Appendix D

Appendix D – Geotechnical Investigation Proposed Residential Development – Iron Lofts NEC Mission Inn Avenue and Commerce Street

Geotechnical Professionals, Inc

July 28, 2025

GEOTECHNICAL PROFESSIONALS INC.

GEOTECHNICAL INVESTIGATION
PROPOSED RESIDENTIAL DEVELOPMENT – IRON LOFTS
NEC MISSION INN AVENUE AND COMMERCE STREET
RIVERSIDE, CALIFORNIA

Prepared for:
Iron Lofts, LLC
c/o Realm
1201 Dove Street
Newport Beach, California 92660

Prepared by:
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Project No. 3044.I

July 28, 2025

GEOTECHNICAL PROFESSIONALS INC.

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Iron Lofts, LLC
c/o REALM
1201 Dove Street
Newport Beach, CA 92660

Attention: Mr. Todd Cadwell

Subject: Report of Geotechnical Investigation
Proposed Residential Development – Iron Lofts
NEC Mission Inn Avenue and Commerce Street
Riverside, California
GPI Project No. 3044.I

Dear Mr. Cadwell:

Transmitted herewith is our report of geotechnical investigation for the subject project. The report presents the results of our evaluation of the subsurface conditions at the site and recommendations for design and construction. This report has been updated from our 2023 report to include the current site plan and site description.

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

Very truly yours,
Geotechnical Professionals Inc.

Patrick McGervey, G.E.
Senior Engineer

Donald A. Cords, G.E.
Principal

Distribution: Addressee (PDF)

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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 apartment development located at the northeast corner of Mission Inn Avenue and Commerce Street in Riverside, California. The general site location is shown on the Site Location Map, Figure 1.

1.2 PROJECT DESCRIPTION

The project consists of a proposed residential development located at the subject 7-acre site.

The preliminary plan for the development consists of 300 residential units surrounded by at-grade parking. Building A is planned to be a 4-story wood frame structure, and Building C will consist of 2-story townhomes. Basements or subterranean levels are not planned.

Preliminary plans indicate that the apartment building will be extend over 2 blocks along Commerce Street from Mission Inn Avenue to 5th Street. The townhomes (Building C) will front Mission Inn Avenue. Two existing historical buildings (Buildings D and E) along Commerce Street and Mission Inn Avenue will remain for renovation as part of the project. The remainder of the abandoned buildings along Commerce Street will be demolished. The preliminary site configuration is shown on Figure 2, Site Plan.

Other improvements include at-grade parking (378 spaces), drive aisles, carports, EV charging stations, a swimming pool, a spa, and landscape areas. In general, the apartment buildings will front Commerce Street with at-grade parking to the east.

The existing site was formerly a metal recycling facility until ceasing operations in 2015 and subsequent demolition. Based on information provided by you, we understand that a Remedial Action Plan (RAP) prepared for the Department of Toxic Substance Control for the site (GSI, 2020) requires an environmental cleanup program with excavations and removal from the site of the near-surface impacted soils to depths of approximately 2 feet. The recycling facility formerly maintained underground storage tanks on the western portion of the site (GSI, 2020).

Based on information provided by the Project Structural Engineer, VCA, we understand the new buildings will be supported on a post-tensioned mat foundation system. The apartment building loads are anticipated to be on the order of 380 psf uniformly distributed across the post-tensioned slab, with point loads of less than 150 kips and wall loads on the order of 3.5 to 5.5 kip/ft. The townhome (Building C) loads are anticipated to be on the order of 190 psf uniformly distributed across the post-tensioned slab, with point loads of less than 50 kips and wall loads on the order of 2 to 3 kips/ft.

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. Also, when the project grading and foundation plans become available, we should be provided with copies for review and comment.

1.3 PURPOSE OF INVESTIGATION

The primary purpose of this investigation and report is to provide an evaluation of the existing geotechnical conditions at the site as they relate to the design and construction of the proposed development. More specifically, this investigation was aimed at providing geotechnical recommendations for earthwork, and design of foundations.

2.0 SCOPE OF WORK

Our scope of work for this investigation consisted of review of existing information, field exploration, laboratory testing, engineering analysis, and the preparation of this report.

Our subsurface exploration for this site consisted of five cone penetration tests (CPTs) and six hollow stem auger borings. The CPTs were advanced to depths of 50 feet below existing grades and the exploratory borings were drilled to depths of 16 to 31 feet below existing grades. A description of field procedures and logs of the explorations are presented in the attached Appendices A and B. The approximate locations of the subsurface explorations are shown on the Proposed Site Plan and Existing Site Plan, Figures 2 and 3 respectively.

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

Soil corrosivity testing was performed by HDR. R-value testing was performed by Geologic Associates. This work was performed 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 and assessments of seismic hazards. The results of our evaluations are presented in the remainder of the report.

3.0 SITE CONDITIONS

3.1 SURFACE CONDITIONS

The site currently contains vacant lots, single-story commercial buildings, asphalt concrete pavement in poor condition, surficial storage, and sparsely vegetated earth. Based on historical photographs (HistoricAerials), the site has been used for commercial purposes with similar structures since around 1948 (earliest available photograph).

The site is bound to the northwest by Commerce Street adjacent to the former Riverside Canal and railroad tracks, southwest by Mission Inn Avenue, northeast by 5th Street, and southeast by single family residential homes. A residential development completed in 2019 by REALM is located directly across Mission Inn Avenue.

In general, the site is relatively flat and slopes gently downward from east to west, with a change in ground surface elevation from about Elevation +881 feet to +876 feet across the site.

3.2 SUBSURFACE SOILS

Our field investigation disclosed a subsurface profile consisting of undocumented fills overlying natural soils. Detailed descriptions of the subsurface conditions encountered in our explorations are provided in Appendices A and B. A brief summary of the subsurface conditions are provided below.

To minimize contact and spreading of potentially contaminated soils, we didn't sample within the upper 5 feet of the soil profile. Undocumented fills soils were encountered in our explorations within the upper 5 to 5½ feet below existing site grades. Deeper undocumented fill soils may be present and associated with the existing buildings onsite, or the previous underground storage tanks within the western portion of the site. Where sampled, the fills consist of sandy silts, silty sands and sandy clays. The fill soils consisting of silty sands and sandy silts were difficult to distinguish from the underlying natural soils. Documentation regarding the placement and compaction of the fill soils was not provided.

The natural soils consist predominantly of interbedded layers of sandy silts, silty sands, silts, clays and sands. In general, the sandy soils were loose to medium dense, and the fine grained soils were stiff to hard. The silty sands and sandy silts varied from slightly moist to very moist.

The natural soils have moderate strength and low to moderate compressibility characteristics.

The near surface soils, in general, exhibit low expansion potential. The hydrocollapse potential of tested in-place sandy silts was found to be low (approximately 0.2 percent).

3.3 GROUNDWATER AND CAVING

Groundwater was not measured in our explorations to a depth of 50 feet below existing grade. Groundwater was reported by others (GSI 2020) to be greater than 100 feet below existing grade in 2009 at a nearby property. Groundwater is not anticipated to be encountered during grading or to adversely impact the development.

Due to the methods of drilling, the potential for caving was difficult to determine but dry, loose to medium dense, granular soils can be anticipated to exhibit caving.

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.

Prior to development of the site, we understand that a remedial action plan for an environmental cleanup program will be implemented with excavations and export of the near-surface impacted soils to depths of approximately 2 feet below existing grades.

After the completion of the remedial action plan, the following geotechnical issues should be incorporated into the design and construction of the proposed structures:

- Prior to placement of fills or construction of the building foundations, undocumented fills, disturbed soils, and a portion of the compressible natural soils should be removed and replaced as properly compacted fill. The depth of removals for each building and details regarding grading are provided in "Earthwork" section of this report.

- The proposed buildings may be supported on post-tensioned slab foundations after completion of remedial grading. The use of conventional foundations for the apartment and townhome buildings is feasible, but we understand that post-tensioned slabs are the preferred foundation alternative for the buildings.

- Carport structures are typically supported on drilled pier foundations. Based on the density and moisture content of the soils encountered, the caving potential of the soils is considered to be moderate to high in the dry sands. The drilled pile contractor should evaluate the potential drilling conditions when planning the installation methods.

- If storm water infiltration is used for the subject site, there is a risk of settlement/subsidence in the pavement areas. Infiltration systems should not be constructed within 30 feet of the buildings. We recommend the underground infiltration systems be located at least 10 feet horizontally from the drilled pile foundations. If infiltration systems are located closer to the drilled pile foundations, there is a potential for settlement of the carports.

- Resistivity testing of the on-site soils indicates that they are severely corrosive to metal.

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

We assume the seismic design of the proposed development will be in accordance with the 2022 California Building Code (CBC) criteria. For the 2022 CBC, a Site Class D may be used.

Using the Site Class, which is dependent on geotechnical issues, and the appropriate seismic design maps, the corresponding seismic design parameters from the CBC are as follows:

<u>2022 CBC:</u>

$S_S = 1.5g$	$S_{MS} = F_a * S_S = 1.5g$	$S_{DS} = 2/3 * S_{MS} = 1.0g$
$S_1 = 0.6g$	$S_{M1} = F_V * S_1 = 1.0g$	$S_{D1} = 2/3 * S_{M1} = 0.68g$

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

4.2.2 Strong Ground Motion Potential

Based on published information (geohazards.usgs.gov), the most significant fault in the proximity of the site is the San Jacinto Fault, which is located about 7 miles from the site.

During the life of the project, the site will likely be subject to strong ground motions due to earthquakes on nearby faults. Based on the USGS website (earthquake.usgs.gov), we computed that the site could be subjected to a peak ground acceleration (PGA_M) of 0.60g for a mean magnitude 6.9 earthquake. This acceleration has been computed using the mapped Maximum Considered Geometric Mean peak ground acceleration from the ASCE 7-16 (for 2022 CBC) and a site coefficient (F_{PGA}) based on Site Class. The predominant earthquake magnitude was determined using a 2-percent probability of exceedance in a 50-year period, or an average return period of 2,475 years. The structural design will need to incorporate measures to mitigate the effects of strong ground motion.

4.2.3 Potential for Ground Rupture

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

4.2.4 Liquefaction

The State of California has not yet prepared seismic hazard zone maps for the City of Riverside. The site is located within an area shown as having a low potential for soil liquefaction as determined by the City of Riverside (Riverside, 2018).

Soil liquefaction is not likely to occur at the project site due to the deep groundwater.

4.2.5 Seismic Ground Subsidence

Seismic ground subsidence (not related to liquefaction induced settlements) occurs when strong earthquake shaking results in the densification of loose to medium dense sandy soils above groundwater. We estimate the total magnitude of subsidence would be up to ¾-inch.

We estimate differential settlement due to subsidence across a 40 foot span may be on the order of one-half the total subsidence. The majority of the subsidence will occur in soils at depths from 10 to 35 feet below the existing ground surface.

4.3 EARTHWORK

The earthwork anticipated at the project site will consist of clearing, overexcavation of undocumented fills and disturbed soils, subgrade preparation, and placement and compaction of fill.

4.3.1 Clearing and Grubbing

The areas to be developed will be stripped of any vegetation and cleared of debris and pavements during the remedial grading for environmental cleanup. Buried obstructions below the remedial grading, such as footings, utilities, and tree roots should be removed. Deleterious material generated during the clearing operation should be removed from the site. Inert demolition debris, such as concrete and asphalt may be crushed for reuse in engineered fills, in accordance with the criteria presented in the “Materials for Fill” section of this report.

