ML)** grey brown, very moist, very stiff	25
	20.4		22	S	50	**SILTY CLAY (CL)** light brown, very moist, very stiff, with sand	20
						Total Depth 51.5	

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

DATE DRILLED:
1-20-22

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
25

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MYFORD II

LOG OF BORING NO. B-2

FIGURE B-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)
			B	0	6-Inch AC over 4 BASE	
					Fill: **SILTY CLAY (CH)** brown, wet, very stiff, trace sand	
					Natural: **SILTY CLAY (CL)** brown, very moist, very stiff, trace sand, with calcium carbonate	65
22.5	88	41	D	5		
21.3	98	19	D		**CLAYEY SILT (ML)** brown, very moist, stiff	
32.4	90				**SILTY CLAY (CH)** dark brown, wet, stiff, with calcium carbonate	60
35.9	83	14	D	10		
43.8	74	13	D			55
49.7	68	7	D	15	@ 15 feet, firm	
						50
24.2		26	S	20	@ 20 feet, light brown, very moist, very stiff	
					@ 21 feet, with gravel	
						45
9.2	131	50	D	25	**CLAYEY SAND (SC)** brown, slightly moist, dense, with gravel	
						40
27.9		16	S	30	**SILTY CLAY (CH)** dark brown, wet, stiff, with calcium carbonate	
					Total Depth 31.5 feet	

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

DATE DRILLED: 1-20-22

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): 20

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MYFORD II

LOG OF BORING NO. B-3

FIGURE B-3

Moisture (%)	Dry Density (PCF)	Penetration Resistance (Blows/Foot)	Sample Type	Depth (Feet)	Description of Subsurface Materials	Elevation (Feet)
				0	7-Inch AC over 6-Inch BASE	
34.4	84	23	D		Fill: **SILTY CLAY (CH)** dark brown, slightly moist, very stiff	
24.9	85	27	D	5	Natural: **SILTY CLAY (CH)** dark brown, slightly moist, very stiff, with calcium carbonate @ 4 feet, very moist, trace gravel	65
20.3	102	19	D		@ 6 feet, brown, stiff, trace sand	
37.2	81	14	D	10		60
17.1	95	10	D		@ 12 feet, firm	
17.5	114	12	D	15	@ 15 feet, firm to stiff, with sand, trace gravel	55
18.4		10	S	20		50
					Total Depth 21.5 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: 1-28-22

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): Not Encountered

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MYFORD II

LOG OF BORING NO. B-4

FIGURE B-4

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
				0	7-Inch PCC over 3-Inch BASE	
32.5	90	36	D		Fill: **SILTY CLAY (CH)** dark brown, wet, very stiff	
					Natural: **SILTY CLAY (CL)** dark brown, wet, very stiff	
32.4	82	26	D		**CLAYEY SILT (MH)** dark brown, wet, very stiff, with calcium carbonate	65
31.1	88	23	D	5	**SILTY CLAY (CH)** dark brown, wet, very stiff, with calcium carbonate	
13.2	104	21	D		**SILTY SAND (SM)** brown, very moist, medium dense	
34.0	88				**SILTY CLAY (CH)** dark brown, wet, very stiff, with calcium carbonate	60
30.0	89	23	D	10		
24.2	100	15	D		@ 13 feet, stiff, very moist, trace gravel	55
21.4	103	14	D	15	@ 15 feet, trace sand	
						50
15.6		14	S	20	**SILTY SAND (SM)** red brown, very moist, medium dense, trace clay	
					Total Depth 21.5 feet	

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

DATE DRILLED:
1-20-22
EQUIPMENT USED:
8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft):
Not Encountered

GPI

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MYFORD II

LOG OF BORING NO. B-5

FIGURE B-5

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
			B	0	4-Inch AC over 5-Inch BASE	
					Fill: **SILTY CLAY (CH)** brown, wet, stiff	
38.7	80	15	D			
					Natural: **SILTY CLAY (CH)** brown, wet, stiff, with calcium carbonate	65
				5		
26.9	93	7	D			
22.3	96				**CLAYEY SILT (ML)** brown, very moist, loose	
40.8	76	7	D		**SILTY CLAY (CH)** dark brown, wet, firm, with calcium carbonate	60
				10		
30.5	88	1	D			
						55
25.3	96	12	D	15	@ 15 feet, trace sand	
						50
24.1		6	S	20	**SANDY SILT (ML)** brown, wet, firm	
					Total Depth 21.5	

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

DATE DRILLED:
1-28-22

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

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MYFORD II

LOG OF BORING NO. B-6

FIGURE B-6

APPENDIX C

APPENDIX C

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 B.

