GEOTECHNICAL
PROFESSIONALS INC.

GEOTECHNICAL INVESTIGATION
PROPOSED BEHAVIORAL HEALTH CAMPUS
RIVERSIDE UNIVERSITY HEALTH SYSTEM
PERRIS WELLNESS VILLAGE
NEC WATER STREET AND HARVILL AVENUE
PERRIS, CALIFORNIA

Prepared for:
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Prepared by:
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Project No. 3194.2I

November 10, 2023 (revised December 20, 2023)

GEOTECHNICAL
PROFESSIONALS INC.

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PMB LLC
329 South Highway 101, Suite 160
Solana Beach, California 92075

Attention: Mr. Ben Rosenfeld
SVP, Development

Subject: Report of Geotechnical Investigation
Proposed Behavioral Health Campus
Riverside University Health System
Perris Wellness Village
NEC Water Street and Harvill Avenue
Perris, California
GPI Project No. 3194.2I

Dear Mr. Rosenfeld:

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

Further wet signed copies of the report can be provided upon request if required for County submittal.

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

Very truly yours,
Geotechnical Professionals Inc.

James E. Harris V, P.E.
Project Engineer

Donald A. Cords, G.E.
Principal

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1.0 INTRODUCTION

1.1 GENERAL

This report presents the results of the geotechnical investigation performed by Geotechnical Professionals Inc. (GPI) for the proposed Behavioral Health Campus in Perris, California. The geographical site location is shown on the Site Location Map, Figure 1.

1.2 PROJECT DESCRIPTION

We understand the proposed development will consist of one 2-story building (Urgent Care Services), two 3-story buildings (Extended Residential Care and Children and Youth Services), and two 4-story buildings (Community Wellness and Education Center and Supportive Transitional Housing) on a 19.4-acre site. A small single-story storage building is also planned just east of the Urgent Care building. Other site improvements will include basketball courts, pickleball courts, a fountain, play areas, a small amphitheater, trash enclosures, pavements, flatwork, and landscape areas. The proposed site layout is shown on the Site Plan, Figure 2.

The provided site plan also shows a future 2-story building (Administrative Building) near the northeast corner of the property with future parking along the northern limits of the property.

Based on information provided by James Chen, the Project Structural Engineer, column loads for the building will be on the order of 350 to 500 kips with the exceptions of the Supportive Transitional Housing and storage building. These structures are planned to be supported on continuous footings. The maximum wall loads for the housing building are expected to be on the order of 5 kips per lineal foot. The maximum wall loads for the storage building are expected to be on the order of 1 to 2 kips per lineal foot. The buildings are expected to be comprised of steel frame or engineered metal stud construction with slab-on-grade floors. We expect grades at the site to remain within approximately 1 to 3 feet of existing grades.

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 planning earthwork, and design of foundations, floor slabs, and pavements.

2.0 SCOPE OF WORK

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

The field exploration program consisted of twenty Cone Penetration Test (CPTs) and sixteen exploratory borings. The locations of the subsurface explorations are shown on the Site Plan, Figure 2.

The CPT's were advanced to practical refusal with depths ranging from 15½ to 50 feet below existing site grades. Detailed logs of the CPT's and a summary of the equipment used are presented in Appendix A. The exploratory borings were drilled using 8-inch diameter truck-mounted hollow-stem auger drilling equipment and hand auger equipment. The borings were advanced to depths ranging from 6 to 41 feet below existing site grades. Details of the drilling and Logs of Borings are presented in Appendix B.

Laboratory soil tests were performed on selected representative samples as an aid in soil classification and to evaluate the engineering properties of the soils. The geotechnical laboratory testing program included determinations of moisture content and dry density, grain size distribution, hydroconsolidation, shear strength (direct shear), compaction, subgrade strength, expansion potential, and corrosion. Laboratory testing procedures and results are summarized in Appendix C.

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

Terra Geosciences performed a seismic shear-wave survey to assess the average shear wave velocity of the subsurface soils. The result of the testing and the report by Terra Geosciences are presented in Appendix D.

Engineering evaluations were performed provide geotechnical and foundation recommendations. The results of our evaluations are presented in the remainder of this report.

3.0 SITE CONDITIONS

3.1 SURFACE CONDITIONS

The site is currently unoccupied and covered by vegetation. According to historic aerials (historicaerials.com), the site was occupied by multiple small buildings in the northwest corner and southwest corner between 1959 and 2014. The site is bounded on the north by Placentia Avenue, on the east by Harvill Avenue, on the south by Water Street, and on the west by a mostly undeveloped property with a few small structures in the northeast corner.

The ground surface at the site slopes very gently to the northeast. Based on Google Earth, ground surface elevations at the site are approximately +1540 feet at the southwest and slopes to the north east to approximately +1510 feet.

3.2 SUBSURFACE SOILS

Our field investigation disclosed a subsurface profile consisting of undocumented fill overlying natural soils. Detailed descriptions of the conditions encountered are shown on the Logs of Borings in Appendix B. A brief summary of the subsurface conditions are provided below.

The fill soils extended to depths of approximately 2 to 5 feet below existing grades at the locations of our exploratory borings. The undocumented fill soils consist of silty sands and clayey sands. The fills were generally dry to slightly moist. Documentation regarding the placement and compaction of the fill was not provided.

The natural soils consisted of silty sand and sands in the upper 20 feet. These soils were generally medium dense to dense and dry to slightly moist. From approximately 20 feet to 50 feet below the existing ground surface, the soils consist of silty sands and clayey sands with some interbedded layers of clay silts and silty clays. The sandy soils are dense to very dense and dry to moist. The fine-grained soils are hard. The natural soils exhibit moderate strength and low compressibility characteristics.

3.3 GROUNDWATER AND CAVING

Groundwater was not encountered in our explorations to a depth of 50 feet below the existing grade. This area has not been mapped by the State of California.

Groundwater was measured by others in a well located approximately 1.1-miles east of the site at an elevation of approximately +1,407 feet in March of 2023 (wdl.water.ca.gov).

Caving was not observed in our borings and should be considered unlikely.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 GENERAL

Based on the results of our investigation, it is our opinion that from a geotechnical engineering viewpoint it is feasible to develop the site as proposed. The most significant geotechnical issues that will affect the design and construction of the proposed structures are as follows:

- The proposed building may be supported on spread and continuous foundations with slab-on-grade floors. Footings and slabs should be supported on properly compacted fill.
- Prior to construction of the building foundations, undocumented fills and disturbed soils should be removed and replaced as properly compacted fill. The depth of removals and details regarding grading are provided in the “Earthwork” section of this report.
- Removals are also recommended in the pavement for drives and parking, athletic courts, and under minor structures, in order to provide a consistent, moist layer of engineered fill for uniform support. The depth of removals and details regarding grading are provided in “Earthwork” section of this report.
- The on-site soils should be considered moderately corrosive to buried metals. If buried metal elements are required, a corrosion engineer should be consulted.

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

4.2 SEISMIC CONSIDERATIONS

4.2.1 General

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

We assume the seismic design of the proposed development will be in accordance with the California Building Code (CBC) 2022 edition. Analysis of the data obtained during the seismic shear-wave survey indicated that the “weighted average” shear-wave velocity within the upper 100 feet of the subject site, V_{s30}, is 1,628 feet per second (fps). In accordance with ASCE 7-16, this shear-wave velocity corresponds to a Site Class C (Very Dense Soil and Soft Rock). 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:

2022 CBC:

S_S = 1.49g	$S_{MS} = F_a * S_S$ = 1.79g	S_{DS} = 2/3 * S_{MS} = 1.19g
S_1 = 0.55g	$S_{M1} = F_V * S_1$ = 0.80g	S_{D1} = 2/3 * S_{M1} = 0.53g

Note: The Project Structural Engineer should confirm these values prior to their use.

4.2.2 Strong Ground Motion Potential

During the life of the project, the site will likely be subject to strong ground motions due to earthquakes on nearby faults. Based on the ASCE 7 website (asce7hazardtool.online), we computed that the site could be subjected to a peak ground acceleration (PGA_M) of 0.60g for a magnitude 7.0 earthquake. This acceleration has been computed using the mapped Maximum Considered Geometric Mean peak ground acceleration from ASCE 7-16 (ASCE, 2016) 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

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 not located within an area that has been mapped for liquefaction by the State of California. The site is located within an area shown as having a low potential for soil liquefaction as determined by the County of Riverside (koordinates.com). Soil liquefaction is not likely to occur at the project site due to the deep groundwater.

4.3 EARTHWORK

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

4.3.1 Clearing

Prior to grading, the areas to be developed should be stripped of vegetation, pavements, foundations, and cleared of debris. Buried obstructions, such as utilities and tree roots, should be removed. Although none were encountered, any cesspools or septic systems exposed during construction 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 a lean sand-cement slurry. Deleterious materials generated during the clearing operations should be removed from the site. At the conclusion of the clearing operations, a representative of GPI should observe and accept the site prior to further grading.

4.3.2 Excavations

Excavations at the site will include removal of undocumented fills, disturbed soils, a portion of the natural soils, foundation excavations and trenching for utility lines.

Prior to placement of fills or construction of the buildings, undocumented fills, disturbed soils, and a portion of the natural soils within the building areas should be removed and replaced as properly compacted fill. These materials require densification to provide uniform and adequate support of foundations, slab-on-grade floors, and pavements.

For planning purposes, we recommend that removals within footprints of proposed buildings supported on spread footings extend to 5 feet below existing grades or 1 foot below the bottom of footings, whichever is deeper. Deeper removals may be required to remove undocumented fills and disturbed soils.

In proposed athletic courts removals should extend to at least 2-feet below existing grades or proposed subgrade, whichever is deeper, to provide moist, uniform support for pavements.

In proposed pavement areas, removals should extend to at least 1-foot below existing grades or proposed subgrade, whichever is deeper, to provide moist, uniform support for pavements. Existing grade refers to elevations at locations of explorations.

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.

The actual depths of removal will need to be confirmed in the field during grading by a representative of GPI.

The removals should extend laterally beyond the building lines by at least 5 feet or beyond the edge of footing a minimum distance equal to the depth of overexcavation/compaction below <u>finish</u> grade (i.e., a 1:1 projection below the edge of footings), whichever is greater.

Where not removed by the aforementioned excavations, existing utility trench backfill should be removed and replaced as properly compacted fill. This is especially important for deeper fills such as existing sewers and storm drains. For planning purposes, removals over the utilities should extend to within 1-foot of the top of the pipe. For utilities, which are 5 feet or shallower, the removal should extend laterally 1-foot beyond both sides of the pipe. For deeper utilities, the removals should include a zone defined by a 1:1 projection upward (and away from the pipe) from each side of the pipe. The actual limits of removal will be confirmed in the field. We recommend that all known utilities be shown on the grading plan.

Temporary construction excavations may be made vertically without shoring to a depth of 5 feet below adjacent grade. For deeper cuts up to 15 feet, the slopes should be properly shored or sloped back to at least 1:1 or flatter. 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 any adjacent existing site facilities, should be properly shored to maintain support of adjacent elements. Excavations and shoring systems should meet the minimum requirements given in the most current State of California Occupational Safety and Health Standards.