Although not encountered, we understand that underground storage tanks were located in the western portion of the site. Underground storage tanks, 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, any cesspools can be backfilled with a lean sand-cement slurry. At the conclusion of the clearing operations, a GPI representative should observe and accept the site prior to any further grading.

4.3.2 Excavations

Excavations at this site will include removals of undocumented fill soils, removals of disturbed/compressible soils, footing excavations, and trenching for proposed utility lines.

<u>Removal Depths</u>

We understand that the upper at least two feet of soils at the site will be removed and replaced with imported soils as part of the remedial environmental cleanup.

Prior to placement of fills or construction of the building foundations, undocumented fills, disturbed soils, and a portion of the compressible natural soils should be removed and replaced as properly compacted fill. These materials require densification to provide adequate support of foundations.

For planning purposes, we recommend the removal/replacement for the apartment building to extend to a minimum depth of 5 feet below existing grades or 3 feet below the bottom of the post-tensioned slab, whichever is deeper. For footings of minor lightly-loaded structures, such as small walls and trash enclosures, we recommend the removals and replacement extend to at least 2 feet below the bottom of footings.

Removals are not required under new or modified footings associated with the retrofit of the existing buildings (Buildings D and E). We recommend that in-place density tests be performed in the existing soils at the footing excavations by a representative of GPI to verify the soils have a relative compaction of at least 90 percent of the maximum dry density in accordance with ASTM D1557. For planning purposes, remedial compaction of the footing bottoms should be anticipated to achieve the above compaction requirements.

If not removed during the remedial grading, we recommend removals in pavement areas extend 1-foot below existing or proposed grades, whichever is deeper.

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

Lateral Limits

Where space is available, the base of the removals should extend laterally a minimum of 5 feet beyond the building lines or the depth of removal, whichever is greater. Building lines include the footprint of the building and other foundation supported improvements, elevator pits, stairwells, screen walls, loading docks, ramps, and canopies. Prior to grading, the corners of the areas to be overexcavated should be accurately staked in the field by the project surveyor.

The project surveyor should confirm the limits of removal relative to the actual building to confirm that the recommended remedial excavations have been performed. GPI does not practice surveying; therefore, we cannot confirm any lines, grades, or limits of excavations.

Existing Utilities

Where not removed by the aforementioned excavations, existing utility trench backfill should be removed and replaced as properly compacted fill within the proposed building pads. The limits of removal should be confirmed in the field. We recommend known utilities be shown on the grading plan.

Caving Potential and Cuts

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

Excavation Height (ft)	**Slope (h:v)**
≤5	Vertical
≤8	¾:1
≤15	1:1

If cuts greater than 15 feet are planned, we should be contacted to provide further recommendations. The allowable slope inclinations are measured from the toe to the top of the cut. Even at these inclinations, some raveling should be anticipated. The exposed slope face should be kept moist (but not saturated) during construction to reduce local sloughing.

Surcharge loads should not be permitted within a horizontal distance equal to the height of cut from the top of the excavation or 5 feet from the top of the slopes, whichever is greater, unless the cut is properly shored. Excavations that extend below an imaginary plane inclined at 45 degrees below the edge of adjacent existing site facilities should be properly shored to maintain support of adjacent elements. Excavations and shoring systems should meet the minimum requirements given in the State of California Occupational Safety and Health Standards.

Slot Cuts

Deeper removals along property lines or adjacent to existing improvements will require shoring or slot cuts. Recommendations for shoring are provided in the “Retaining Structures” section of the report. Removals that will undermine existing adjacent pavements or hardscape may utilize “ABC” slot cuts to depths not greater than 8 feet. Unsurcharged slot cuts up to 8 feet in height should not be wider than 4 feet and should be backfilled to finished grade prior to excavation of the adjacent four slots (two on each side of the excavated slot). We can provide slot widths for other slot heights if required. A test slot should be performed prior to production slots to confirm the stability of the planned cuts.

4.3.3 Subgrade Preparation

After removals are complete and prior to placing any fills or construction of the proposed structure, the subgrade soils should be scarified to a depth of 12 inches, moisture-conditioned, and compacted to at least 90 percent (95 percent for granular soils) of the maximum dry density in accordance with ASTM D 1557 and to a firm and unyielding condition.

In areas to receive pavements, the top 12 inches below the pavement base should be compacted to a minimum of 95 percent of the maximum dry density.

4.3.4 Material for Fill

The on-site soils are, in general, suitable for use as compacted fill under the buildings, pavement, and flatwork, and behind retaining/ramp walls. Clayey soils, if encountered at the site, should not be used within 12 inches of concrete pavements, or flatwork and are not suitable for use as retaining wall backfill.

Imported fill material should be predominately granular (contain between 10 and 40 percent fines-portion passing No. 200 sieve), and relatively non-expansive (an Expansion Index of less than 20). GPI should be provided with a sample (at least 50 pounds) and notified at least 72 hours in advance of the location of soils proposed for import. Each proposed import source should be sampled, tested and accepted for use prior to delivery of the soils to the site. Soils imported prior to acceptance by GPI may be rejected if not suitable.

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

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

If open-graded rock is used as backfill, the material should be placed in lifts and mechanically densified. Open-graded rock should be separated from the on-site soils by a suitable filter fabric (Mirafi 140N or equivalent).

4.3.5 Placement and Compaction of Fills

Fill soils should be placed in horizontal lifts, moisture-conditioned, and mechanically compacted to at least 90 percent (95 percent for granular soils) of maximum dry density in accordance with ASTM D-1557. 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 and heavy loaders	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 placement of subsequent lifts.

The moisture content of the on-site materials should be between 0 to 2 percent over the optimum moisture content to readily achieve the required degree of compaction.

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

4.3.6 Shrinkage and Subsidence

Shrinkage is the loss of soil volume caused by compaction of fills to a higher density than before grading. Subsidence is the settlement of in-place subgrade soils caused by loads generated by large earthmoving equipment. For earthwork volume estimating purposes, an average shrinkage value of 10 to 15 percent and subsidence of 0.1 feet may be assumed for the surficial soils (depths of 2 to 5 feet). These values are estimates only and exclude losses due to removal of vegetation or debris. Actual shrinkage and subsidence will depend on the types of earthmoving equipment used and should be determined during grading.

4.3.7 Trench/Wall Backfill

Utility trench backfill should be mechanically compacted in lifts. Wall backfill should consist of on-site or imported, non-expansive sandy soils. Lift thickness should not exceed those values given in the "Compacted Fill" section of this report. Jetting or flooding of trench or wall backfill materials should not be permitted. A representative of GPI should observe and test trench and wall backfill as they are placed.

4.3.8 Observation and Testing

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

Although not encountered during our onsite testing, the apartment development to the southwest of the site (GPI, 2014) contained soils with a moderate potential for collapse (2.2%).

A collapsible soil will undergo a reduction in volume upon wetting. Collapsible soil will typically have a low dry density and low moisture content. Collapsible soils may support large pressures with low compressibility when dry but experience significant compression upon wetting without an increase in pressure. Fine grained, aeolian or alluvial soils deposited in a semi-arid to arid climate tend to be the most susceptible to collapse.

To lessen the potential impacts of collapsible soils at the site, consideration should be given to minimizing the potential for wetting of soils underlying structures and other improvements. We recommend landscaping requiring significant irrigation be avoided within 10 feet of buildings to help mitigate potential for infiltration of water under the structures. If this setback is not feasibly, lining the bottom of the landscape areas should be considered.

Stormwater infiltration systems should not be constructed within 30 feet of the buildings.

4.5 FOUNDATIONS

4.5.1 Foundation Type

The proposed apartment and townhome buildings and may be supported on post-tensioned slab foundations, provided the subsurface soils are prepared in accordance with the recommendations given in this report. The use of conventional foundations for the apartment buildings is feasible; however, we understand that post-tensioned slabs are the preferred foundation alternative for the apartment buildings.

The post-tensioned slab foundations should be supported on compacted fill. Footings for minor at-grade structures, such as the pool shade structures, screen or retaining walls, or trash enclosures, should also be supported on properly compacted fill. Carport structures are typically supported on drilled pier foundations.

4.5.2 Post-Tensioned Slab Foundations

A post-tensioned slab foundation can be used to support the apartment and townhome buildings. The post-tensioned slab can be placed directly on engineered fills derived from the on-site or import silty sand soils with very low potential (E.I. $\leq$ 20).

Based upon loading provided by the Project Structural Engineer, the post-tension slabs for the apartment and townhome buildings will impose an average pressure of approximately 380 and 190 psf, respectively. These pressures are significantly less than the allowable bearing capacity of the compacted fill and natural soils underlying the buildings.

We estimate that the ground surface under the center of a loaded area of the apartment buildings (Buildings A and B) having applied pressures of approximately 380 psf will settle approximately ½-inch. The edges and corners of this area under the same loading conditions are expected to settle ¼-inch and less than ¼-inch for the above applied pressures, respectively. We estimate that the ground surface under the center of a loaded area of the townhome buildings (Building C) having applied pressures of approximately 190 psf will settle approximately ¼-inch. The edges and corners of this area under the same loading conditions are expected to settle less than ¼-inch for the above applied pressures, respectively.

Since the on-site or import soils underlying the post-tensioned slabs will have a very low expansion potential, the structural engineer needs not consider the soil shrinkage or swelling in the design of the post-tensioned slab.

These settlements assume a uniformly applied pressure and do not include the effects of the post-tension slab. The actual settlement of the post-tension slab will probably be less and will depend on the stiffness of the slab and its ability to distribute the loads and should be determined by the Structural Engineer.

For the structural analysis of the post-tensioned slab foundation, we recommend using an uncorrected modulus of subgrade reaction of 200 pci. This value is based on a 1-foot square bearing area and silty clays and clays underlying the foundation level. We recommend this modulus be reduced by 75 percent value to a value of 50 pci to account for the size of the post-tensioned slab foundation.

The allowable soil bearing pressure will be significantly greater than the average bearing pressures required for the post-tensioned slab foundation as discussed above. At localized or thickened areas of the slab, such as continuous footings, columns and point of load applications, a static allowable net bearing pressure of 2,000 pounds per square foot may be used with a minimum width of 24 inches and minimum embedment depth of 24 inches. These allowable bearing pressures are for dead-plus-live loads, and may be increased one-third for short-term, transient, wind and seismic loading. The total static settlement under the thickened portions of the post-tensioned slab will be on the order of 1 inch or less.

4.5.3 Shallow Foundations

Based on the shear strength and elastic settlement characteristics of the compacted fill derived from on-site soils, minor lightly-loaded structures, including retaining walls, site walls, and trash enclosures, 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. Based on the shear strength and elastic settlement characteristics of the natural on-site soils, a static allowable net bearing pressure of up to 1,500 pounds per square foot (psf) may be used for both continuous footings and isolated column footings for the retrofitting of the existing buildings.

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 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)
Minor Structures		
2,000	24	18
1,500	18	18
1,000	15	15
Retrofitting of Existing Buildings		
1,500	24	18
1,000	18	18

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

A minimum footing width and depth of 15 inches should be used even if the actual bearing pressure is less than 1,000 psf.