ATTERBERG LIMITS

Liquid and plastic limits were determined for selected samples in accordance with ASTM D4318. Results of the Atterberg Limits test are summarized on Figure C-1.

DIRECT SHEAR

Direct shear tests were performed on undisturbed and remolded bulk samples in accordance with ASTM D 3080. The bulk samples were remolded to approximately 90 percent of the maximum dry density. The test specimens were placed in the shear machine, and a normal load comparable to the in-situ overburden stress was applied. The samples were inundated, allowed to consolidate, and then were sheared to failure at a strain rate of 0.0007 inches per minute. 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 C-2 to C-4.

CONSOLIDATION

One-dimensional consolidation tests were performed on undisturbed samples in accordance with ASTM D 2435. After trimming the ends, the samples were placed in the consolidometer and loaded to up to either 0.4 or 0.5 ksf. Thereafter, the samples were incrementally loaded to a maximum load of up to either 25.6 or 34.1 ksf. The samples were inundated at either 1.6 or 2 ksf. Sample deformation was measured to 0.0001 inch. Rebound behavior was investigated by unloading the sample back to either 0.4 or 0.5 ksf.

Results of the consolidation tests, in the form of percent consolidation versus log pressure are presented in Figures C-5 to C-7.

COMPACTION TEST

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

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	OPIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)
B-2	0 – 5	Silty Clay (CH)	15.5	108

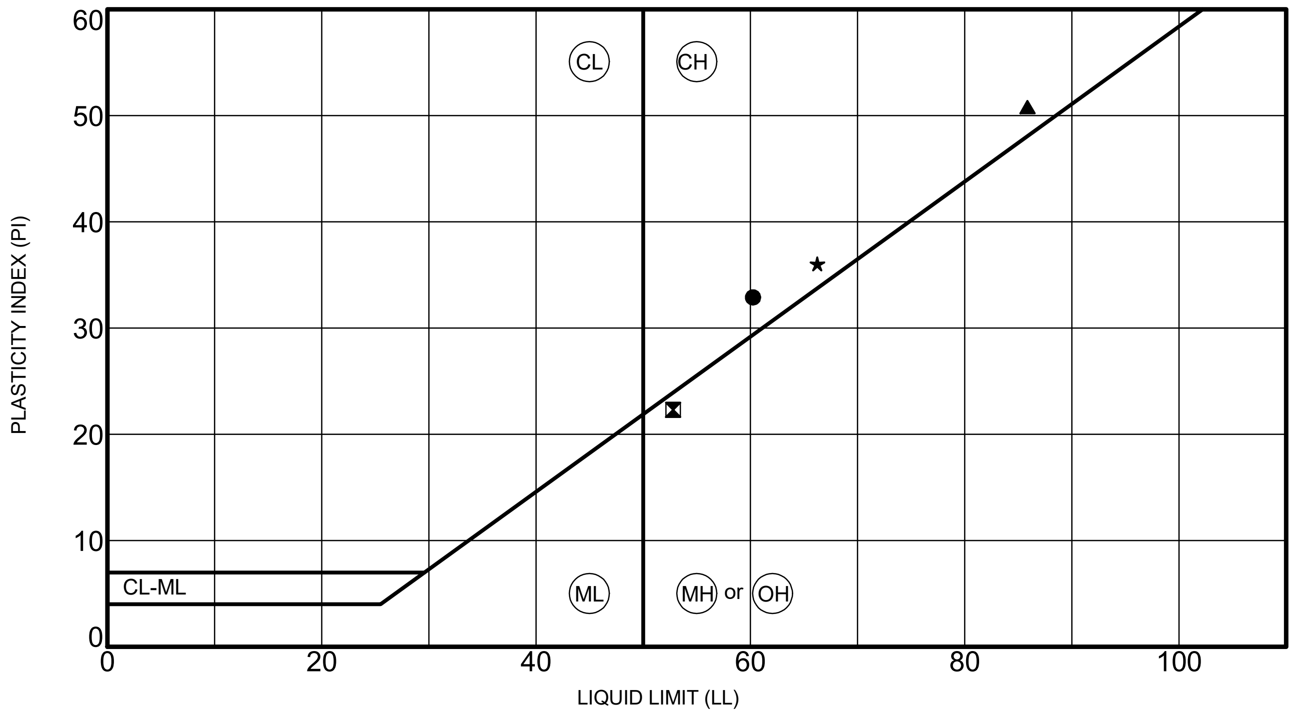
EXPANSION INDEX

Expansion index tests were performed on bulk samples. The tests were performed in accordance with ASTM 4829 to assess the expansion potential of the on-site soils. The results of the tests are summarized below:

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	EXPANSION INDEX
B-2	0 – 5	Silty Clay (CH)	65
B-3	0 – 5	Silty Clay (CH)	89
B-5	0 – 5	Silty Clay (CH)	102

CORROSIVITY

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



	SAMPLE LOCATION		LL	PL	PI	Fines, %	Classification
●	B-2	0-5	60	27	33		SILTY CLAY (CH)
⧗	B-2	6.0	53	30	22		CLAYEY SILT (MH)
▲	B-3	15.0	86	35	51		SILTY CLAY (CH)
★	B-5	5.0	66	30	36		SILTY CLAY (CH)

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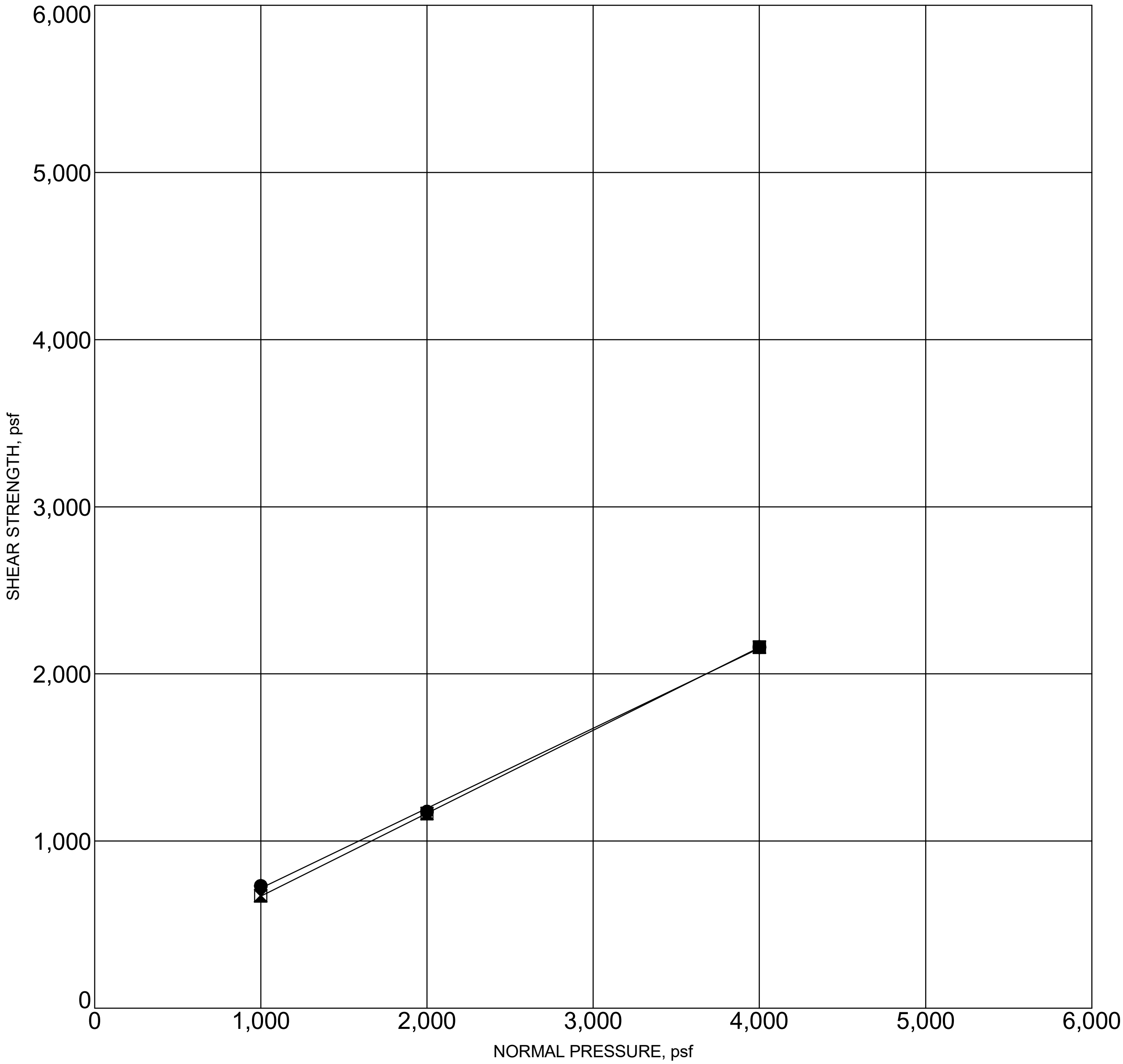
ATTERBERG LIMITS TEST RESULTS