4.3.3 Subgrade Preparation

Prior to placing fills, the exposed subgrade should be scarified to a depth of 12 inches, moisture conditioned and compacted within the footprints of the proposed buildings. The exposed subgrade surface should be proof-rolled a minimum of 6 passes using a vibratory pad foot roller excreting a dynamic force of at least 40,000 pounds. The subgrade should be compacted to a minimum of 95 percent (90 percent for silts and clays) of maximum density in accordance with ASTM D 1557.

We recommend significant moistening of the subgrade soils to help mitigate soils susceptible to hydroconsolidation beneath removals and to facilitate in-place compaction as the moisture content of the soils is generally well below optimum moisture. Moistening can usually be accomplished by deep ripping and liberal watering (including “rainbirds” or flooding) prior to compaction.

4.3.4 Material for Fill

The on-site soils are, in general, suitable for use as compacted fill under the structures.

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

Soils used for compacted fills should not contain particles greater than 6 inches in size.

If encountered, on-site inert demolition debris, such as concrete and asphalt, may be reused in the compacted fills provided approval is obtained from the reviewing regulatory agency and the owner. The material should be crushed to the consistency of aggregate base and blended with the on-site or imported soils.

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 95 percent (90 percent for silts and clays) of the 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
Track Equipment, 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.

Granular fills should be placed at a moisture content of 0 to 2 percent over the optimum moisture content. While not anticipated, fills consisting of the on-site clays and silts should be placed at a moisture content of 1 to 3 percent over the optimum moisture content in order to achieve the required compaction. In general, the moisture content of the soils encountered in the upper 5 to 10 feet of the explorations was generally well under the optimum moisture content. As such, significant moisture conditioning (wetting) will be required prior to replacing the soils as properly compacted fill. The contractors should allow for moistening of these materials in their bids.

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

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

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 about 10 to 15 percent and subsidence of 0.2 feet may be assumed for the surficial soils. 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 and wall backfill consisting of the on-site material or imported sandy soils should be mechanically compacted in lifts. Jetting or flooding should not be permitted. Significant moistening of the on-site soils should be anticipated prior to backfill. Lift thickness should not exceed those values given in the "Compacted Fill" section of this report. GPI should observe and test trench and wall backfills as they are placed.

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

4.3.8 Observation and Testing

A representative of GPI should observe excavations, subgrade preparation, and fill placement activities. Sufficient in-place field density tests should be performed during fill placement and in-place compaction to evaluate the overall compaction of the soils. Soils that do not meet minimum compaction requirements should be reworked and tested prior to placement of any additional fill.

4.4 SHALLOW FOUNDATIONS

4.4.1 Foundation Type

The proposed buildings may be supported on spread and continuous foundations. The subsurface soils should be 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 natural and recompacted on-site soils, a static allowable net bearing pressure of up to 6,000 pounds per square foot (psf) may be used for both continuous footings and isolated column footings for the building. These bearing pressures are for dead-plus-live-loads, and may be increased by 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 trash enclosures and site walls along property lines, where reduced excavation limits are required, we recommend a maximum allowable bearing capacity of 1,500 pounds per square foot be used with minimum footing widths and depths of 15 inches.

4.4.3 Minimum Footing Width and Embedment

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

STATIC BEARING PRESSURE (psf)	MINIMUM FOOTING WIDTH (inches)	MINIMUM FOOTING* EMBEDMENT (inches)
6,000	72	48
5,000	72	36
4,000	48	36
3,500	48	24
3,000	36	24
2,500	18	24
2,000	18	18
1,500	15	15

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

A minimum footing width of 15 inches should be used even if the actual bearing pressure is less than 1,500 psf.

4.4.4 Estimated Settlements

Total combined static and seismic settlement of the column footings (500 kips maximum column load) is expected to be less than 1-inch. Maximum differential settlements between similarly loaded adjacent footings or along a 40-foot span are expected to be less than ½-inch.

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.

For minor structures supported at-grade on properly compacted fill, total static settlement of is expected to be less than ¾-inch. Maximum differential settlements between similarly loaded adjacent footings or along a 40-foot span are expected to be less than ½-inch.

4.4.5 Lateral Load Resistance

Soil resistance to lateral loads will be provided by a combination of frictional resistance between the bottom of foundations and underlying soils, and by passive soil pressures acting against the embedded sides of the foundations. For frictional resistance, a coefficient of friction of 0.40 may be used for design. In addition, an allowable lateral bearing pressure equal to an equivalent fluid weight of 325 pounds per cubic foot may be used, provided the foundations are poured tight against the compacted fill. These values may be used in combination without reduction.

4.4.6 Footing Excavation Observation

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

4.7 FOUNDATION CONCRETE

Laboratory testing by Project X (Appendix C) indicates that the near surface soils exhibit a soluble sulfate content ranging from 32 mg/kg (0.0032 percent by weight) to 73 mg/kg

(0.0073 percent by weight). For the 2022 CBC, foundation concrete should conform to the requirements outlined in ACI 318, Section 4.3, for negligible levels of soluble sulfate exposure from the on-site soil (Category S0). Chloride levels in the samples of the upper soils ranged 12 mg/kg (0.0012 percent by weight) to 27 mg/kg (0.0027 percent by weight). For concrete exposed to soil moisture, such as footings and floor slabs, we recommend a chloride Category C1.

4.8 BUILDING FLOOR SLABS

Slab-on-grade floors should be supported on granular, non-expansive (EI $\leq$ 20), compacted soils as discussed in the "Placement and Compaction of Fills" section. Based on our explorations, granular, non-expansive soils are readily available on-site in the near-surface soils. We do not anticipate swell pressures to negatively impact the building floor slab based on the non-expansive characteristics of the on-site soils. There is not a geotechnical requirement for slab thickness or reinforcing based on the non-expansive characteristics of the on-site soils.

A vapor/moisture retarder should be placed under slabs that are to be covered with moisture-sensitive floor coverings (parquet, vinyl tile, etc.) or will be storing moisture sensitive supplies. Currently, common practice is to use a 15-mil polyolefin product such as Stego Wrap for this purpose. The need for a sand layer with the vapor barrier is not a geotechnical issue and is a decision for the Project Architect. A vapor/moisture retarder is anticipated under most of the structures.

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

For lateral resistance design, a coefficient of friction value of 0.40 between select fill and concrete may be used. For a slab on a vapor/moisture retarder, a coefficient of 0.1 should be used.

4.9 LATERAL EARTH PRESSURES

The following recommendations are provided for retaining walls or shoring less than 10 feet in height.

Active earth pressures can be used for designing cantilevered walls that can yield laterally at least ½-percent of the wall height under the imposed loads. For level, drained backfill, derived from imported granular, non-expansive soils, a lateral pressure of an equivalent fluid weighing 35 pounds per cubic foot may be used.

At-rest pressures should be used for restrained walls that remain rigid enough to be essentially non-yielding. For on-site, level, drained backfill, a lateral pressure of an equivalent fluid weighing 61 pounds per cubic foot can be used.

As outlined in the California Building Code, site retaining walls 6 feet or taller should be designed to resist seismic lateral earth pressures. A lateral pressure equivalent to a fluid with a unit weight of 25 pounds per cubic foot may be used. This pressure should be combined with the active earth pressure presented above. If the retaining walls are designed using the at-rest pressure provided above, only the difference between the active plus seismic pressures and the at-rest pressure needs to be included as the seismic pressure.

The recommended pressures are based on the assumption that the supported earth will be fully drained, preventing the build-up of hydrostatic pressures. For traditional backfilled retaining walls, a drain consisting of perforated pipe and 1 cubic foot of gravel per lineal foot, wrapped in filter fabric should be used. The fabric (non-woven filter fabric, Mirafi 140N or equivalent) should be lapped at the top.

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

The Structural Engineer should specify the use of select, granular wall backfill on the plans. Wall footings should be designed as discussed in the “Foundations” section.

4.10 CORROSIVITY

Resistivity testing of representative samples of the on-site surficial soils by Project X indicate that the soils are moderately corrosive to ferrous metals. GPI does not practice corrosion engineering. Should the use of buried metal pipe be proposed, a corrosion engineer, such as Project X, should be consulted.

4.11 DRAINAGE

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

4.12 EXTERIOR CONCRETE AND MASONRY FLATWORK

Exterior concrete and masonry flatwork should be supported on non-expansive, compacted fill. 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.

4.13 STORM WATER INFILTRATION

Current regulations require that storm water be infiltrated in 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. A separate investigation regarding infiltration has been performed and the results of that investigation will be presented in a stand-alone report.

4.14 PAVED AREAS

Preliminary pavement design has been calculated using an R-value of 40 based upon laboratory testing of the near-surface soils at the site. The California Division of Highways Design Method was used for design of the recommended preliminary pavement sections. Final pavement design should be based on R-value testing performed near the conclusion of rough grading. The following pavement sections are recommended for planning purposes only.

PAVEMENT AREA	TRAFFIC INDEX	SECTION THICKNESS (inches)	
		Asphalt Concrete	**Aggregate Base Course**
Auto Parking	4	3	4
Circulation Drives	5	3	4
Truck Drives	6	3	6.5
		Portland Cement Concrete	**Aggregate Base Course**
Auto Parking	4	5.5	---
Circulation Drives	5	5.5	---
Truck Drives	6	6	---

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

If vehicular pavers are to be used for the project, the paver and leveling sand may be supported on the thickness of aggregate base shown above for the appropriate traffic index.

The Portland cement concrete used for paving should have an approximate compressive strength of 3,500 psi at the time the pavement is subjected to truck traffic.

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.

4.15 GEOTECHNICAL OBSERVATION AND TESTING

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

5.0 LIMITATIONS

The report, exploration logs, and other materials resulting from GPI's efforts were prepared exclusively for use by PMB 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.

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 for all geotechnical aspects of the project including this report.

Our investigation and evaluations were performed using generally accepted engineering approaches and principles available at this time and the degree of care and skill ordinarily exercised under similar circumstances by reputable Geotechnical Engineers practicing in this area. No other representation, either expressed or implied, is included or intended in our report.

Respectfully submitted,
Geotechnical Professionals Inc.