Maximum total static settlement will depend on the loads imposed on the foundations. In general, the total static settlements anticipated for wall loads of lightly-loaded structures will be limited to less than ½-inch. Maximum differential settlements are expected to be less than ¼-inch over a span of 40 feet. Total static settlement of isolated pad or continuous wall footings (up to 50 kips for columns and 6 kips per lineal foot for walls) for the retrofitting of the existing building is expected to be on the order of 1 inch or less. Differential static settlement between similarly loaded column footings or along a 40-foot span of a continuous footing is expected to be on the order of ½-inch or less.

The above estimates are based on the assumption that the recommended earthwork will be performed and that the footings will be sized in accordance with our recommendations. The project structural engineer should consider the total and differential settlement due to the above sources when designing the foundations of the minor, lightly-loaded structures.

4.5.4 Lateral Load Resistance

Soil resistance to lateral loads will be provided by a combination of frictional resistance between the bottom of post-tensioned slabs or spread footings and underlying soils and by passive soil pressures acting against the embedded sides of the slabs or footings. For frictional resistance, a coefficient of friction of 0.35 may be used for design. In addition, an allowable lateral bearing pressure equal to an equivalent fluid weight of 300 pounds per cubic foot may be used, provided the footings are poured tight against undisturbed natural soils or compacted fill. These values may be used in combination without reduction.

4.5.5 Carport Foundations

Carport structures can be supported on drilled pier foundations in accordance with the recommendations provided below.

Drilled pile foundations will develop their frictional capacity with relatively small deflection (about ¼-inch). The allowable capacities will depend on the pile diameter and the total depth of the pile below the pile cap. The downward vertical capacities shown below do not include capacity for end bearing, as it is difficult to adequately clean the base of small diameter drilled shafts. The values presented are for static loads, and can be increased by one-third for short-term wind and seismic loads. The allowable uplift capacity for a given pile may be taken as one-half of the compressive capacity provided below. We recommend a minimum pile depth of 8 feet.

DEPTH BELOW PILE CAP (feet)	**ALLOWABLE COMPRESSIVE CAPACITY (kips)**	
	24-inch Diameter	**30-inch Diameter**
8	12	16
10	18	24
12	24	32
14	30	40
16	36	48

If other diameters of piles are considered, an allowable frictional resistance of 150 pounds per square feet can be used for downward static loads in the upper 5 feet of the soil profile and 500 pounds per square foot for soils below 5 feet.

Soil resistance to lateral loads can be provided by the piles. The design of the piles will be governed by lateral force considerations. For design by the simplified pole formula presented in Section 1807A.3.2.1 of the 2022 California Building Code, a unit passive resistance of 300 pounds per square foot per foot (to a maximum of 3,000 pounds per square foot) may be used for the piles with level ground in lieu of the presumptive lateral bearing values presented in Table 1806A.2. As stated in the code, a passive resistance of 600 pounds per square foot per foot (to a maximum of 6,000 pounds per square foot) may be used for isolated piles as determined by the Project Structural Engineer. We recommend that the upper 1-foot of the subgrade soils be ignored in determining the required depth of embedment to allow for surface disturbance adjacent to the pile unless the pile is located within a paved surface.

Pile excavations should be filled with concrete on the same day they are drilled. Concrete mix designs should include provisions to minimize shrinkage, which can lead to lower frictional resistance of the pile shaft and reduced allowable capacity. The concrete should be placed with special equipment so that it is not allowed to fall freely more than 5 feet or strike the walls of the excavations. Drilling for piles should not be performed within 5 feet of recently excavated or recently poured piles until the concrete has been allowed to set for at least 6 hours. The piles should be poured in a manner that will not result in concrete flowing into adjacent drilled pile excavations and prevent segregation of aggregate. Drilled pile construction should be performed in accordance with the latest edition of ACI 336.1, "Standard Specifications for the Construction of Drilled Piles."

Based on the density and moisture content of the soils encountered, the caving potential of the soils is considered to be moderate to high in the dry sands. The drilled pile contractor should evaluate the potential drilling conditions when planning the installation methods. We do not anticipate encountering groundwater during the installation of piles.

Since the drilled piles will be designed to derive resistance from friction only, rigorous cleaning of loose material from the bottom of the excavation prior to placement of steel and concrete is not considered essential. Every effort should be made to clean the bottom with the drill rig-mounted equipment.

Downhole inspection of drilled pile excavations (by lowering an inspector into the excavation) is not needed as the piles are designed as frictional elements. However, pile and footing excavations should be observed by a representative of GPI to confirm and document the depth, diameter, and embedment in suitable materials.

4.5.6 Foundation Concrete

Laboratory testing by HDR (Appendix C) on a selected combined sample indicates that the near surface soils exhibit a soluble sulfate content of 1,890 mg/kg. For the 2022 CBC, foundation concrete should conform to the requirements outlined in ACI 318, Section 4.3 for sulfate exposure Category S1. Chloride levels in the sample of the upper soils tested were found to be 173 mg/kg, and is considered Category C1 due to the potential to be exposed to moisture.

4.5.7 Foundation Inspection

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

4.6 SLABS-ON-GRADE

A vapor/moisture retarder should be placed under any slabs (including post-tensioned slabs) that are to be covered with moisture-sensitive floor coverings (parquet, vinyl tile, etc.).

Currently, common practice is to use a 15-mil polyolefin product such as Stego Wrap for this purpose. The need or a sand layer with the vapor barrier is not a geotechnical issue and is a decision for the Project Architect.

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

For lateral resistance design, a coefficient of friction value of 0.35 between select fill and concrete may be used. For a slab on a visqueen moisture barrier, a coefficient of 0.1 should be used.

4.7 LATERAL EARTH PRESSURES

Based on information available to us at the time this report was prepared, no major retaining walls or basements were planned for the site. The following recommendations are provided for walls less than 6 feet in height. We recommend that retaining walls be backfilled with on-site or imported non-expansive, granular (sandy) soils. On-site silty or clayey soils should not be used as wall backfill.

Active earth pressures can be used for designing walls that can yield at least 1-inch laterally in 10 feet of wall height under the imposed loads. For level backfill comprised of on-site or imported granular soils (no more than 40 percent passing No. 200 U.S. standard sieve), the magnitude of active pressures are equivalent to the pressures imposed by a fluid weighing 35 pounds per cubic foot (pcf). The Structural Engineer should indicate on the plans the type of backfill recommended above.

At-rest pressures should be used for restrained walls that remain rigid enough to be essentially non-yielding. At-rest pressures for on-site or imported granular soils are equivalent to the pressures imposed by a fluid weighing 52 pounds per cubic foot.

Walls subject to surcharge loads should be designed for an additional uniform lateral pressure equal to one-third and one-half the anticipated surcharge pressure for unrestrained and restrained walls, respectively.

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

Retaining wall footings should be designed as discussed in the *"Shallow Foundations"* section.

4.8 CORROSIVITY

Resistivity testing (Appendix C) of representative samples of the on-site soils indicates that they are severely corrosive to metals. Should the use of buried metallic structures be proposed, a corrosion engineer such as HDR should be retained to provide recommendations to protect these elements from corrosion. GPI does not practice corrosion engineering.

4.9 SWIMMING POOL

The depth of the swimming pool is anticipated to range from approximately 3½ to 5 feet below finished grade. Due to the potential for low density soils at those depths, we recommend that the swimming pools should be supported on properly compacted subgrade soils as recommended in Section 4.3.3 of this report.

The expansion potential of the soils anticipated in the upper 5 feet of the soil profile is very low.

Lateral earth pressures provided in Section 4.6 of this report should be used, as a minimum, for the swimming pool design. Pool walls should be designed to resist lateral surcharge pressures from adjacent footings or structures in addition to lateral earth pressures. If the surcharge load is at least 5 feet or the depth of the pool, whichever is greater, away from the edge of pool, the surcharge load can be ignored.

The soils within the bottom and side excavations should be moistened and maintained at elevated moisture contents of 1 to 2 (sands) percentage points above optimum, until the pool is constructed. We recommend that the moisture-conditioning of the pool excavation on a daily basis until placement of shotcrete. Caving of the sandy soils may occur if moisture is not maintained.

We recommend a representative of GPI observe and approve the soils within the pool excavation immediately prior to covering the soil with shotcrete for the pool structure.

The pool deck, including concrete flatwork and pavers, if used, should be properly drained in order to help prevent excessive infiltration or seepage into the underlying subgrade soils.

Measures should be taken to reduce the potential for water to infiltrate into the subgrade soils around the pool. Such measures should include proper sealant of construction joints in the pool decking and surrounding hardscape.

4.10 DRAINAGE

Positive surface gradients should be provided adjacent to structures so as to direct surface water run-off and roof drainage away from foundations and slabs toward suitable discharge facilities. Long-term ponding of surface water should not be allowed on pavements or adjacent to buildings.

Careful efforts should be made to provide methods of transporting runoff water to collection devices and preventing runoff water from infiltrating soils within the vicinity of the buildings or other improvements. Proper measures should also be take reduce the potential for future pipe leakage and/or landscape irrigation infiltration into the soils near the buildings or other proposed improvements.

4.11 STORMWATER INFILITRATION

Current regulations require that stormwater be infiltrated in the site soils of new developments when possible. GPI performed an infiltrations study for the proposed development and presented the results in a report dated June 16, 2023 (GPI, 2023).

4.12 EXTERIOR CONCRETE FLATWORK (PEDESTRIAN HARDSCAPE)

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

4.13 PAVED AREAS

Based on the soils encountered near the existing grades in our explorations and our understanding that the upper soils onsite will be replaced with imported sandy soils, we have assumed an R-value of 30 for the preliminary design of new pavements. Based on the subgrade soils anticipated, we recommend the following pavement sections for the various levels of traffic (traffic indices) anticipated:

ASPHALT CONCRETE PAVEMENT

PAVEMENT AREA	TRAFFIC INDEX	SECTION THICKNESS (inches)	
		Asphalt Concrete	**Aggregate Base Course**
Auto Parking	4	3	4
Circulation Drives	5	3	6
Truck Drives	6	3.5	8

PORTLAND CEMENT CONCRETE PAVEMENT

PAVEMENT AREA	TRAFFIC INDEX	SECTION THICKNESS (inches)	
		f'c = 3,500 psi PCC	**Aggregate Base**
Auto Parking	4	6.0	---
Circulation Drives	5	6.5	---
Truck Drives	6	7.5	---

If vehicular pavers are to be used for the project, the paver and leveling sand may be supported on the thickness of aggregate base for flexible pavements shown above for the appropriate traffic index. Pavers for vehicular traffic should be a minimum of $3^1/_8$ inches (80 mm) thick.

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

The pavement base course (as well as the top 12 inches of the subgrade soils) 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, excluding processed miscellaneous base.

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

4.14 GEOTECHNICAL OBSERVATION AND TESTING

We recommend that a representative of GPI observe all earthwork during construction to confirm that the recommendations provided in our report are applicable during construction.

The earthwork activities include grading, compaction of fills and subgrade preparation, as well as foundation construction. 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 Iron Lofts, 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 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 express or implied, is included or intended in our report.

Respectfully submitted,
Geotechnical Professionals Inc.

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REFERENCES

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

California Geological Survey, 1998, Seismic Hazard Zone Report for the Anaheim 7.5-minute Quadrangle, California, Seismic Hazard Zone Report 003.

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

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

City of Riverside, Community Development Department, “Liquefaction Zones, Riverside General Plan, 2025, Public Safety Element”, Figure PS-2, plan dated November 2007.