FIGURE C-1



● **PEAK STRENGTH**
Friction Angle= 26 degrees
Cohesion= 240 psf

ULTIMATE STRENGTH
Friction Angle= 26 degrees
Cohesion= 174 psf

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

Sample Location		Classification	DD,pcf	MC,%
B-2	0-5	SILTY CLAY (CH)	97	15.5

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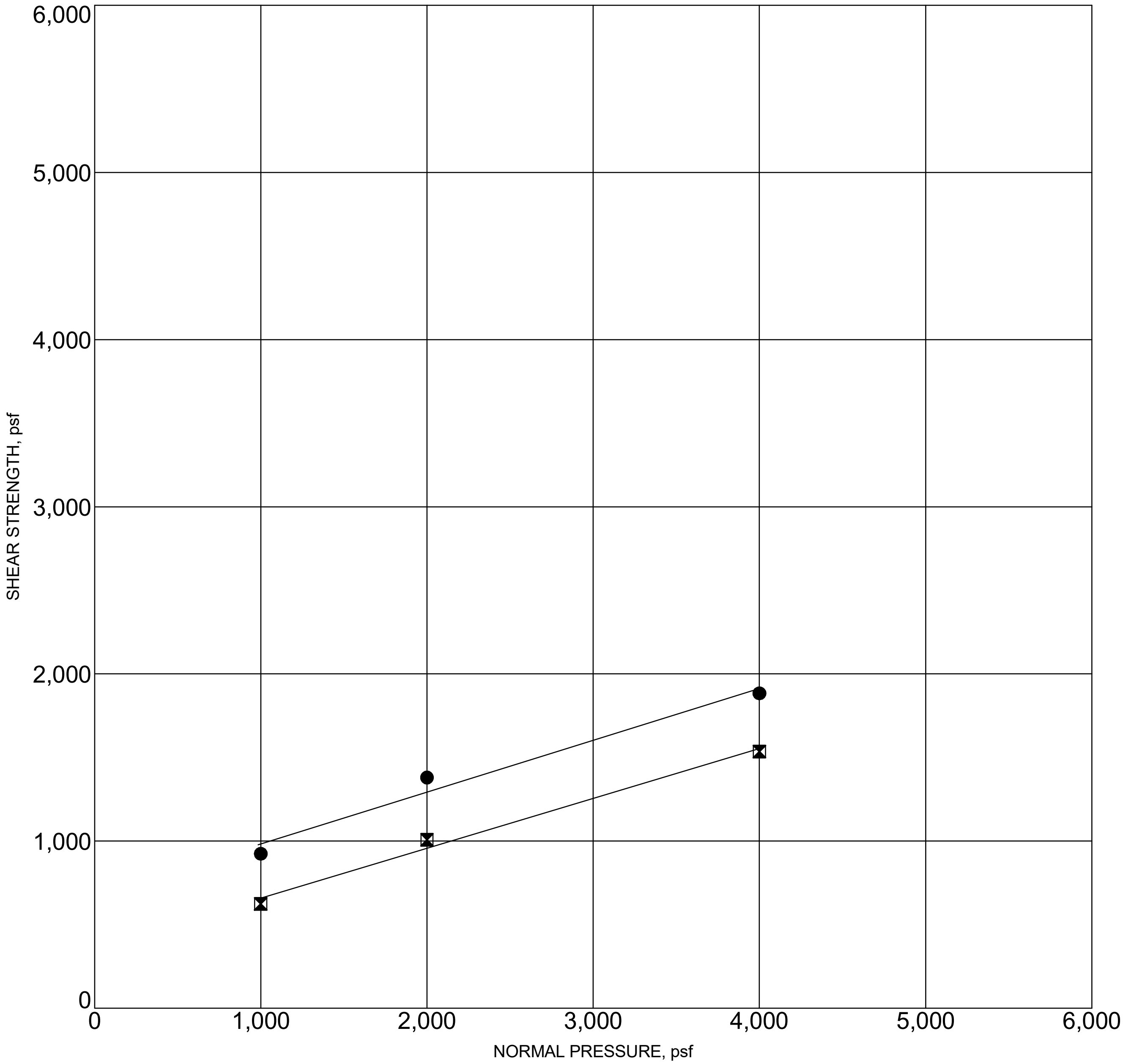
DIRECT SHEAR TEST RESULTS