James E. Harris V, P.E.
Project Engineer

Donald A. Cords, G.E.
Principal

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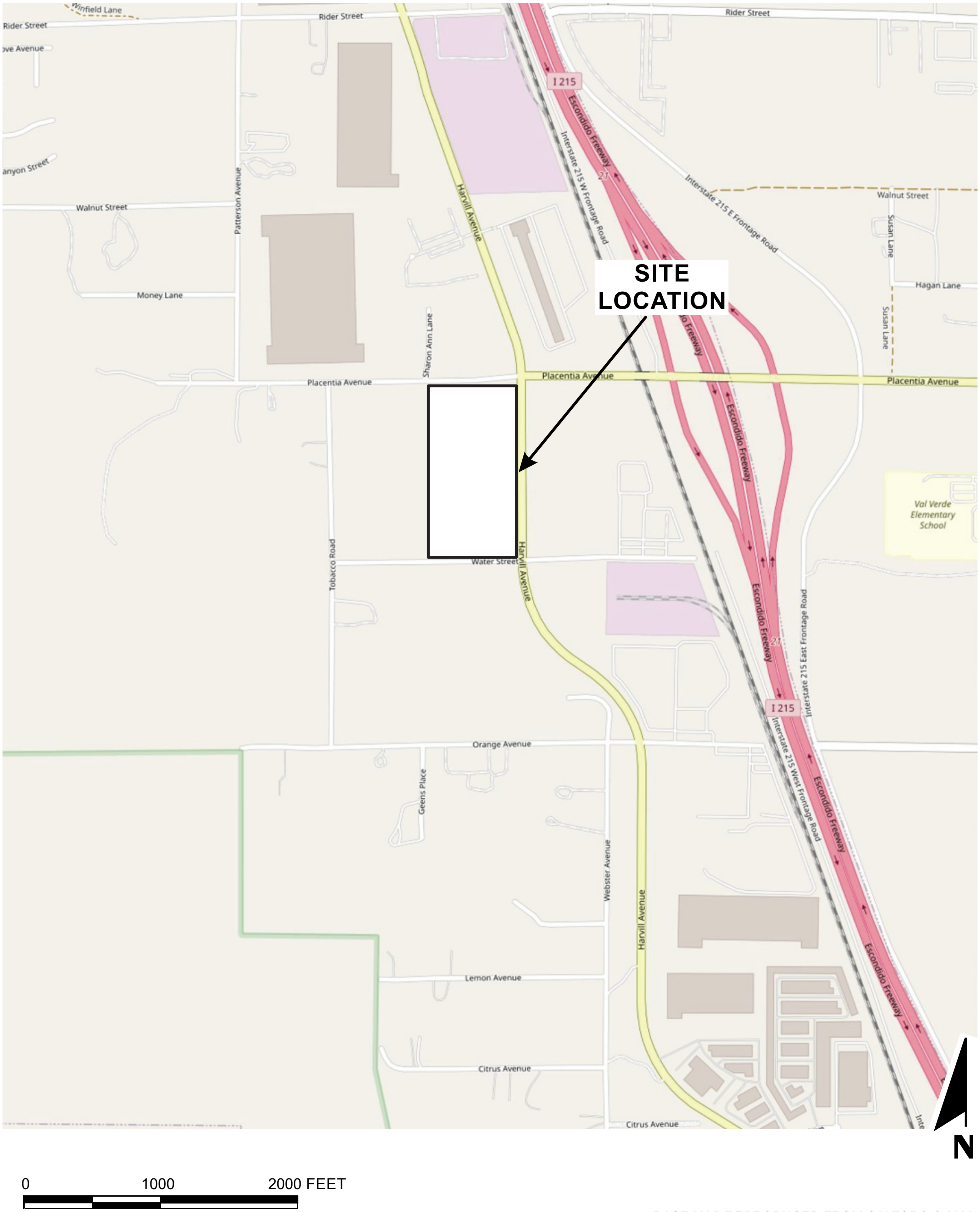

BASE MAP REPRODUCED FROM CALTOPO © 2022

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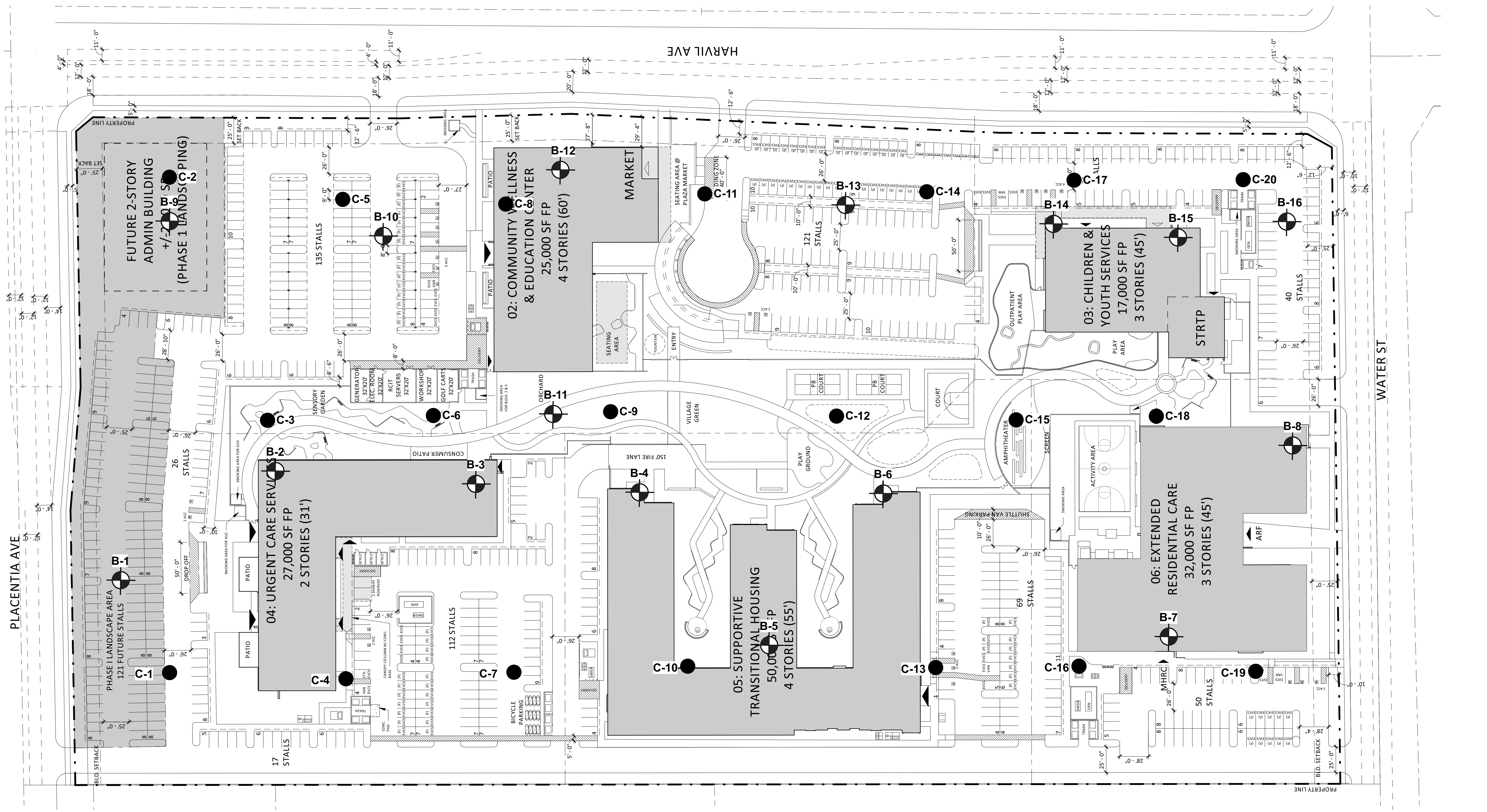
GPI PROJECT NO.: 3194.2I

SCALE: 1" = 1000'

SITE LOCATION MAP

FIGURE 1



BASE PLAN REPRODUCED FROM OVERALL SITE PLAN PROVIDED BY BOULDER ASSOCIATES DATED: 09/12/23

EXPLANATION

B-1 ⌖ APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING

C-2 ● APPROXIMATE LOCATION AND NUMBER OF CONE PENETRATION TEST

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SITE PLAN

FIGURE 2

APPENDIX A

APPENDIX A

CONE PENETRATION TESTS

The subsurface conditions were investigated by performing twenty Cone Penetration Tests (CPTs) at the site. The CPT's were advanced to depths ranging from 15½ to 50 feet below existing grades. The locations of the CPTs are shown on the Site Plan, Figure 2.

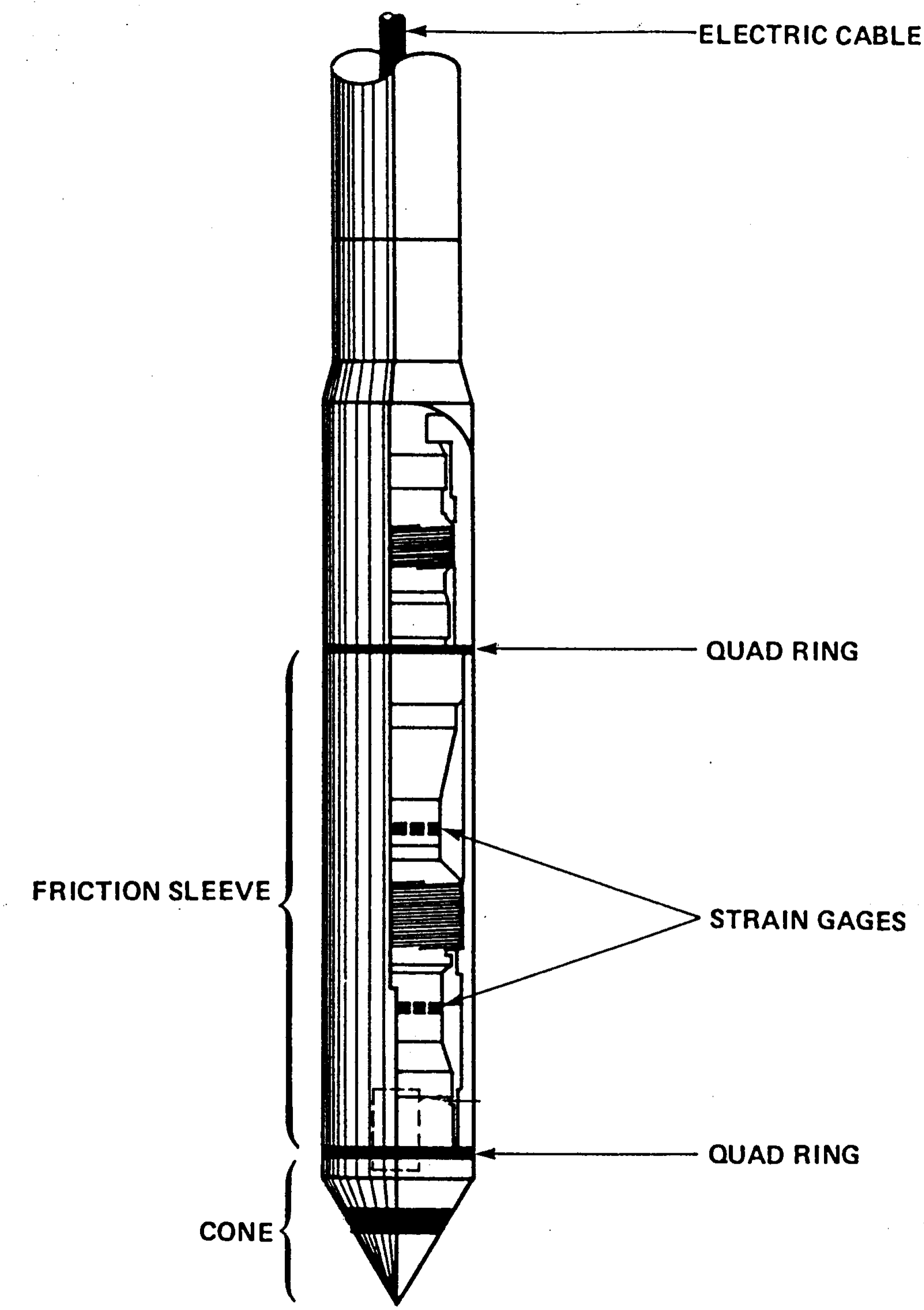
The Cone Penetration Test consists of pushing a cone-tipped probed into the soil deposit while simultaneously recording the cone tip resistance and side friction resistance of the soil to penetration (refer to Figure A-1). The CPTs described in this report were conducted in general accordance with ASTM 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 truck 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-21 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 Google Earth and should be considered very approximate.