Geotechnical Professionals Inc (GPI, 2014) “ Proposed Mission Lofts, SEC Mission Inn Avenue and Commerce Street, Riverside California” Project No. 2591.I dated February 17, 2014

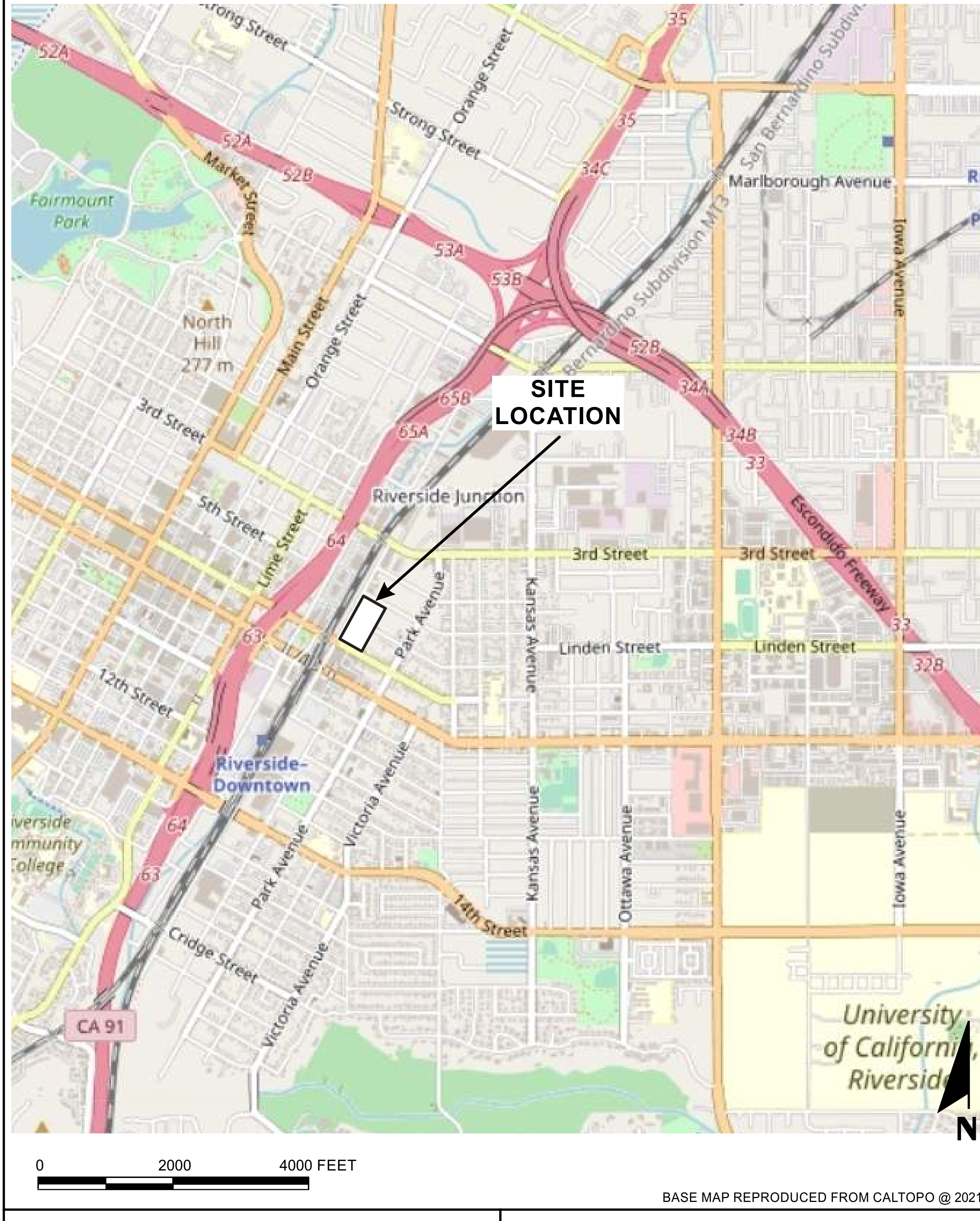
Geotechnical Professionals Inc (GPI, 2023) “ Infiltration Study, Proposed Residential Development – Iron Lofts, NEC Mission Inn Avenue and Commerce Street, Riverside, California” Project No. 3044.1I dated June 16, 2023

GSI Environmental, 2020, “Revised Remedial Action Plan, Riverside Scrap Iron & Metal, 2993 Sixth Street, Riverside, California 2507” GSI Job No. 4790, dated January 2, 2020

http://www.historicaerials.com, Aerial Photography from the Past and Present”, National Environmental Title Research, LLC.

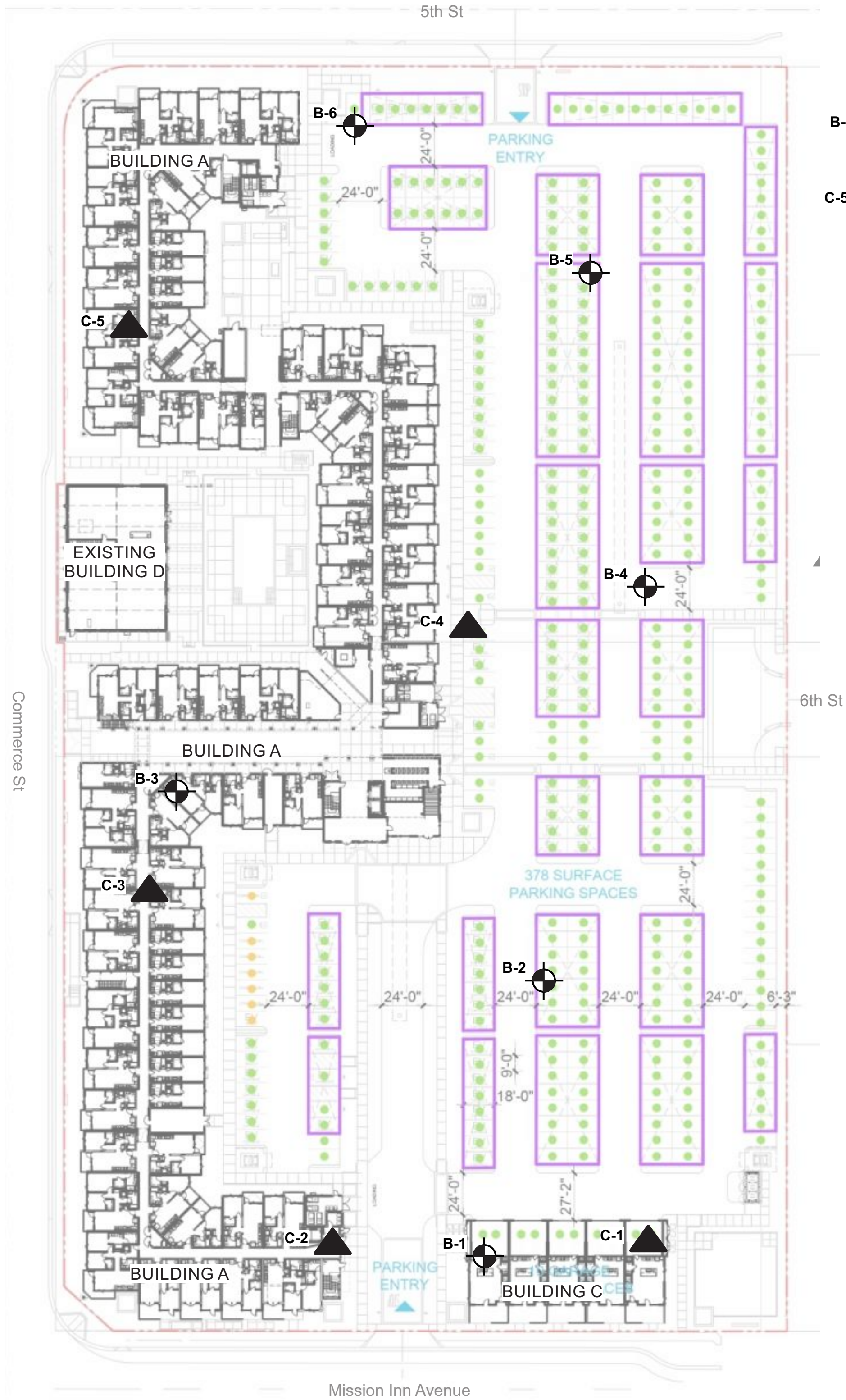
United States Geologic Survey, Earthquake Ground Motion Parameters Application, http://earthquake.usgs.gov/research/hazmaps/design

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



BASE MAP REPRODUCED FROM CALTOPO @ 2021

GPI GEOTECHNICAL PROFESSIONALS, INC.		SITE LOCATION MAP
IRON LOFTS		
GPI PROJECT NO.: 3044.I	SCALE: 1" = 2000'	FIGURE 1



EXPLANATION

B-6 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING

C-5 APPROXIMATE LOCATION AND NUMBER OF CONE PENETRATION TEST

BASE PLAN REPRODUCED FROM SITE PLAN (A1-3) PROVIDED BY KTGY ARCHITECTURE DATED: 11-08-24

GEOTECHNICAL PROFESSIONALS, INC.

IRON LOFTS

GPI PROJECT NO.: 3044.I | SCALE: 1" = 100'

SITE PLAN
(Proposed)

FIGURE 2

EXPLANATION

B-6 APPROXIMATE LOCATION AND NUMBER EXPLORATORY BORING

C-5 APPROXIMATE LOCATION AND NUMBER OF CONE PENETRATION TEST

N

0 100 200 FEET

BASE PLAN REPRODUCED FROM GOOGLE EARTH DATED: 03-20-25

GEOTECHNICAL PROFESSIONALS, INC.

IRON LOFTS

GPI PROJECT NO.: 3044.I

SCALE: 1" = 100'

SITE PLAN
(Existing)

FIGURE 3

APPENDIX A

APPENDIX A

CONE PENETRATION TESTS

The subsurface conditions were investigated by performing five Cone Penetration Tests (CPTs) at the site. The CPT's were advanced to depths of about 50 feet below existing grades. The locations of the CPTs are shown on Site Plans, Figures 2 and 3.

The Cone Penetration Test consists of pushing a cone-tipped probed 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 CPTs described in this report were conducted in general accordance with ASTM specifications (ASTM D5778) using an electric cone penetrometer.