FIGURE C-2



● **PEAK STRENGTH**
Friction Angle= 17 degrees
Cohesion= 672 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 17 degrees
Cohesion= 360 psf

Sample Location		Classification	DD,pcf	MC,%
B-3	7.0	SILTY CLAY (CH)	98	21.3

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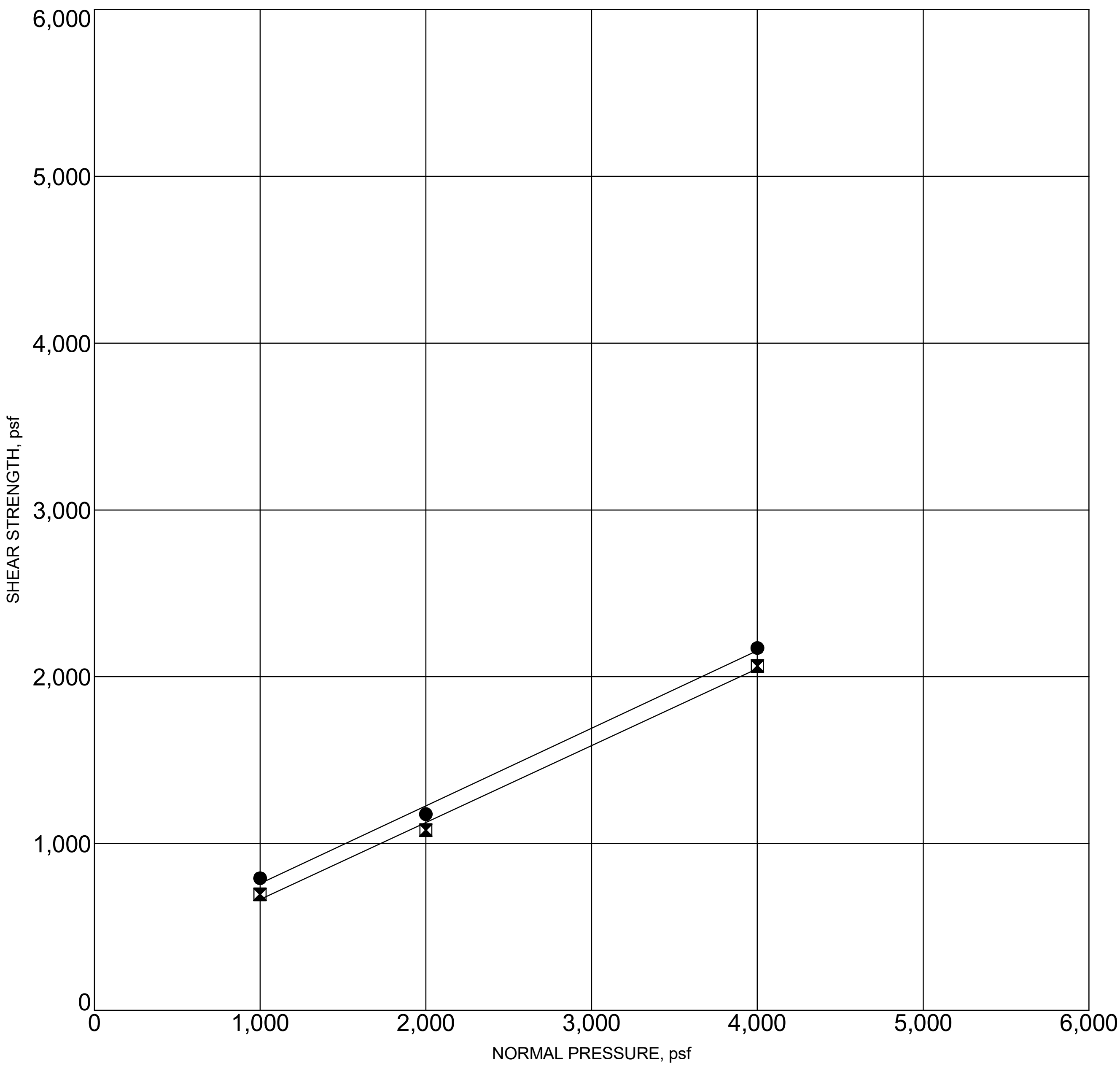
DIRECT SHEAR TEST RESULTS

FIGURE C-3



● **PEAK STRENGTH**
Friction Angle= 25 degrees
Cohesion= 294 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 25 degrees
Cohesion= 204 psf