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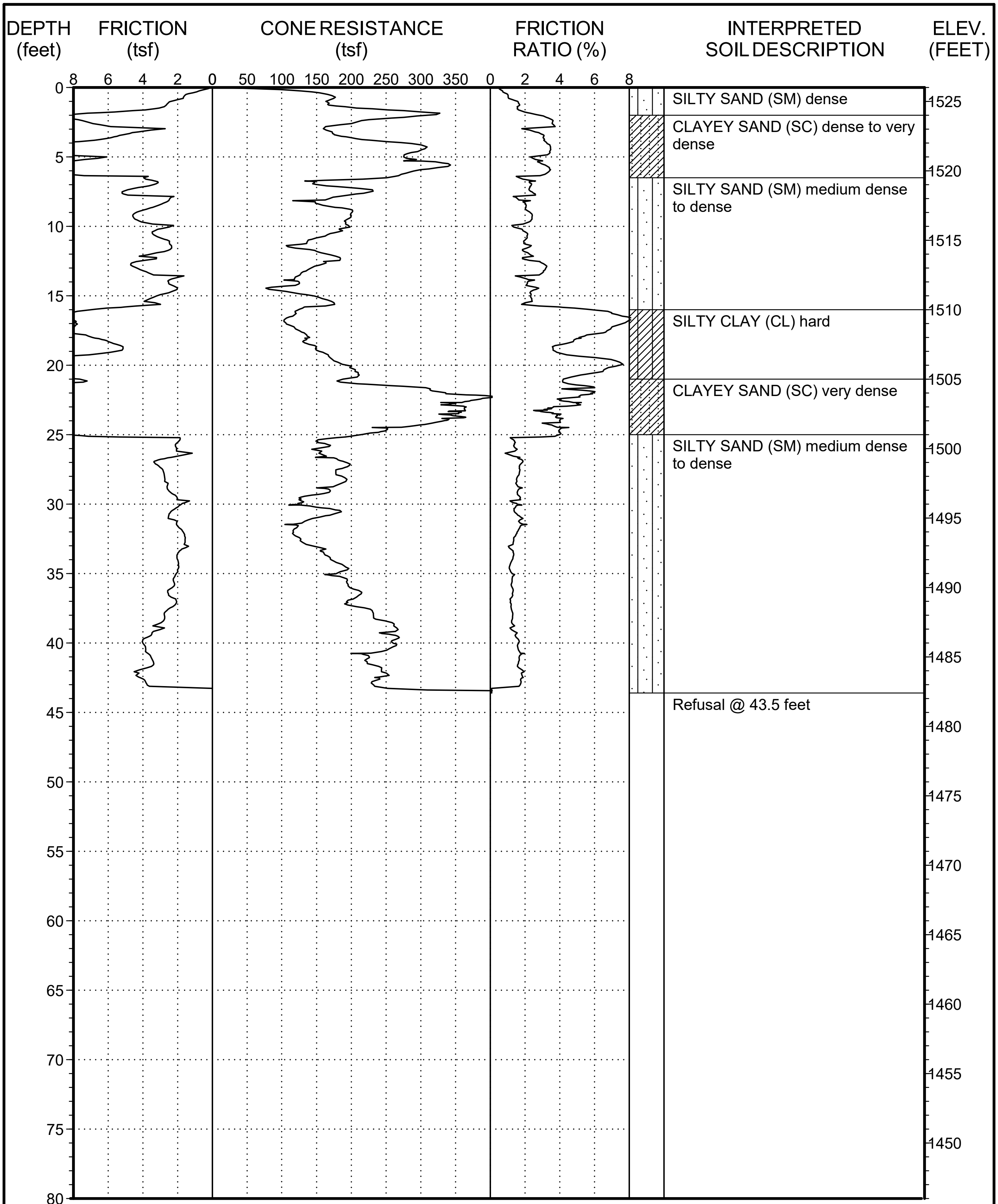
CONE PENET O ETER

FIGURE



Date performed: 7-25-23

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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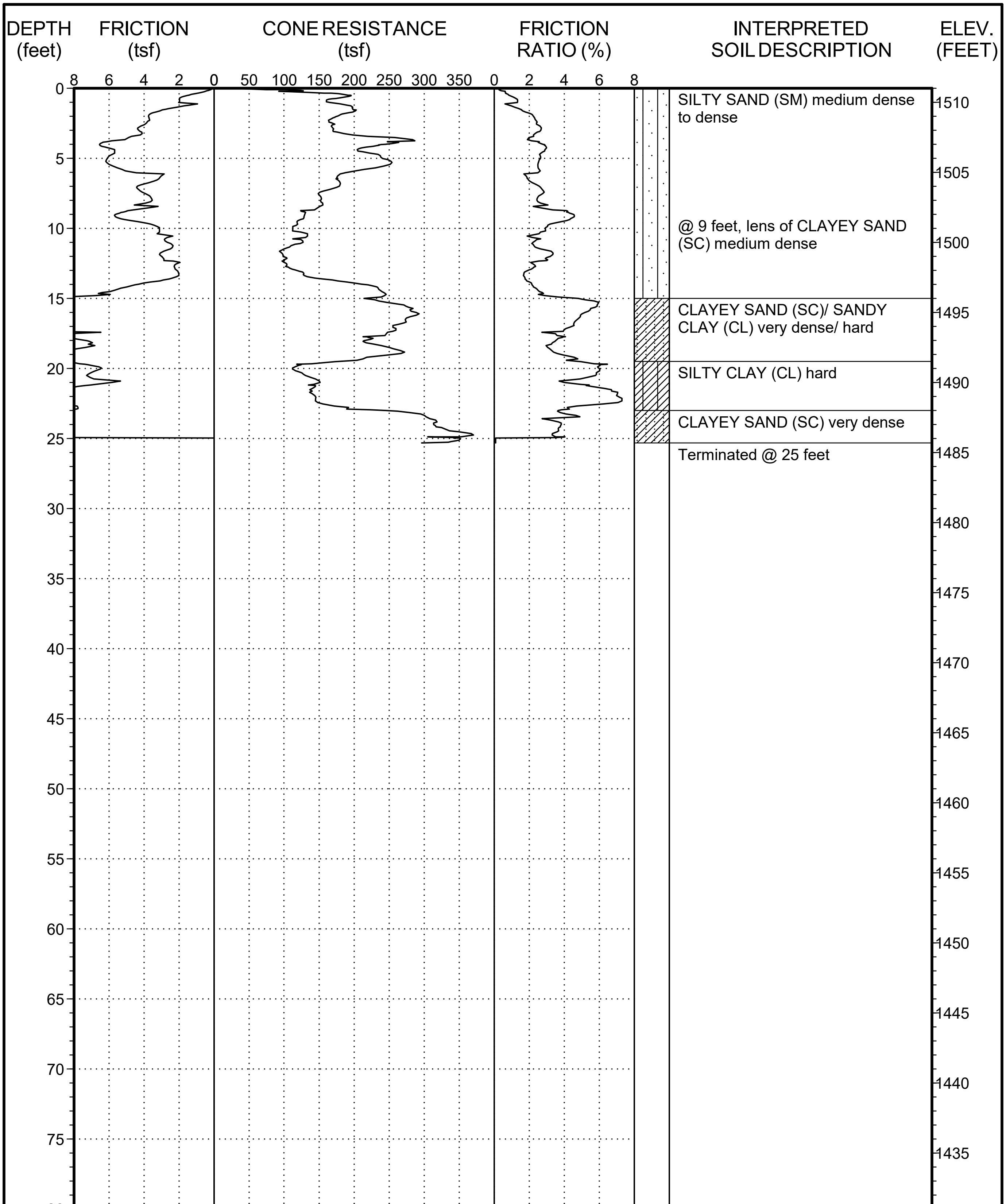
LOG OF CPT NO. C-1

FIGURE A-2



Date performed: 7-25-23

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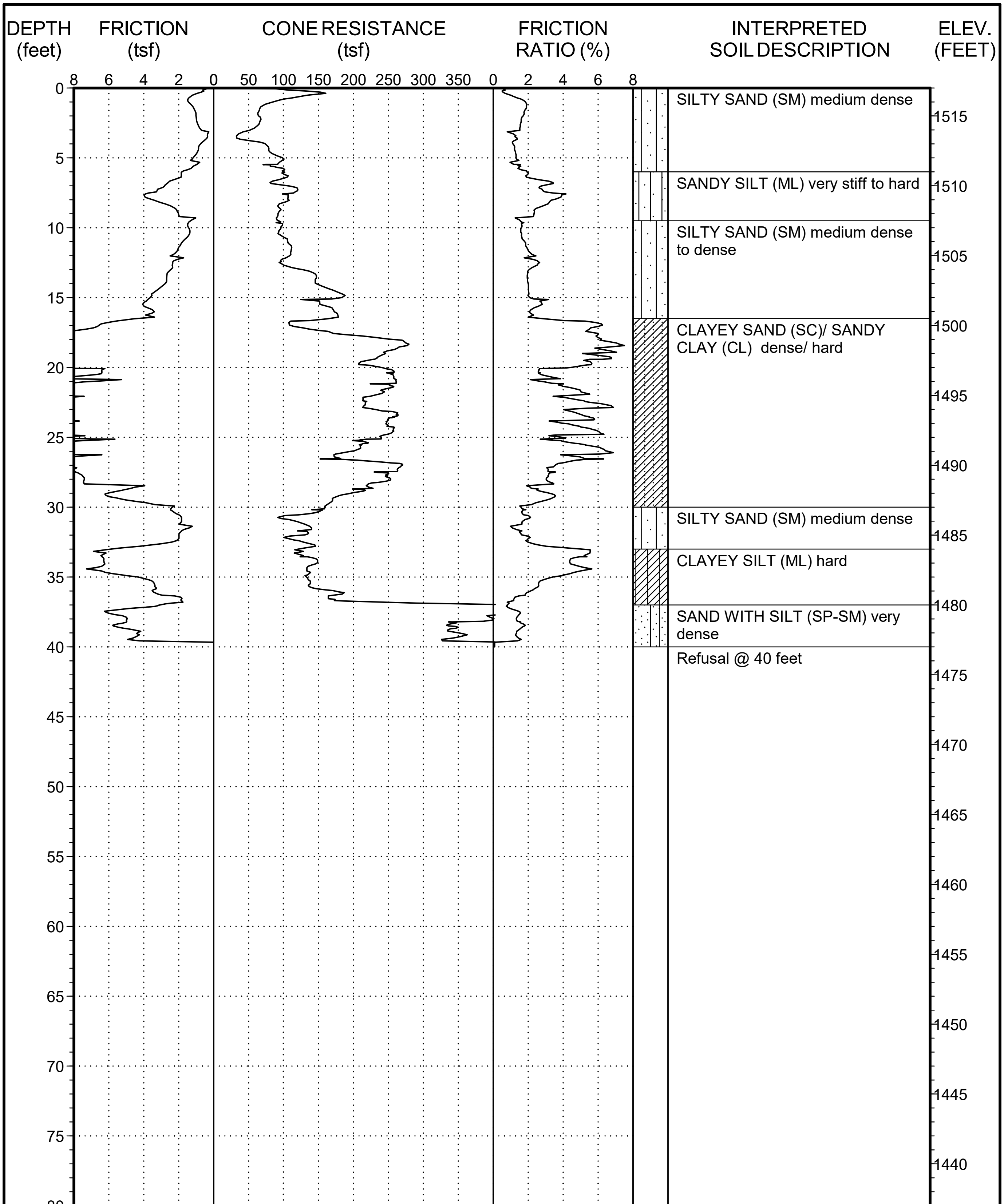
LOG OF CPT NO. C-2