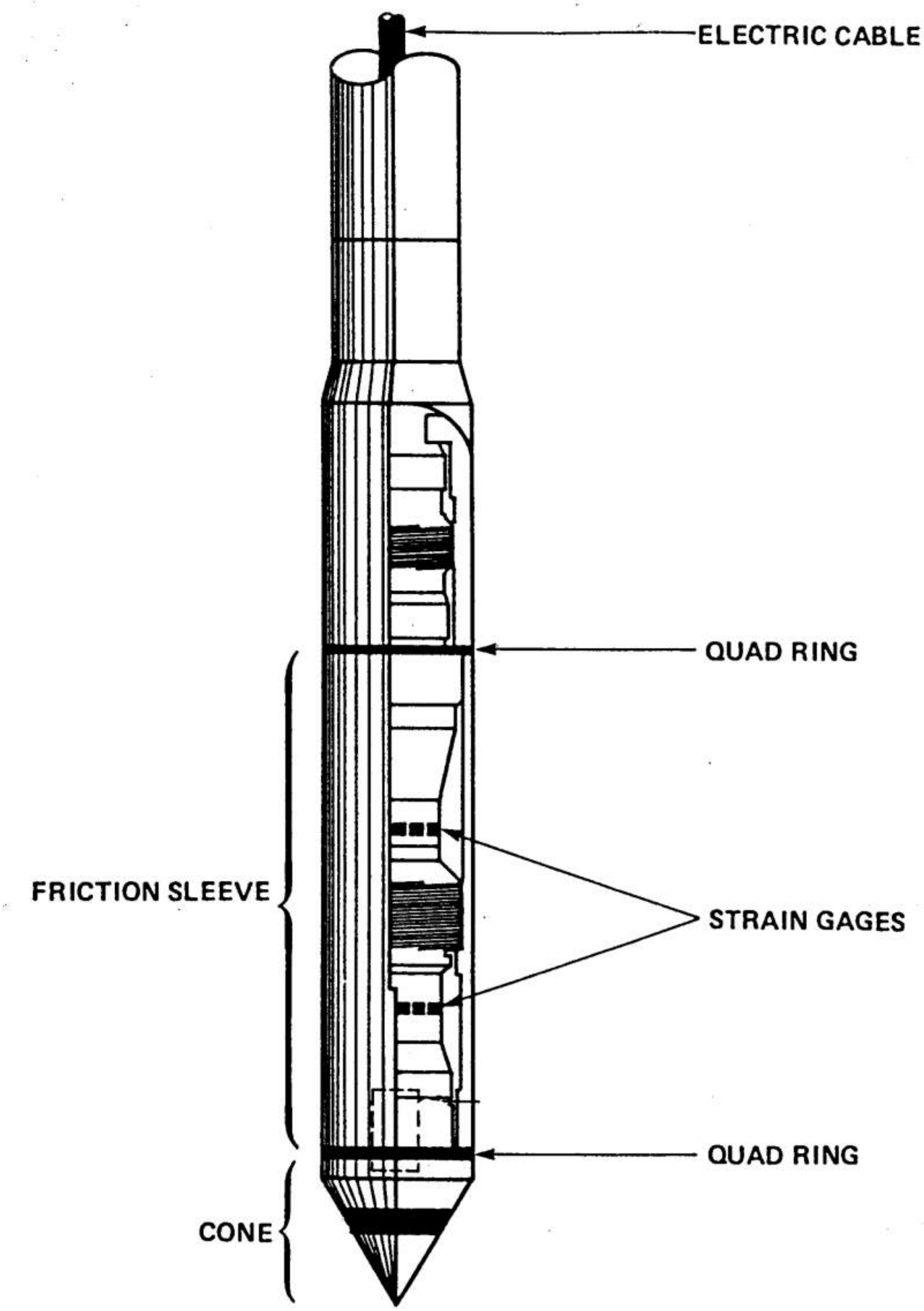
The CPT equipment consists of a cone assembly mounted at the end of a series of hollow sounding rods. A set of hydraulic rams is used to push the cone and rods into the soil while a continuous record of cone and friction resistance versus depth is obtained in both analog and digital form at the ground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

Standard 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 is presented in Figures A-2 to A-6 of this appendix. The field testing and computer processing was performed by Kehoe Testing and Engineering under subcontract to Geotechnical Professionals Inc. (GPI). The interpreted soils descriptions were prepared by GPI.

The CPT locations were laid out in the field by measuring from existing site features. Upon completion, the un-caved portions of the CPT holes were backfilled with bentonite chips.

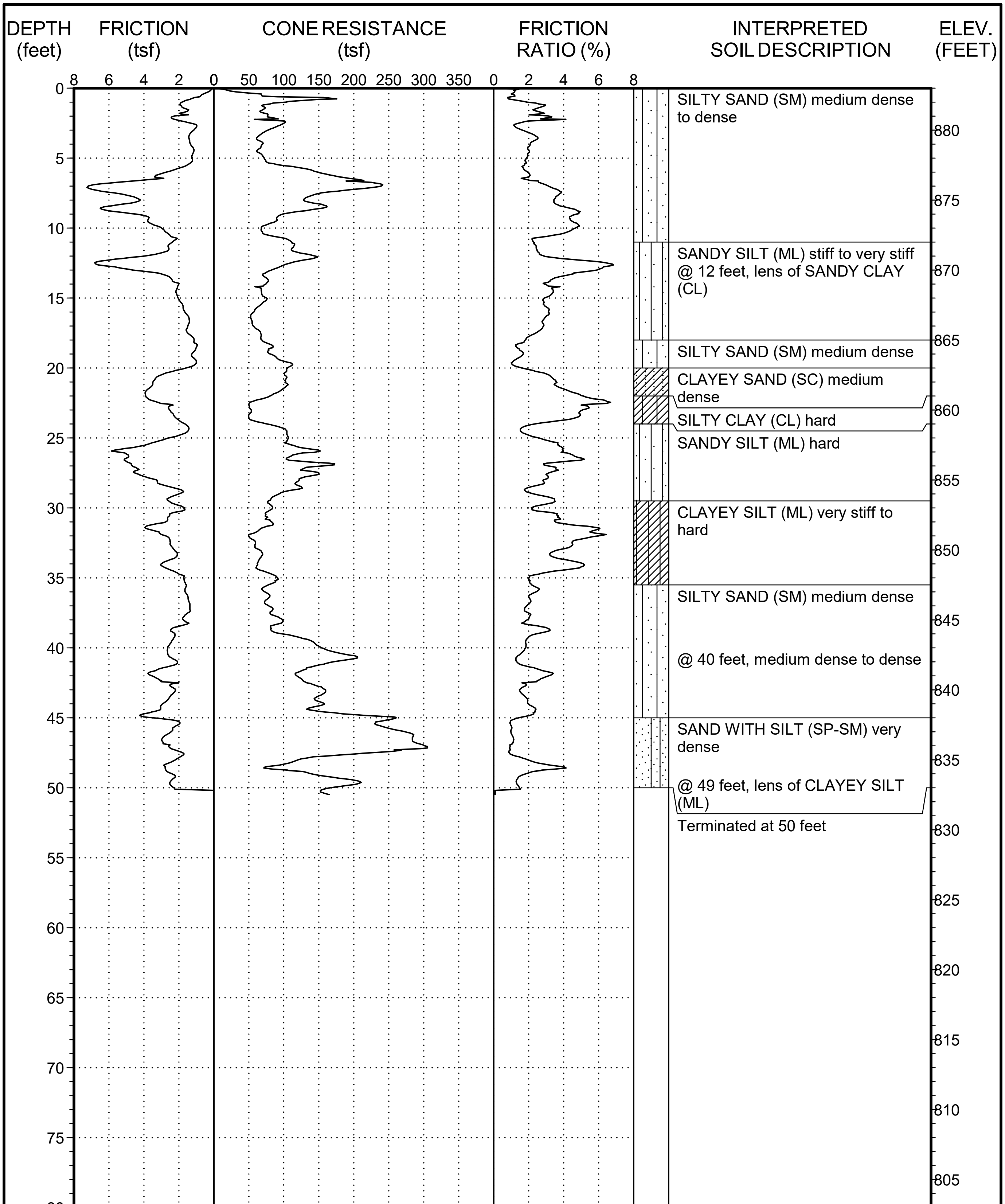
Ground surface elevations at the exploration locations were estimated from the preliminary grading plan by KHR Associates and should be considered approximate.



GEOTECHNICAL
PROFESSIONALS, INC.

CONE PENETROMETER

FIGURE A-1



Date performed: 12-16-22

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.

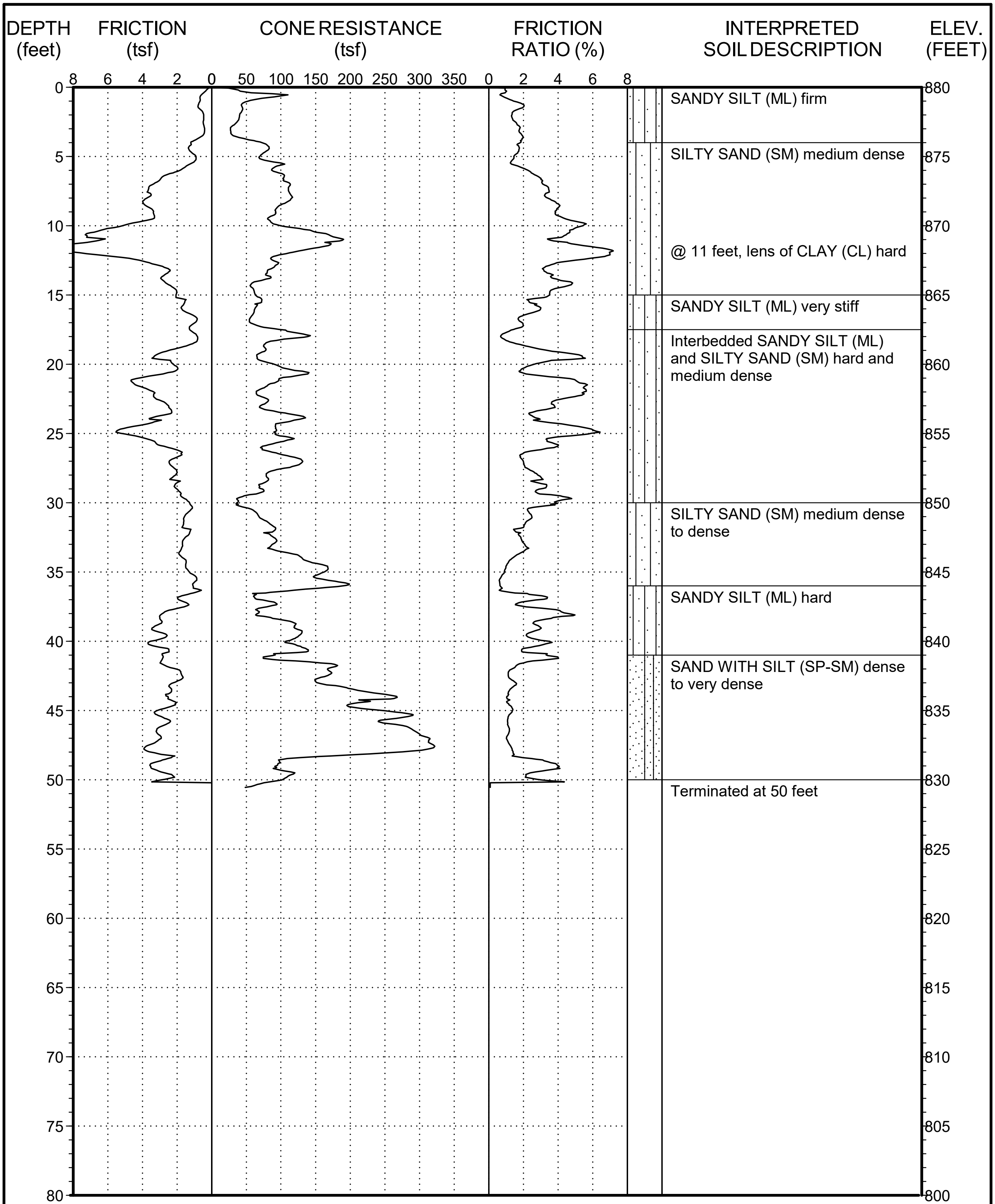
GPI

PROJECT NO.: 3044.1I

IRON LOFTS

LOG OF CPT NO. C-1

FIGURE A-2



Date performed: 12-16-22

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.