Sample Location		Classification	DD,pcf	MC,%
B-5	5.0	SILTY CLAY (CH)	88	31.1

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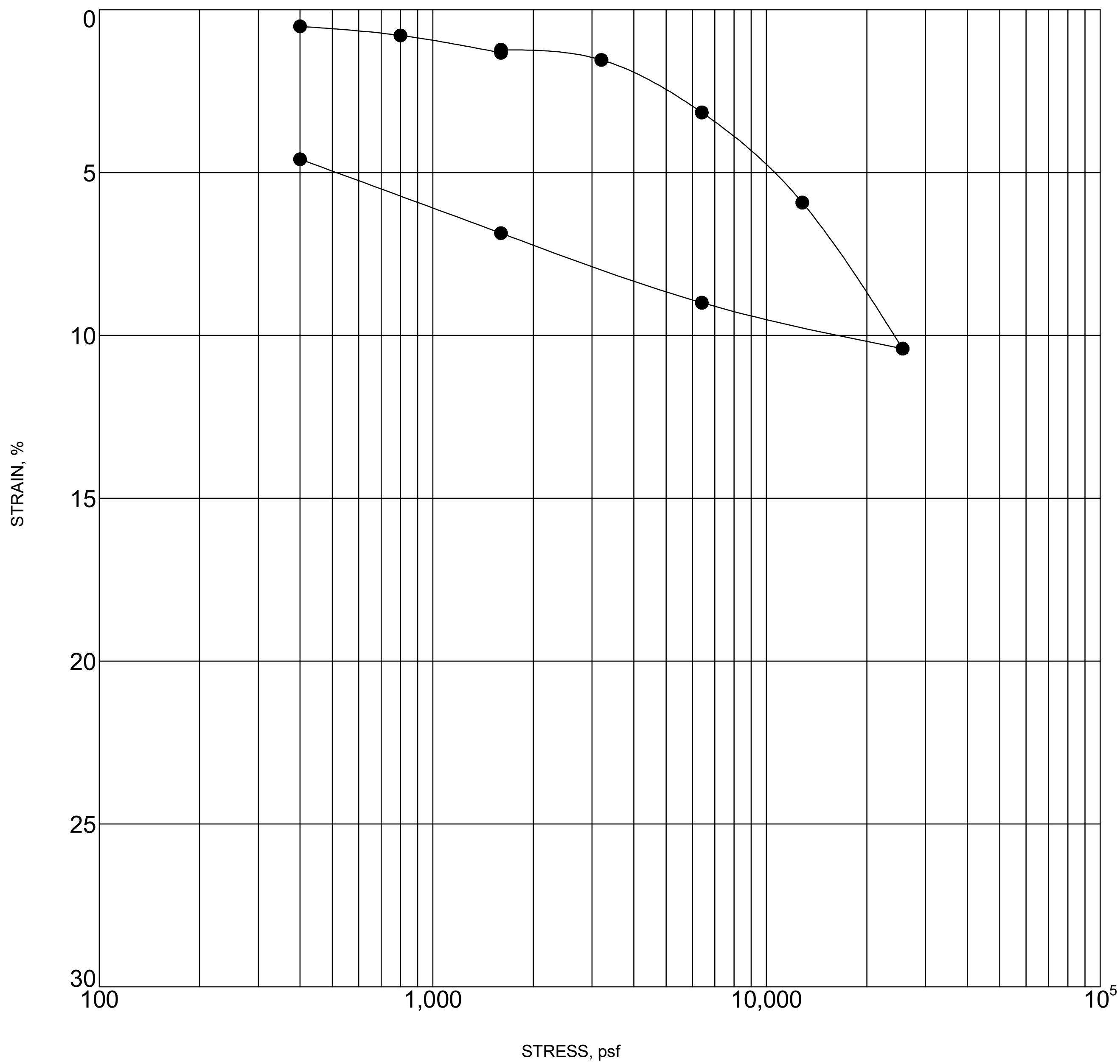
DIRECT SHEAR TEST RESULTS

FIGURE C-4



Sample inundated at 1600 psf

	Sample Location		**Classification**	**DD,pcf**	**MC,%**
●	B-2	6.0	CLAYEY SILT (MH)	94	27.1