FIGURE A-3



Date performed:7-25-23

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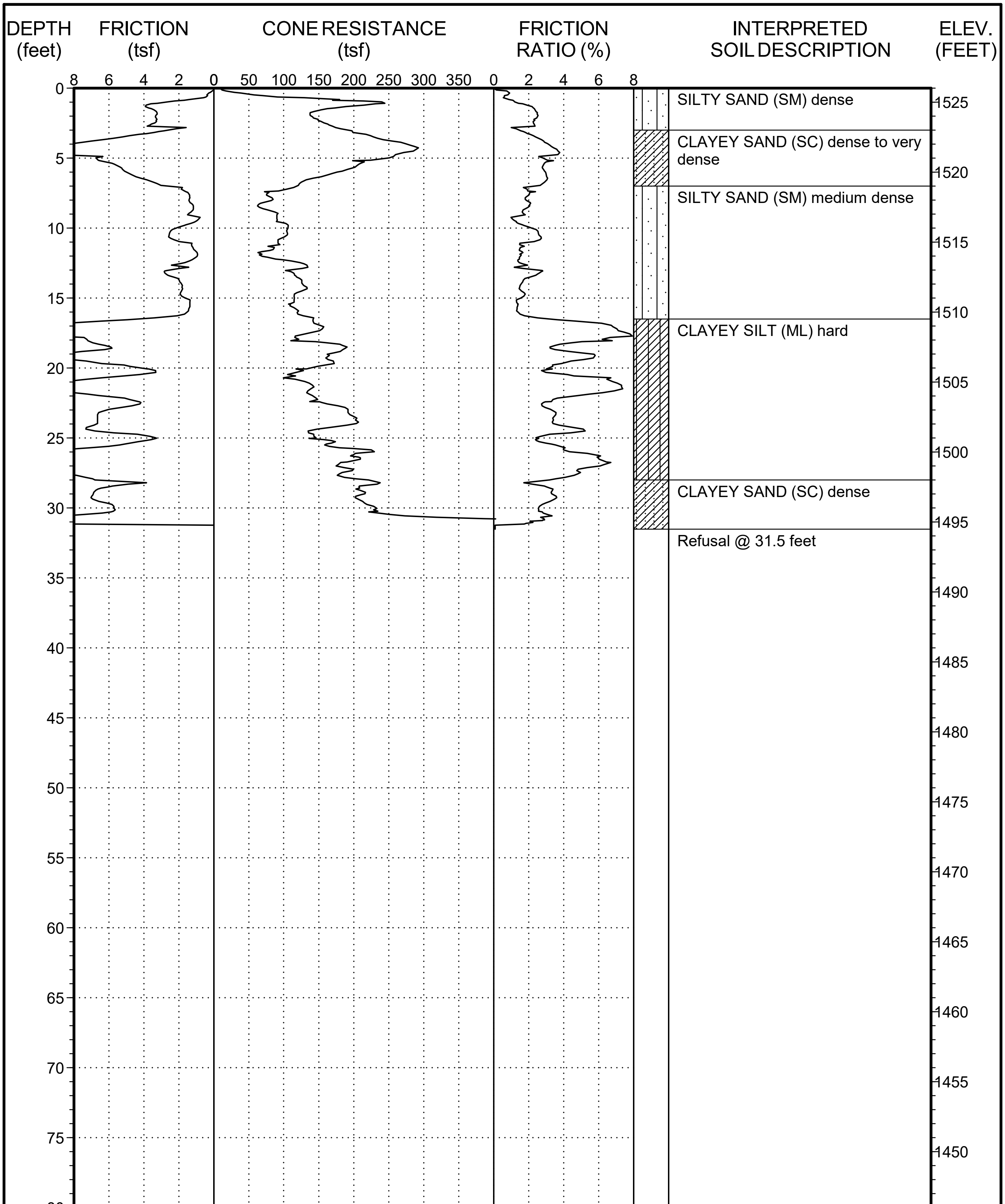
LOG OF CPT NO. C-3

FIGURE A-4



Date performed:7-25-23

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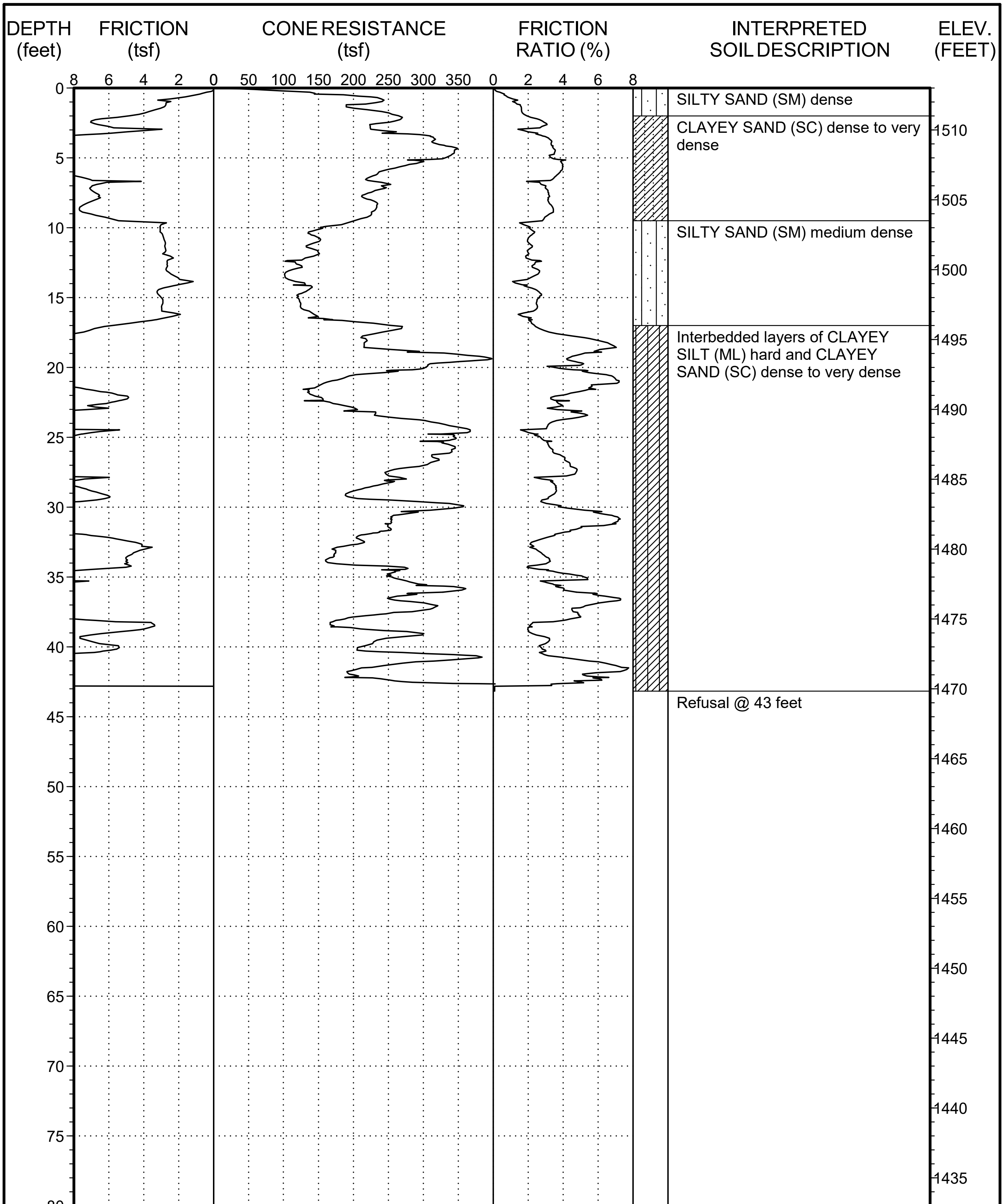
LOG OF CPT NO. C-4