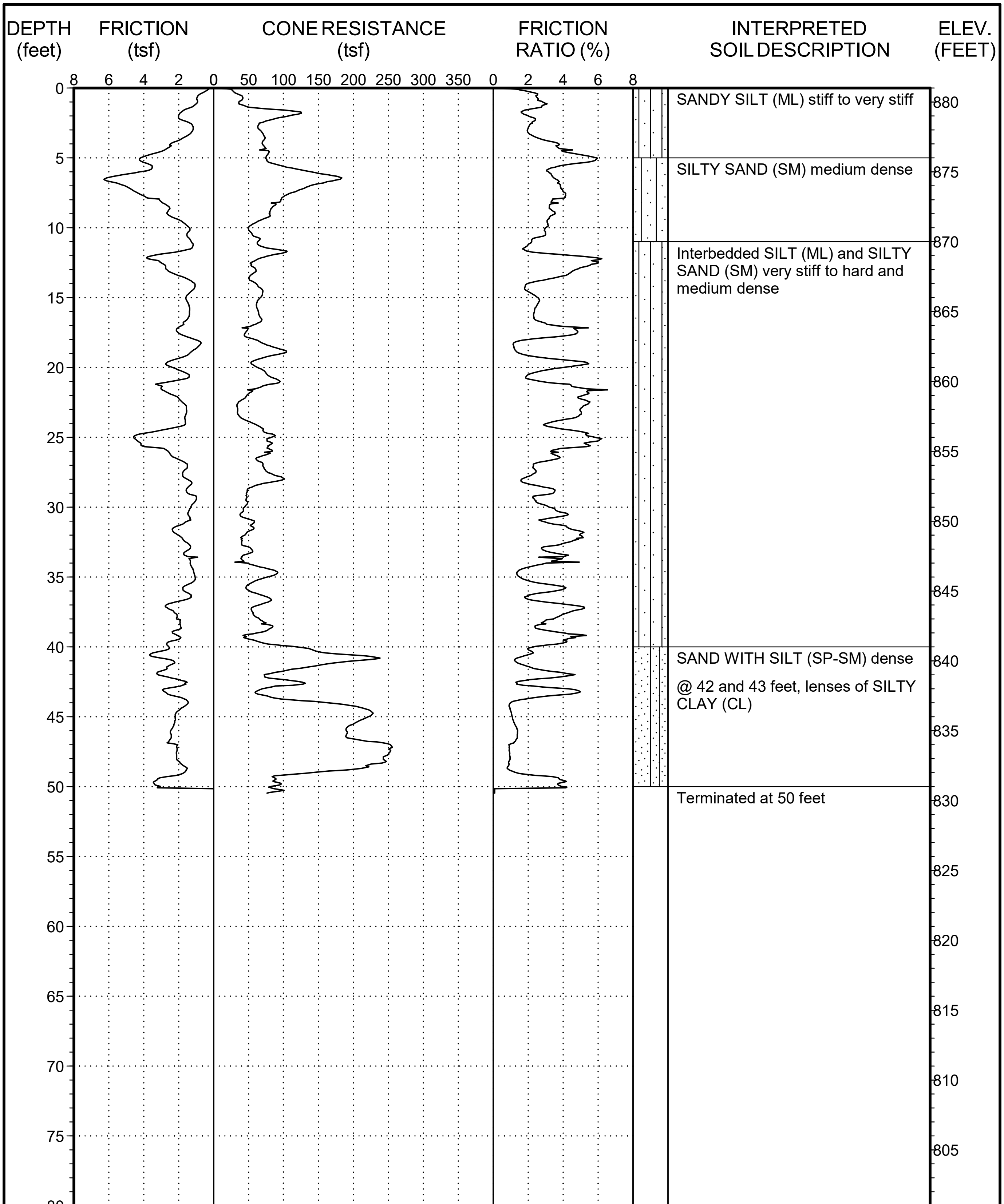
GPI

PROJECT NO.: 3044.1I

IRON LOFTS

LOG OF CPT NO. C-2

FIGURE A-3



Date performed: 12-16-22

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.

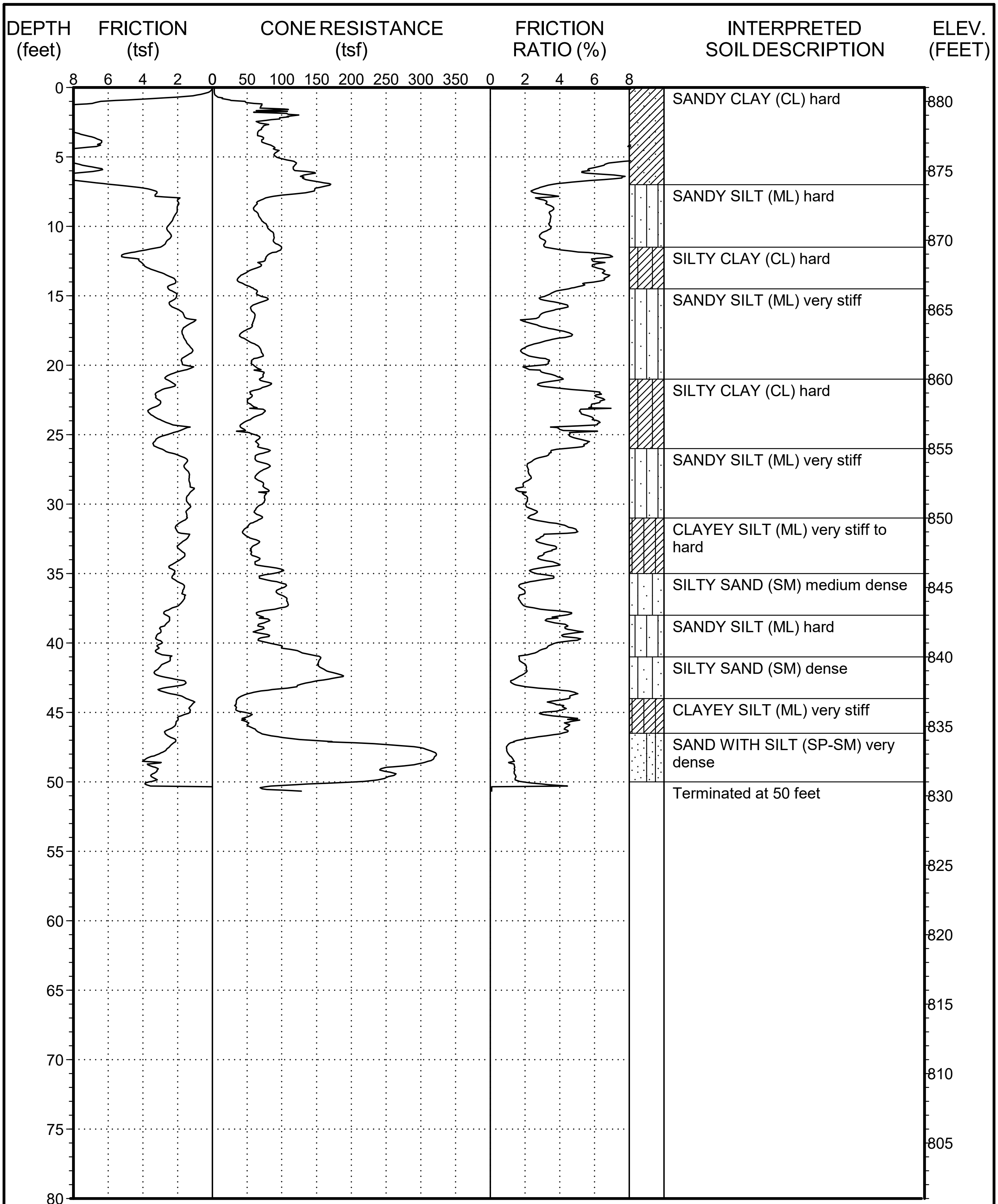
GPI

PROJECT NO.: 3044.1I

IRON LOFTS

LOG OF CPT NO. C-3

FIGURE A-4



Date performed: 12-16-22

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.

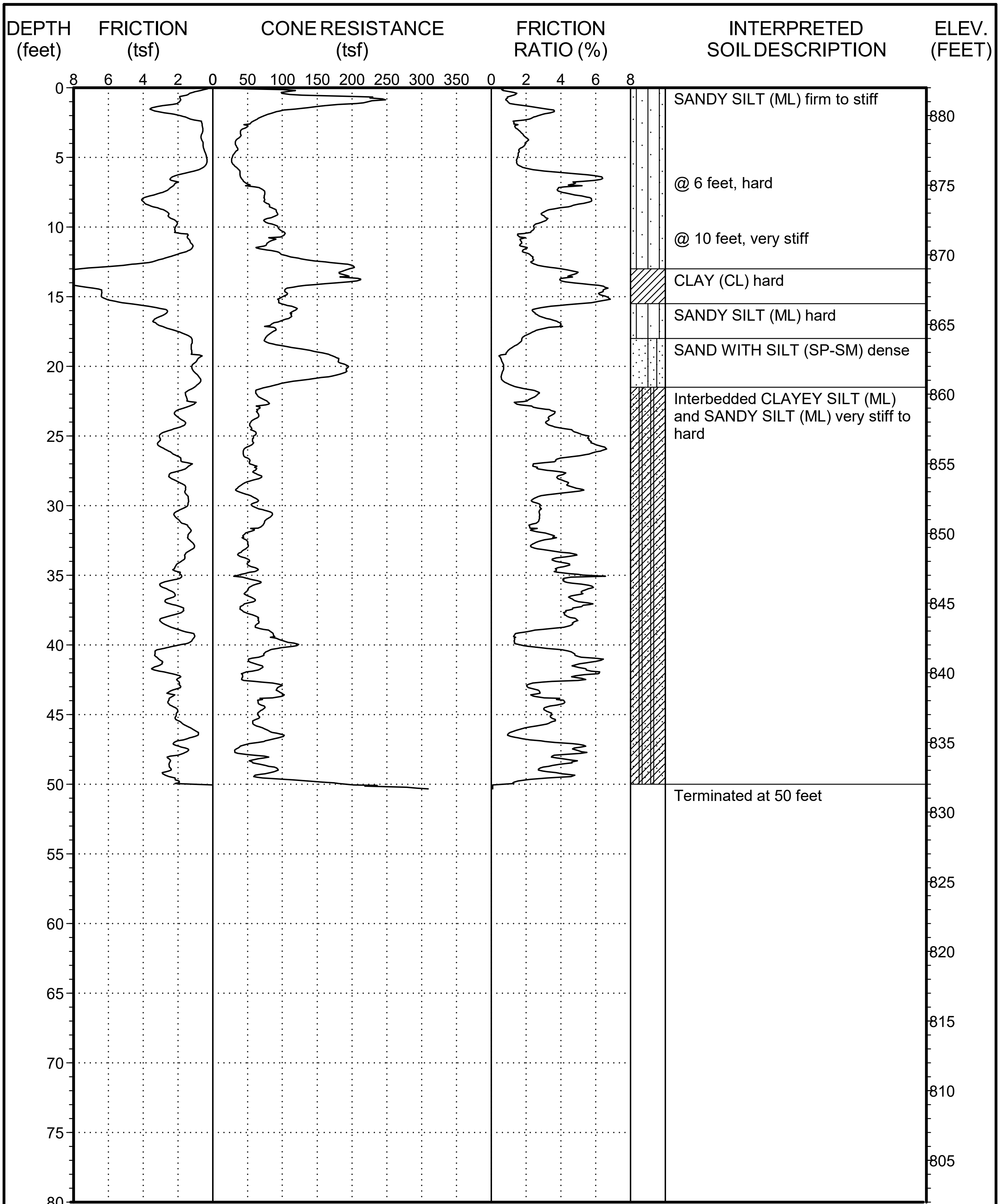
GPI

PROJECT NO.: 3044.1I

IRON LOFTS

LOG OF CPT NO. C-4

FIGURE A-5



Date performed: 12-16-22

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.: 3044.1I

IRON LOFTS

LOG OF CPT NO. C-5

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 ranging from 16 to 31 feet below the existing ground surface. The locations of the explorations are shown on the Site Plans, Figures 2 and 3.