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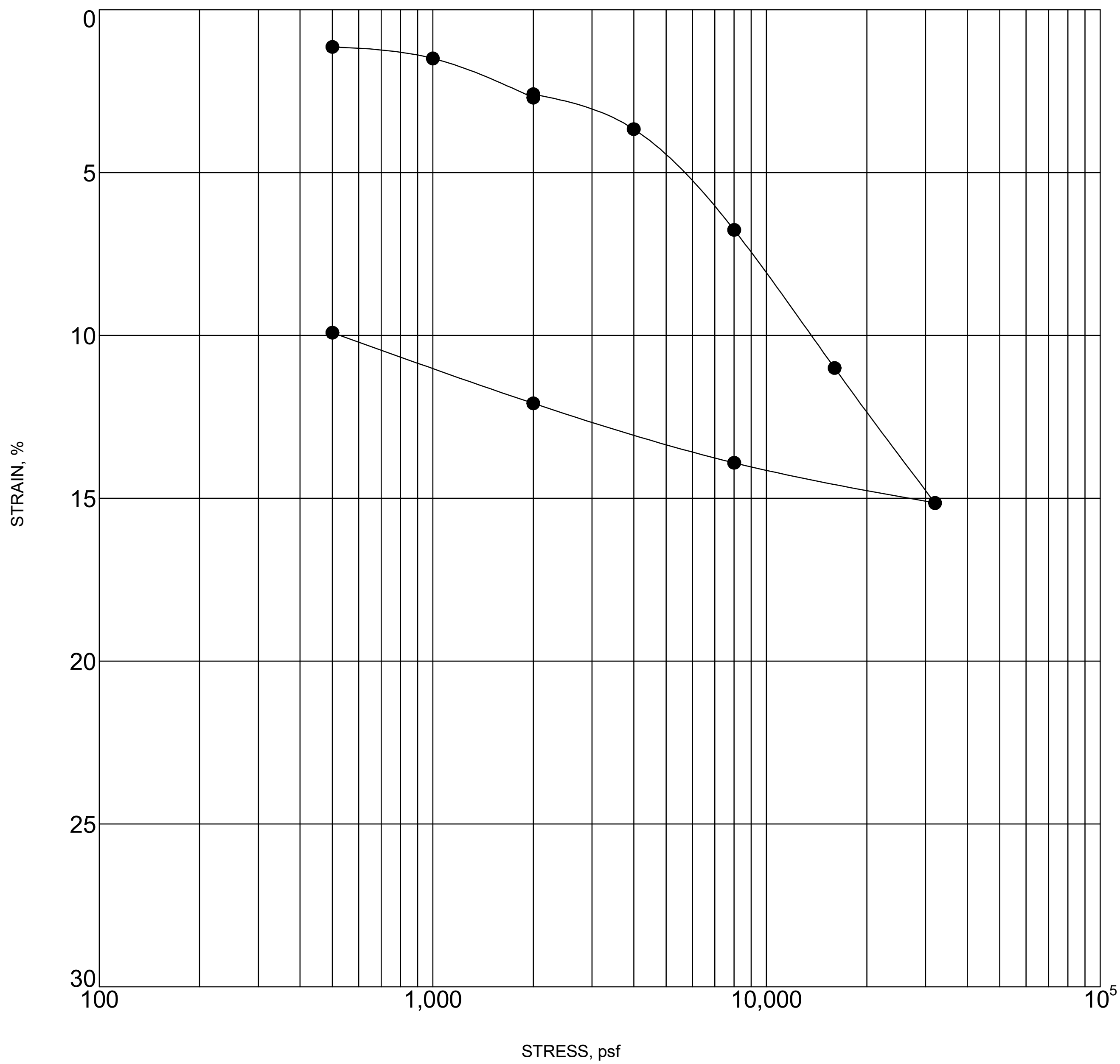
CONSOLIDATION TEST RESULTS