FIGURE A-5



Date performed: 7-25-23

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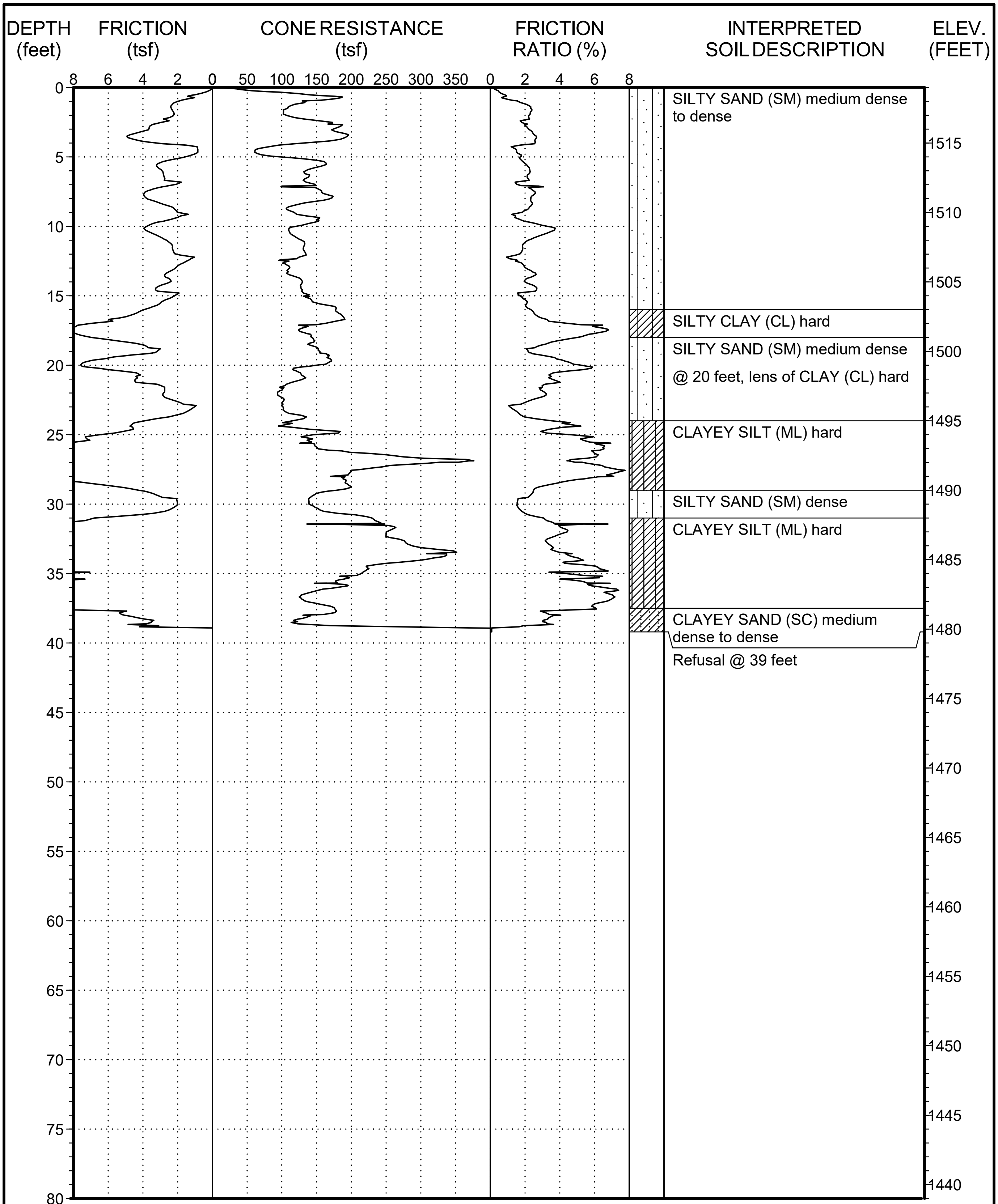
LOG OF CPT NO. C-5

FIGURE A-6



Date performed: 7-25-23

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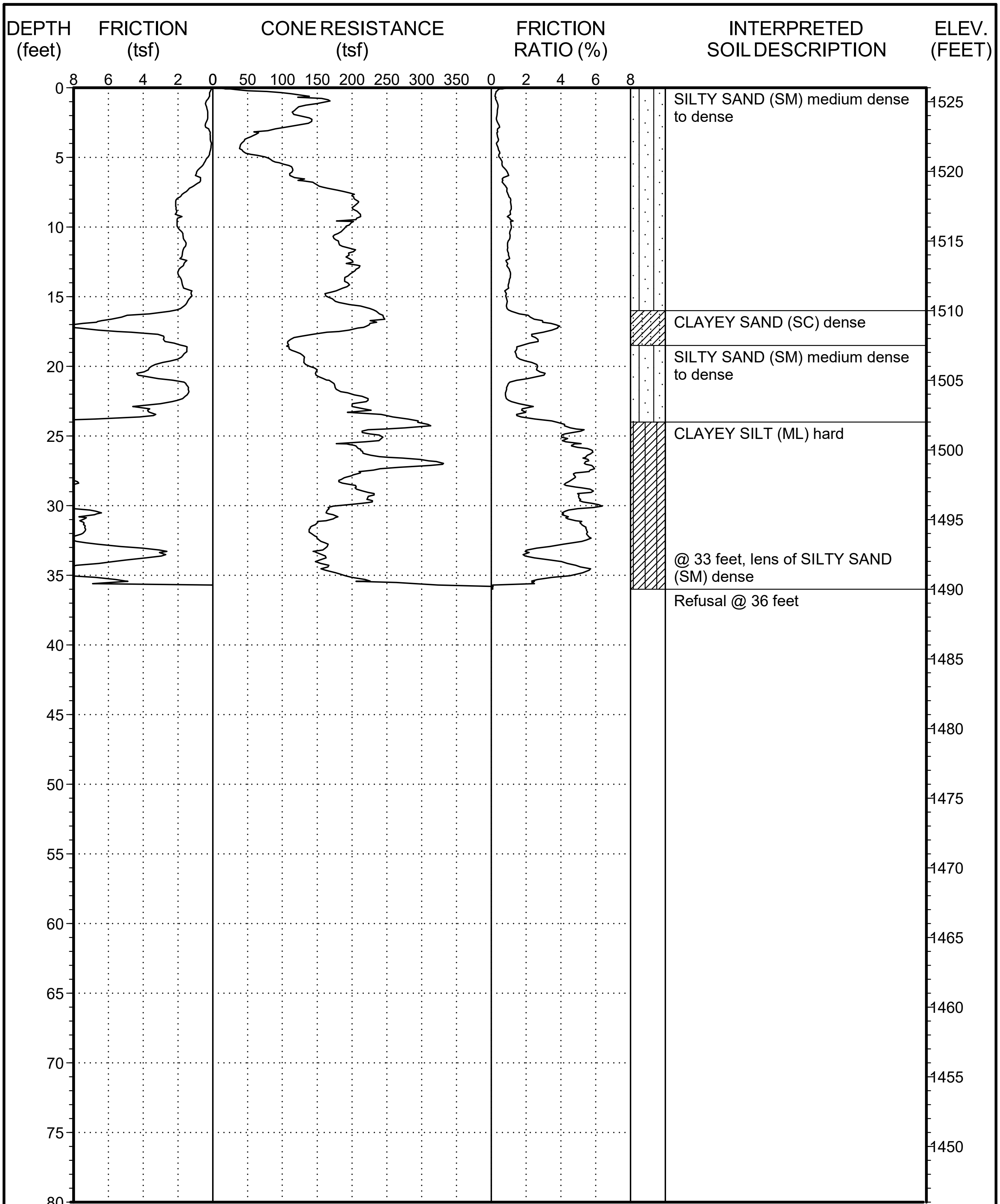
LOG OF CPT NO. C-6

FIGURE A-7



Date performed: 7-25-23

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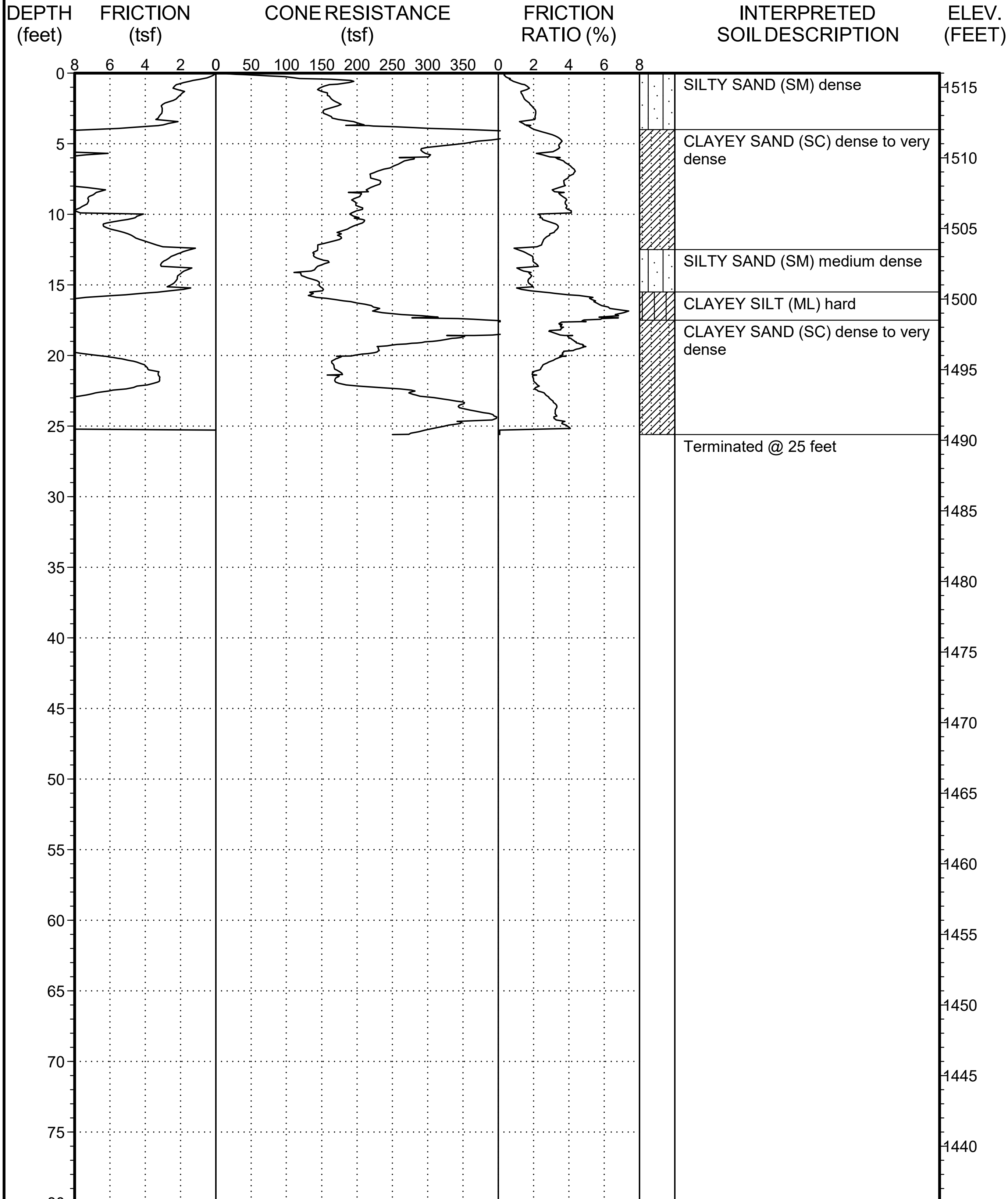
LOG OF CPT NO. C-7