The exploratory borings were drilled using truck mounted hollow-stem auger drill equipment. Relatively undisturbed samples were obtained using a brass-ring lined sampler (ASTM D3550). The brass-rings have an inside diameter of 2.42 inches. The ring samples were driven into the soil by a 140-pound hammer dropping 30 inches. The number of blows needed to drive the sampler into the soil was recorded as the penetration resistance.

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 locations were laid out in the field by measuring from existing site features. Ground surface elevations at the exploration locations were estimated from the preliminary grading plan by KHR Associates 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.	
					0	Fill: **SILTY SAND (SM)** brown	880
	9.1	108	20	D	5	Natural: **SANDY SILT (ML)** brown, slightly moist, stiff	875
	12.6	106	27	D	10	**SILT (ML)** brown, moist, very stiff, with sand	870
	12.2	105	19	D	15	**SANDY SILT (ML)** brown, moist, stiff	865
	11.7	113	26	D	20	**SILTY SAND (SM)** brown, moist, medium dense	860
	18.6	106	18	D	25	**SANDY SILT (ML)** brown, wet, stiff	855
	14.7	107	15	D	30		
						Total Depth 31 feet	

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

DATE DRILLED:
12-27-22
EQUIPMENT USED:
8 " HOLLOW STEM AUGER
GROUNDWATER LEVEL (ft):
NOT ENCOUNTERED

GPI

PROJECT NO.: 3044.1I

IRON LOFTS

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	ELEVATION (FEET)
					0	Fill: **SILTY SAND (SM)** brown	
	9.1	90	15	D	5	Natural: **SILTY SAND (SM)** brown, moist, loose to medium dense	875
	19.8		50/5"	D		@ 7 feet, wet, very dense	
	11.1	108	37	D	10	**SANDY SILT (ML)** brown, moist, very stiff	870
	10.3	103	17	D	15	@ 15 feet, stiff	865
						Total Depth 16 feet	

This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

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

DATE DRILLED: 12-27-22
EQUIPMENT USED: 8 " HOLLOW STEM AUGER
GROUNDWATER LEVEL (ft): NOT ENCOUNTERED

GPI

PROJECT NO.: 3044.1I
IRON LOFTS

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)
				0	Fill: **SILTY SAND (SM)** orange-brown	
						875
8.7	102	50/6"	D	5	Natural: **SANDY SILT (ML)** orange-brown, slightly moist, hard	
11.1	100	40	D		@ 7 feet, moist, very stiff	
						870
10.9	113	56	D	10	@ 10 feet, brown, hard	
						865
9.7	101	18	D	15	@ 15 feet, slightly moist, stiff	
						860
9.9	111	33	D	20	@ 20 feet, slightly moist to moist, very stiff	
					SILTY SAND (SM) brown, moist, medium dense	
						855
15.8	102	34	D	25	**SANDY SILT (ML)** brown, very moist, very stiff	
						850
17.5	92	22	D	30	@ 30 feet, stiff to very stiff	
					Total Depth 31 feet	

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

DATE DRILLED:
12-27-22

EQUIPMENT USED:
8 " HOLLOW STEM AUGER

GROUNDWATER LEVEL (ft):
NOT ENCOUNTERED

GPI

PROJECT NO.: 3044.1I

IRON LOFTS

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 This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
					0	Fill: **SILTY SAND (SM)** brown, moist, dense	
							880
	8.9	101	52	D	5		
	13.3	97				Natural: **SANDY SILT (ML)** brown, moist, hard	
	7.3	104	26	D		@ 7 feet, slightly moist, very stiff	
							875
	7.0	100	25	D	10	**SILT (ML)** brown, slightly moist, very stiff, with sand	
							870
	8.7	96	19	D	15	@ 15 feet, stiff	
						Total Depth 16 feet	

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

DATE DRILLED:
12-27-22

EQUIPMENT USED:
8 " HOLLOW STEM AUGER

GROUNDWATER LEVEL (ft):
NOT ENCOUNTERED

PROJECT NO.: 3044.1I

IRON LOFTS

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	ELEVATION (FEET)
					0	Fill: **SILTY SAND (SM)** brown, slightly moist	
							880
	11.1	110	18	D	5	Natural: **SANDY SILT (ML)** brown, moist, stiff	
	11.3	96	7	D		@ 7 feet, firm	
							875
	12.9	101	19	D	10	@ 10 feet, stiff	
							870
	2.6	105	23	D	15	**SILTY SAND (SM)** brown, dry, medium dense, coarse grained	
						SAND (SP) light brown, dry, medium dense	
						Total Depth 16 feet	

This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

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

DATE DRILLED: 12-27-22
EQUIPMENT USED: 8 " HOLLOW STEM AUGER
GROUNDWATER LEVEL (ft): NOT ENCOUNTERED

GPI

PROJECT NO.: 3044.1I
IRON LOFTS

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)
					0	Fill: **SILTY SAND (SM)** brown	
							880
	13.9	115	34	D	5	Natural: **SILTY SAND (SM)** brown, very moist, medium dense	
	9.8	108	47	D		**SANDY SILT (ML)** brown, slightly moist, hard	875
	9.9	113	64	D	10	**SILTY SAND (SM)** brown, moist, dense	
							870
	5.5	109	19	D	15	@ 15 feet, slightly moist, medium dense	
							865
	16.0	99	21	D	20	**SANDY SILT (ML)** brown, very moist, stiff	
						SILTY SAND (SM) brown, very moist to wet, medium dense	860
	10.9	109	25	D	25	**SANDY SILT (ML)** brown, moist, very stiff	
							855
	9.6	114	29	D	30	@ 30 feet, slightly moist	
						Total Depth 31 feet	

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

DATE DRILLED:
12-27-22

EQUIPMENT USED:
8 " HOLLOW STEM AUGER

GROUNDWATER LEVEL (ft):
NOT ENCOUNTERED

GPI

PROJECT NO.: 3044.1I

IRON LOFTS

LOG OF BORING NO. B-6

FIGURE B-6

APPENDIX C

APPENDIX C

LABORATORY TESTS

INTRODUCTION

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

Test results are presented in the figures that follow.

MOISTURE CONTENT AND DRY DENSITY

Moisture content and dry density were determined from a number of the ring samples. The samples were first trimmed to obtain volume and wet weight and then were dried in accordance with ASTM D2216. After drying, the weight of each sample was measured, and moisture content and dry density were calculated. Moisture content and dry density values are presented on the boring logs in Appendix B.

PERCENT PASSING NO. 200 SIEVE

Selected soil samples were dried, weighed, soaked in water until individual soil particles were separated, and then washed on the No. 200 sieve. That portion of the material retained on the No. 200 sieve was oven-dried and weighed to determine the percentage of the material passing the No. 200 sieve. The percentages passing the No. 200 sieve are tabulated below.

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	PERCENT PASSING No. 200 SIEVE
B-1	10	Silt (ML) with sand	75
B-3	10	Sandy Silt (ML)	70
B-4	10	Silt (ML) with sand	76
B-6	25	Sandy Silt (ML)	53

DIRECT SHEAR

Direct shear tests were performed on undisturbed and remolded samples in accordance with ASTM D3080. The remolded samples were remolded to approximately 90 percent of maximum density (ASTM D1557). The samples were placed in the shear machine, and a normal load comparable to the in-situ overburden stress was applied. The samples were inundated, allowed to consolidate, and then were sheared to failure. The tests were repeated on additional test specimens under increased normal loads. Shear stress and sample deformation were monitored throughout the test. The results of the direct shear tests are presented in Figures C-1 to C-3.

HYDROCONSOLIDATION

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

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	TOTAL COLLAPSE (%)
B-1	15	Silty Sand (SM)	0.2
B-6	7	Sandy Silt (ML)	0.1

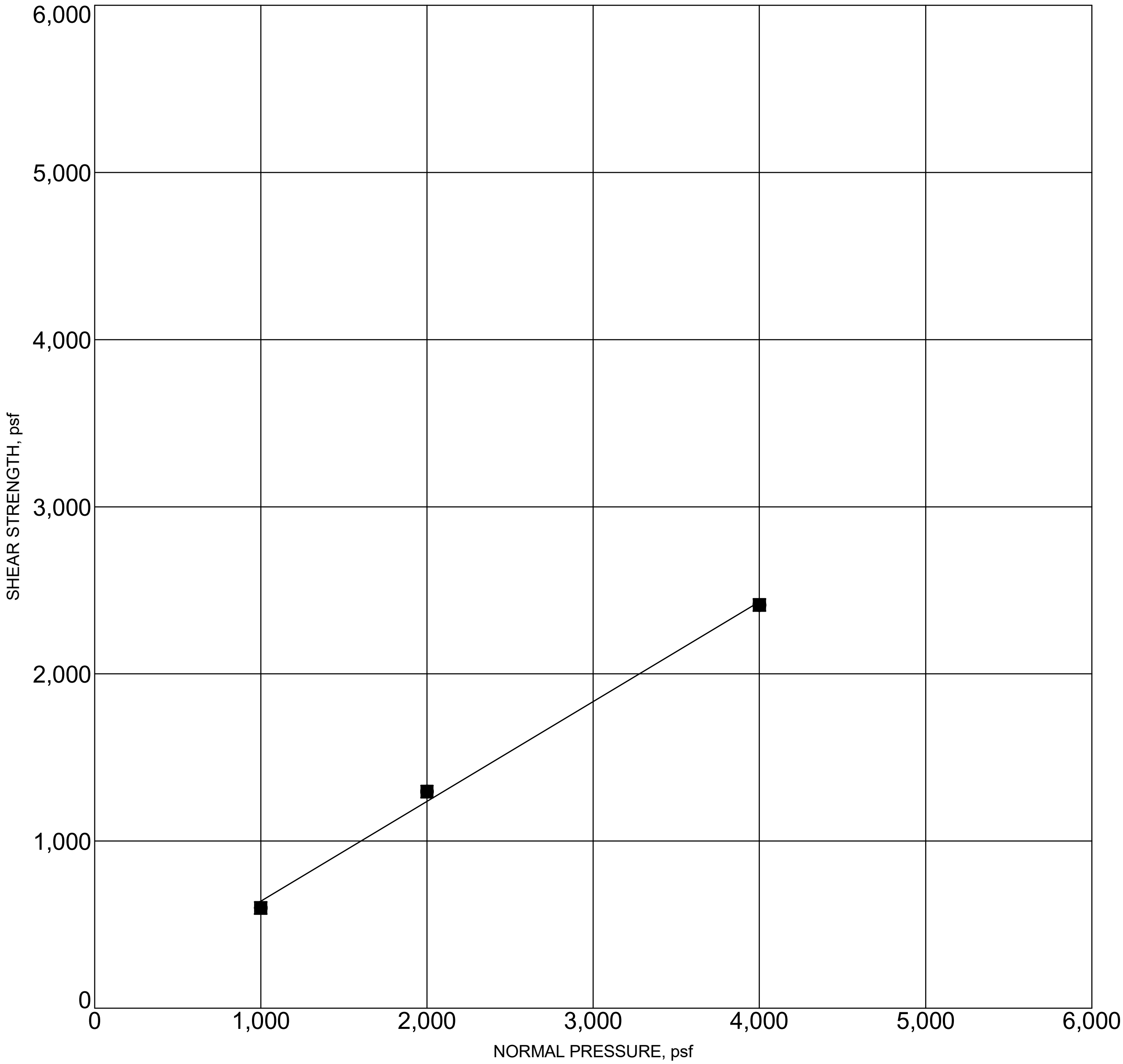
COMPACTION TEST

A maximum dry density/optimum moisture test was performed in accordance with ASTM D1557 on a representative bulk sample of the site soils. The test results are as follows:

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)
Combined B-3,4,5,6	5-7	Silty Sand (SM)	131	8

CORROSIVITY

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



● **PEAK STRENGTH**
Friction Angle= 31 degrees
Cohesion= 42 psf

ULTIMATE STRENGTH
Friction Angle= 31 degrees
Cohesion= 42 psf

Sample Location		Classification	DD,pcf	MC,%
B-2	5.0	SILTY SAND (SM)	90	9.1

PROJECT: IRON LOFTS

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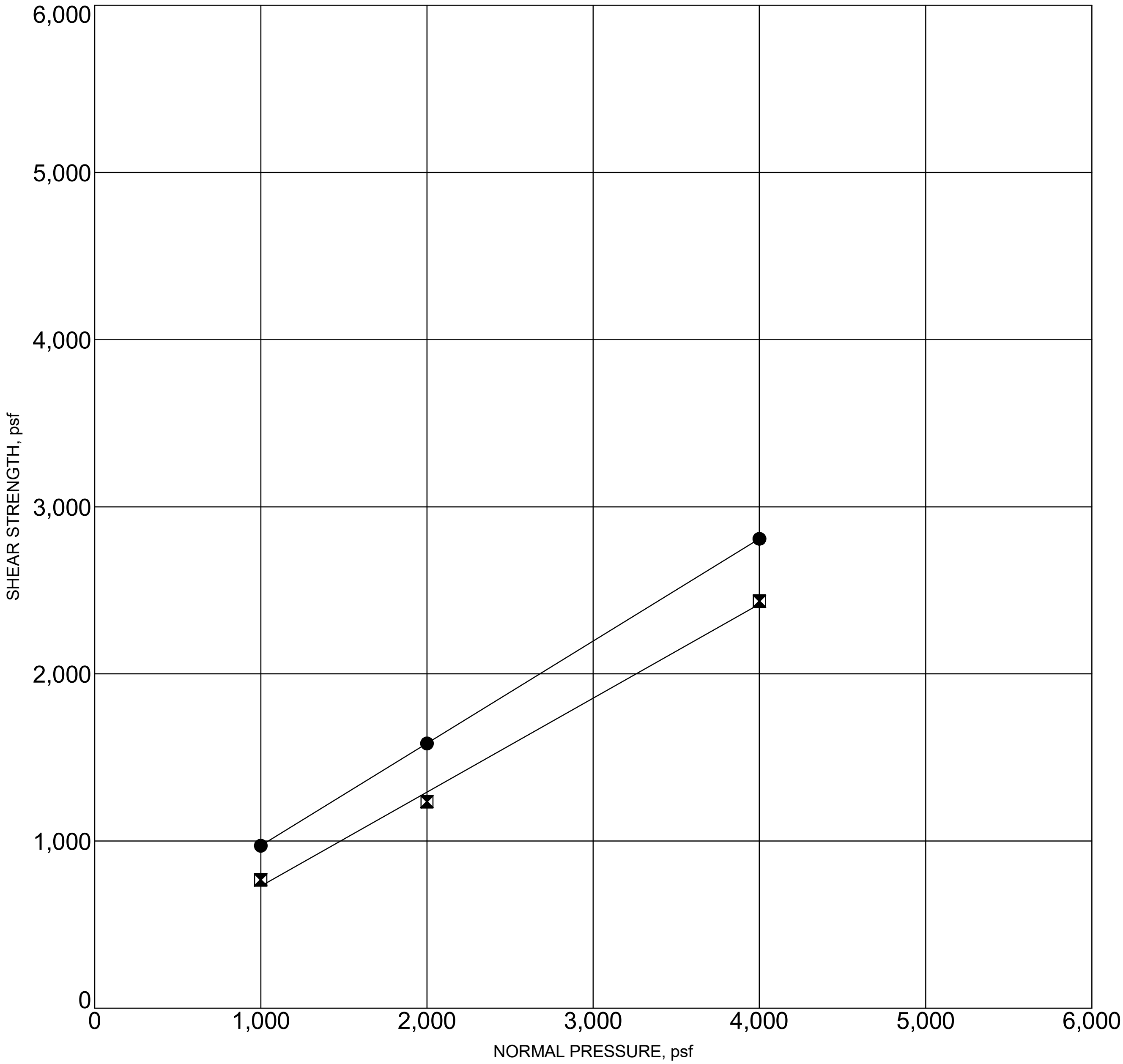
DIRECT SHEAR TEST RESULTS

FIGURE C-1



● PEAK STRENGTH
Friction Angle= 31 degrees
Cohesion= 360 psf

⊠ ULTIMATE STRENGTH
Friction Angle= 29 degrees
Cohesion= 168 psf

Sample Location		Classification	DD,pcf	MC,%
B-6	10.0	SANDY SILT (ML)	113	9.9

PROJECT: IRON LOFTS PROJECT NO.:3044.1I

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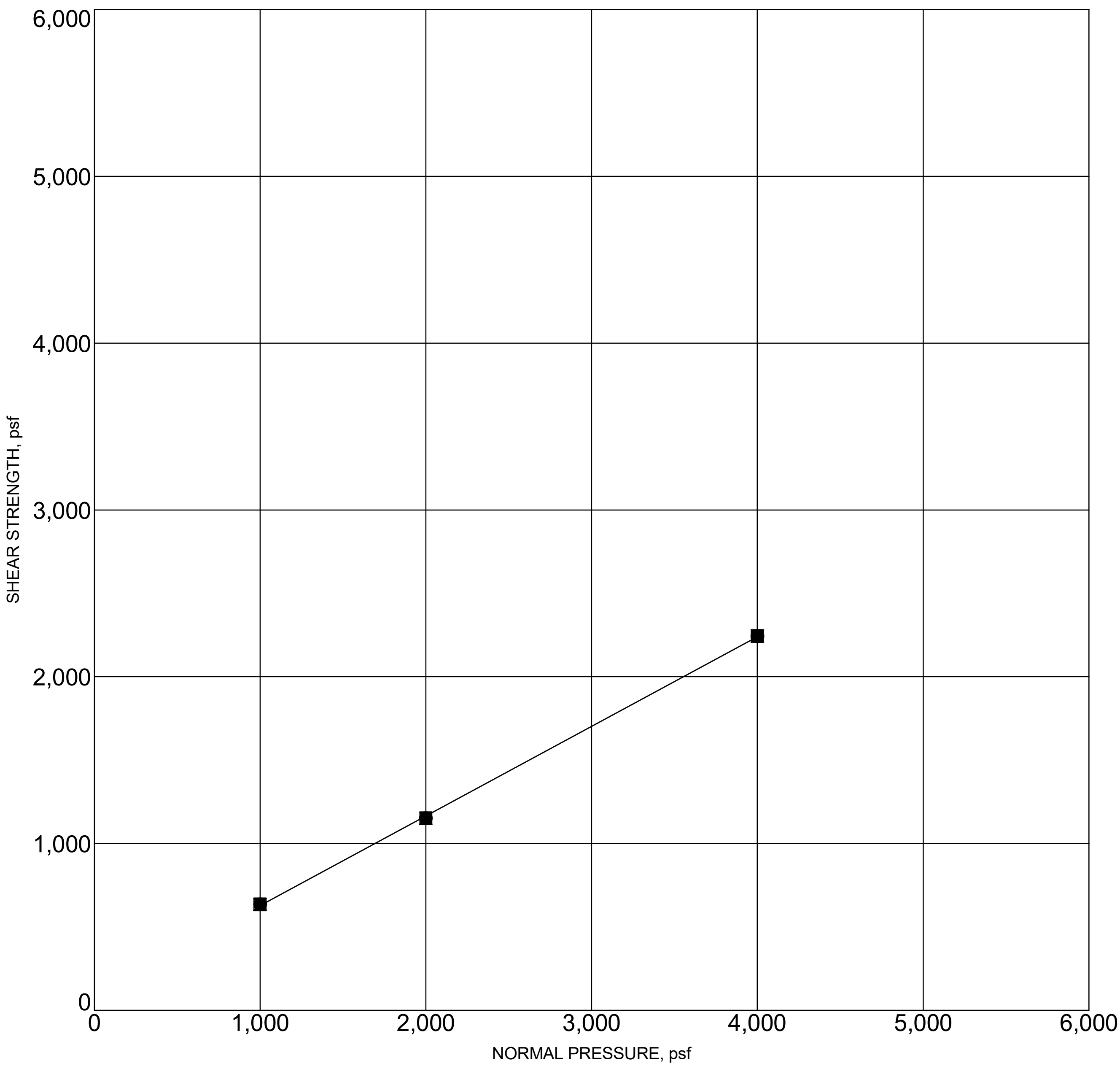
DIRECT SHEAR TEST RESULTS

FIGURE C-2



● **PEAK STRENGTH**
Friction Angle= 28 degrees
Cohesion= 90 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 28 degrees
Cohesion= 90 psf

Note: Samples remolded to 90% of maximum dry density

Sample Location	Classification	DD,pcf	MC,%
B-3/4/5/6 5-7	SANDY SILT (ML)	112	11.0

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DIRECT SHEAR TEST RESULTS

FIGURE C-3

Table 1 - Laboratory Tests on Soil Samples

Geotechnical Professionals, Inc.
IRON LOFTS
Your #3044.1I, HDR Lab #23-0013LAB
10-Jan-23

Sample ID			
			B-3 @ 5
Resistivity		**Units**	
as-received		ohm-cm	15,200
saturated		ohm-cm	800
pH			7.0
Electrical Conductivity		mS/cm	0.87
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	321
magnesium	Mg^{2+}	mg/kg	295
sodium	Na^{1+}	mg/kg	272
potassium	K^{1+}	mg/kg	9.0
ammonium	NH_4^{1+}	mg/kg	ND
Anions			
carbonate	CO_3^{2-}	mg/kg	ND
bicarbonate	HCO_3^{1-}	mg/kg	149
fluoride	F^{1-}	mg/kg	2.3
chloride	Cl^{1-}	mg/kg	173
sulfate	SO_4^{2-}	mg/kg	1,890
nitrate	NO_3^{1-}	mg/kg	2.2
phosphate	PO_4^{3-}	mg/kg	ND
Other Tests			
sulfide	S^{2-}	qual	na
Redox		mV	na

Resistivity per ASTM G187, pH per ASTM G51, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.

Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.

mg/kg = milligrams per kilogram (parts per million) of dry soil.

Redox = oxidation-reduction potential in millivolts

ND = not detected

na = not analyzed