FIGURE C-5



Sample inundated at 2000 psf

	Sample Location		Classification	DD,pcf	MC,%
●	B-2	12.0	SILTY CLAY (CL)	102	22.0

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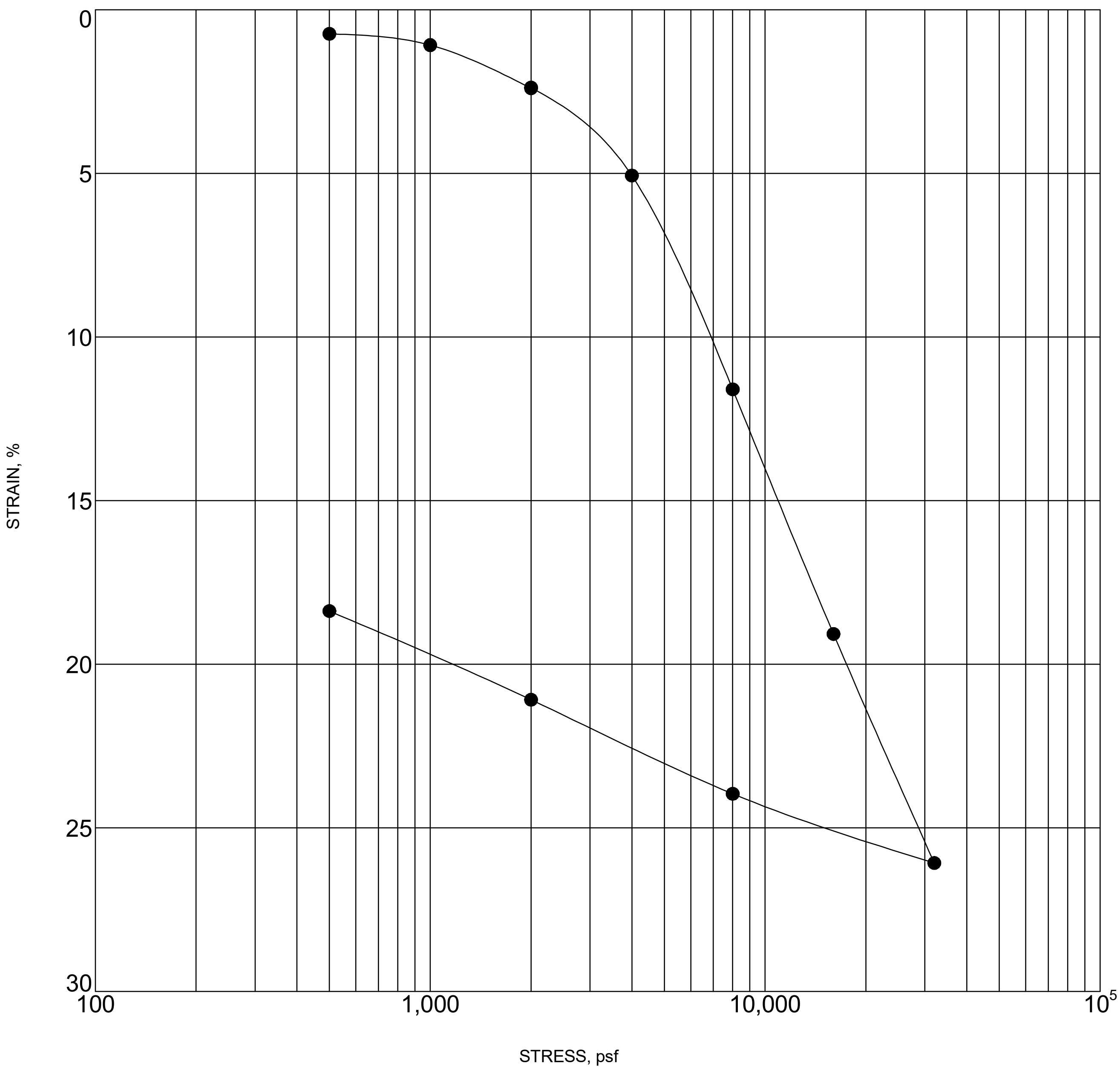
CONSOLIDATION TEST RESULTS

FIGURE C-6



Sample inundated at 2000 psf

	Sample Location		Classification	DD,pcf	MC,%
●	B-3	15.0	SILTY CLAY (CH)	68	49.7

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CONSOLIDATION TEST RESULTS

FIGURE C-7

Table 1 - Laboratory Tests on Soil Samples

Geotechnical Professionals, Inc.
Panattoni Myford II
Your #3078.I, HDR Lab #22-0115LAB
1-Feb-22

Sample ID			
			B-2 @ 0-5'
Resistivity		**Units**	
as-received		ohm-cm	1,240
saturated		ohm-cm	1,160
pH			7.1
Electrical Conductivity		mS/cm	0.47
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	156
magnesium	Mg^{2+}	mg/kg	101
sodium	Na^{1+}	mg/kg	270
potassium	K^{1+}	mg/kg	6.4
ammonium	NH_4^{1+}	mg/kg	ND
Anions			
carbonate	CO_3^{2-}	mg/kg	35
bicarbonate	HCO_3^{1-}	mg/kg	400
fluoride	F^{1-}	mg/kg	13
chloride	Cl^{1-}	mg/kg	41
sulfate	SO_4^{2-}	mg/kg	615
nitrate	NO_3^{1-}	mg/kg	204
phosphate	PO_4^{3-}	mg/kg	ND
Other Tests			
sulfide	S^{2-}	qual	negative
Redox		mV	246

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