FIGURE A-8



Date performed: 7-25-23

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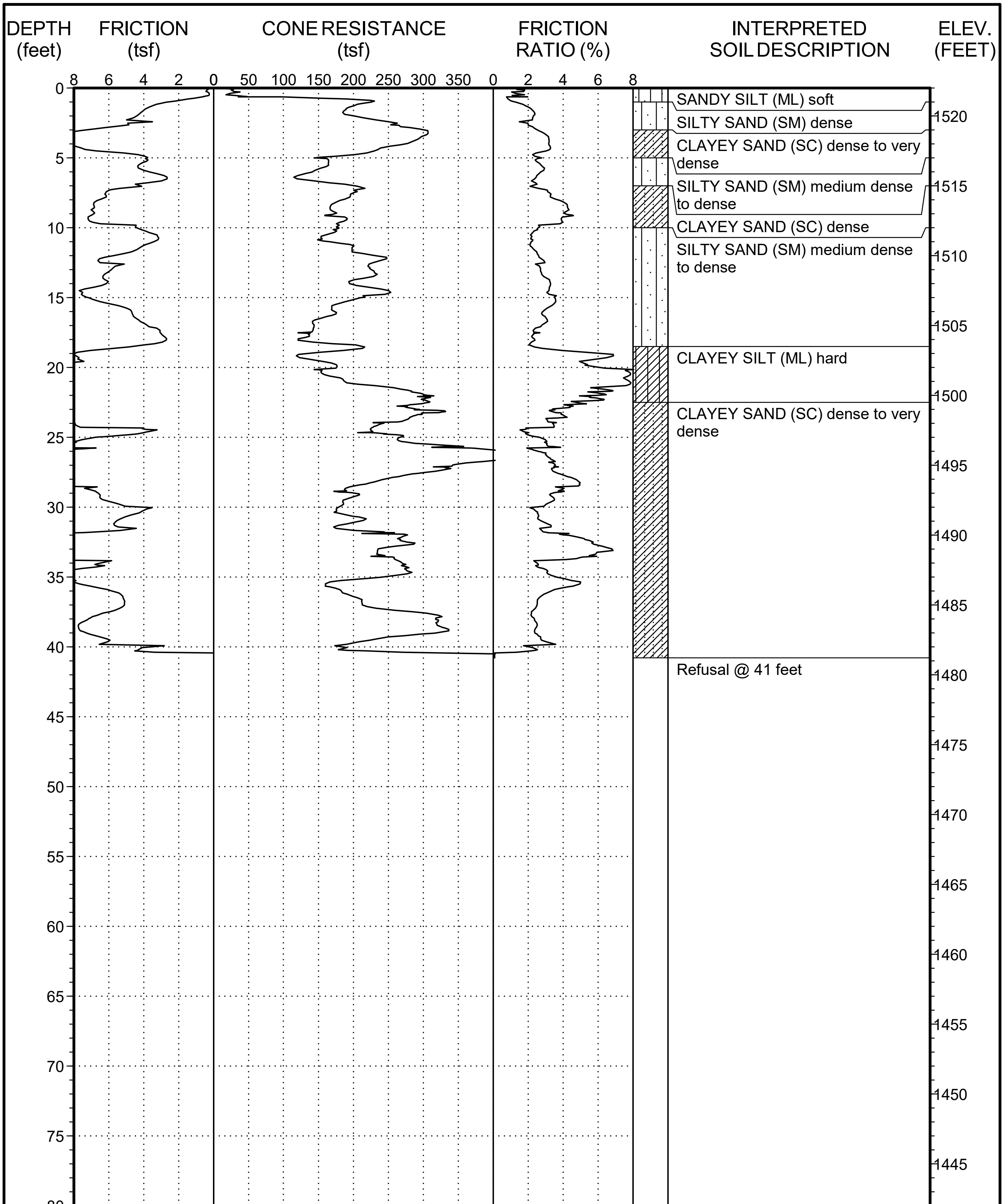
LOG OF CPT NO. C-8

FIGURE A-9



Date performed: 7-25-23

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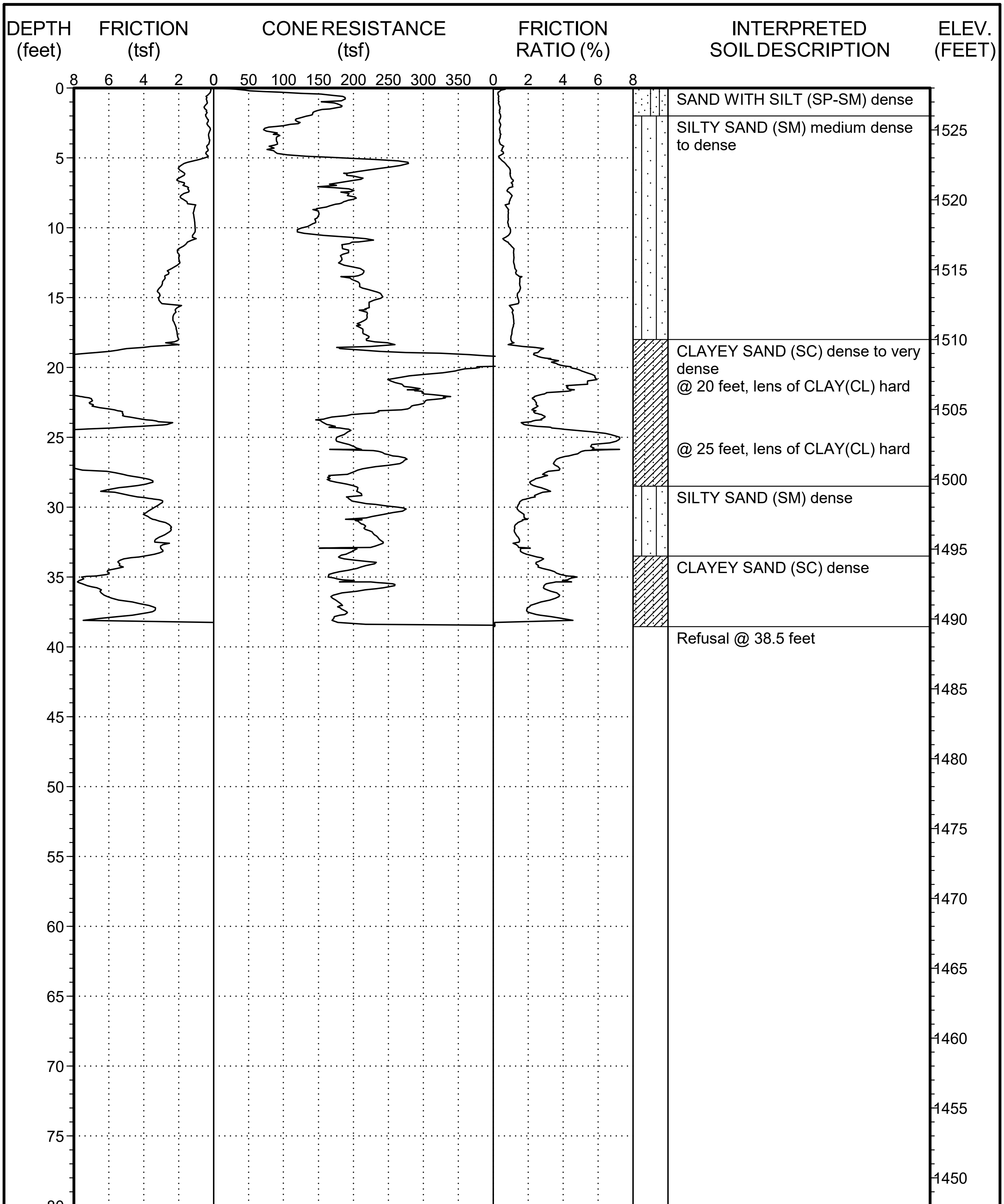
LOG OF CPT NO. C-9

FIGURE A-10



Date performed:7-25-23

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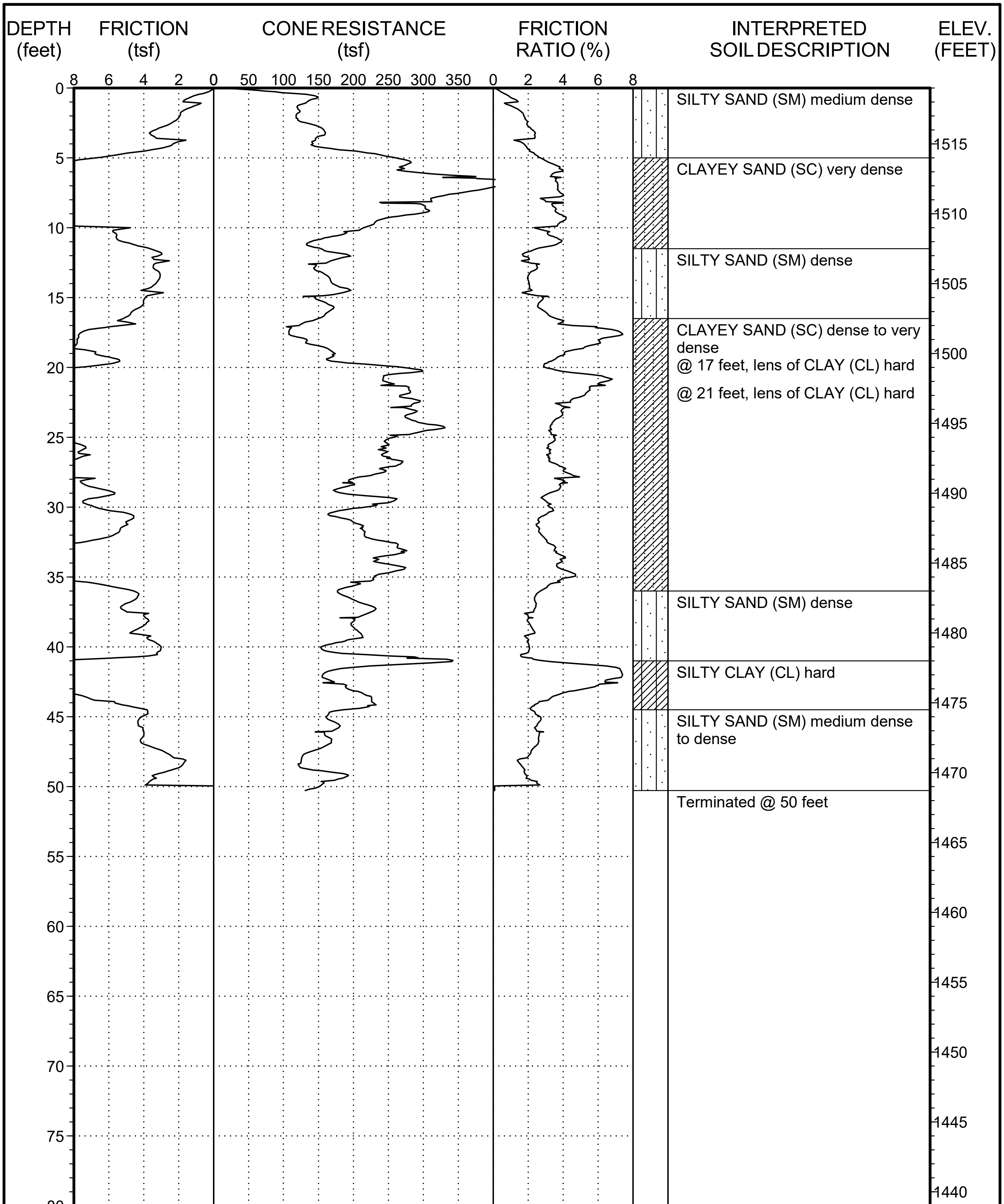
LOG OF CPT NO. C-10

FIGURE A-11



Date performed:7-25-23

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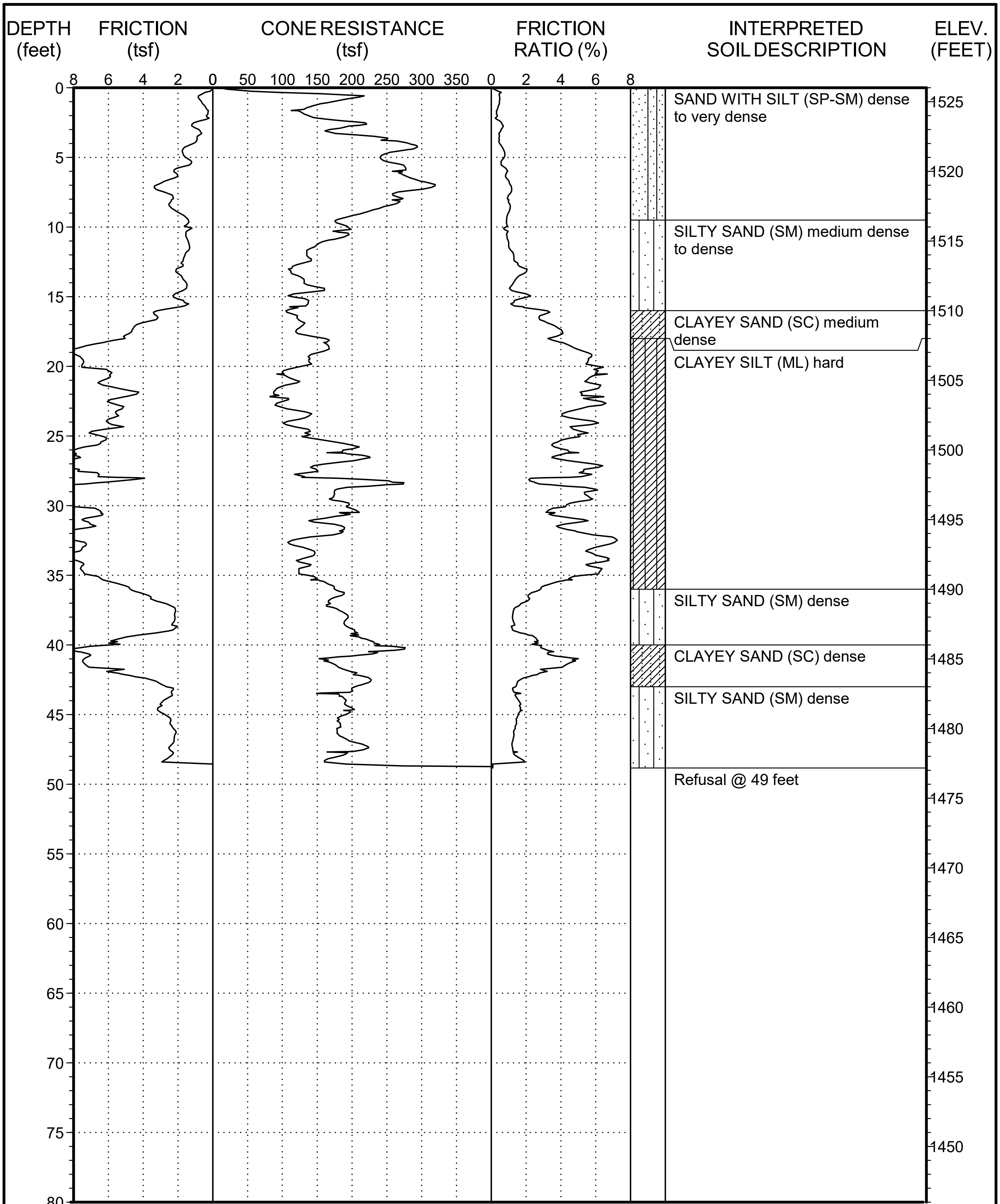
LOG OF CPT NO. C-11

FIGURE A-12



Date performed:7-25-23

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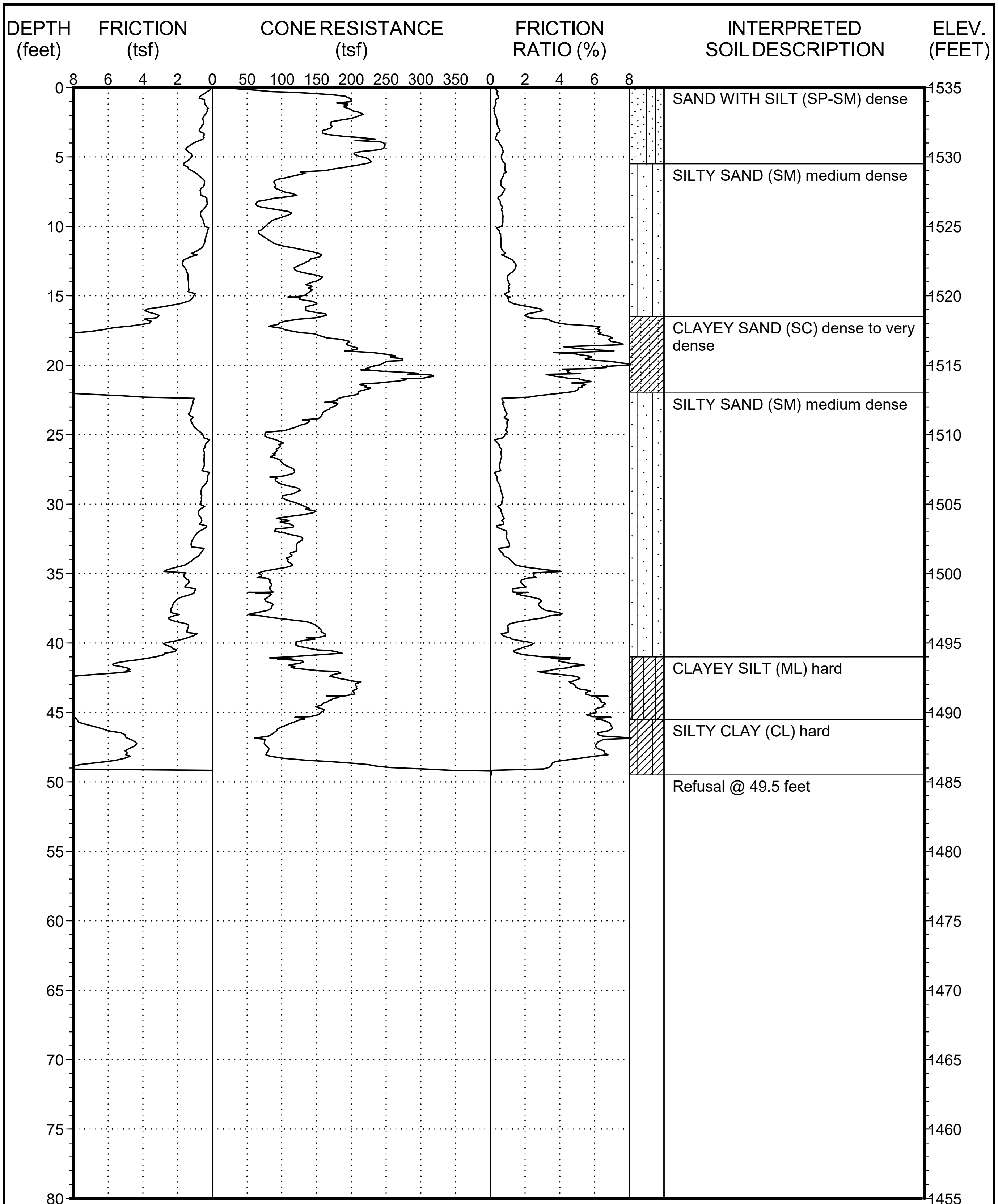
LOG OF CPT NO. C-12

FIGURE A-13



Date performed:7-25-23

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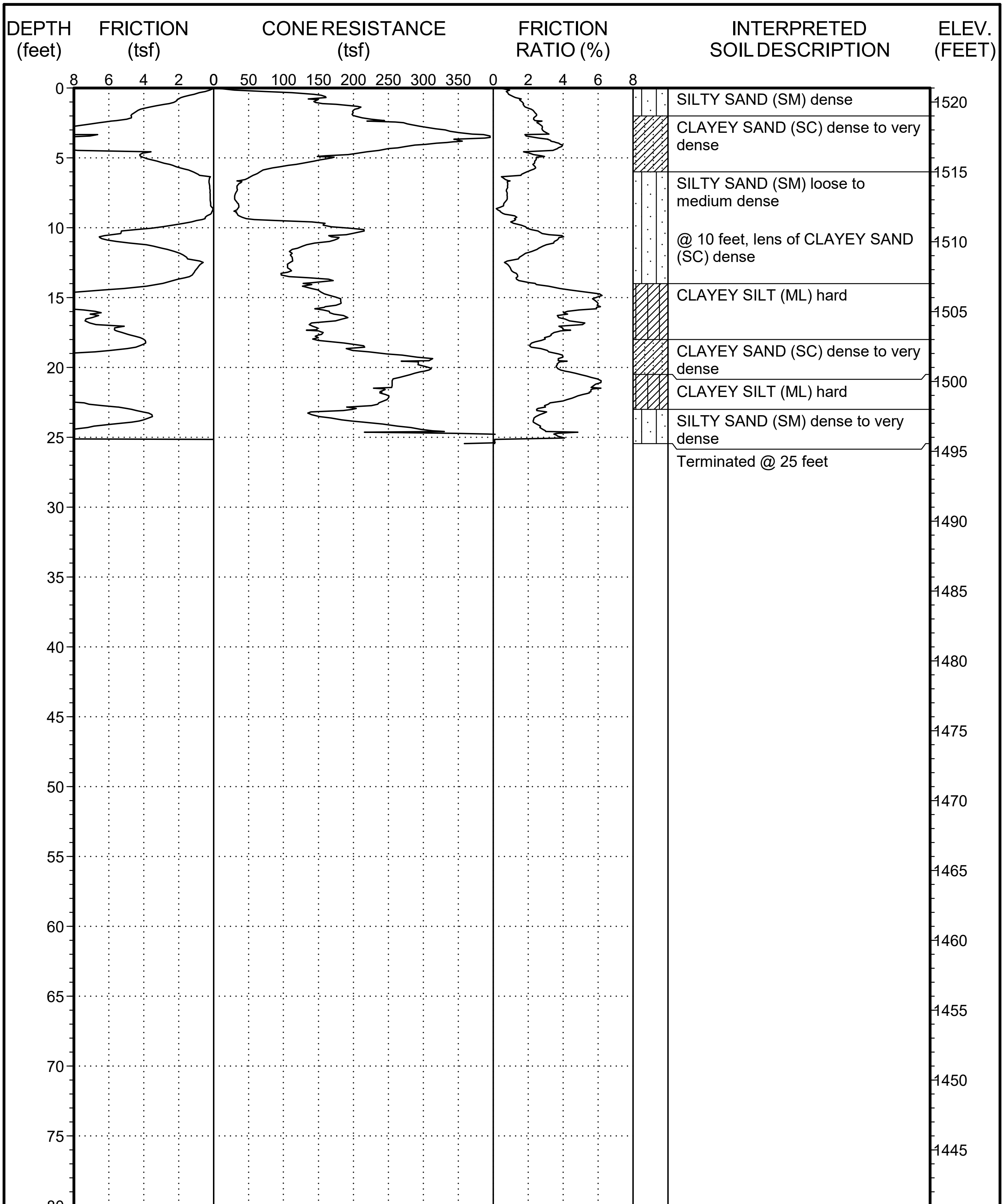
LOG OF CPT NO. C-13

FIGURE A-14



Date performed:7-25-23

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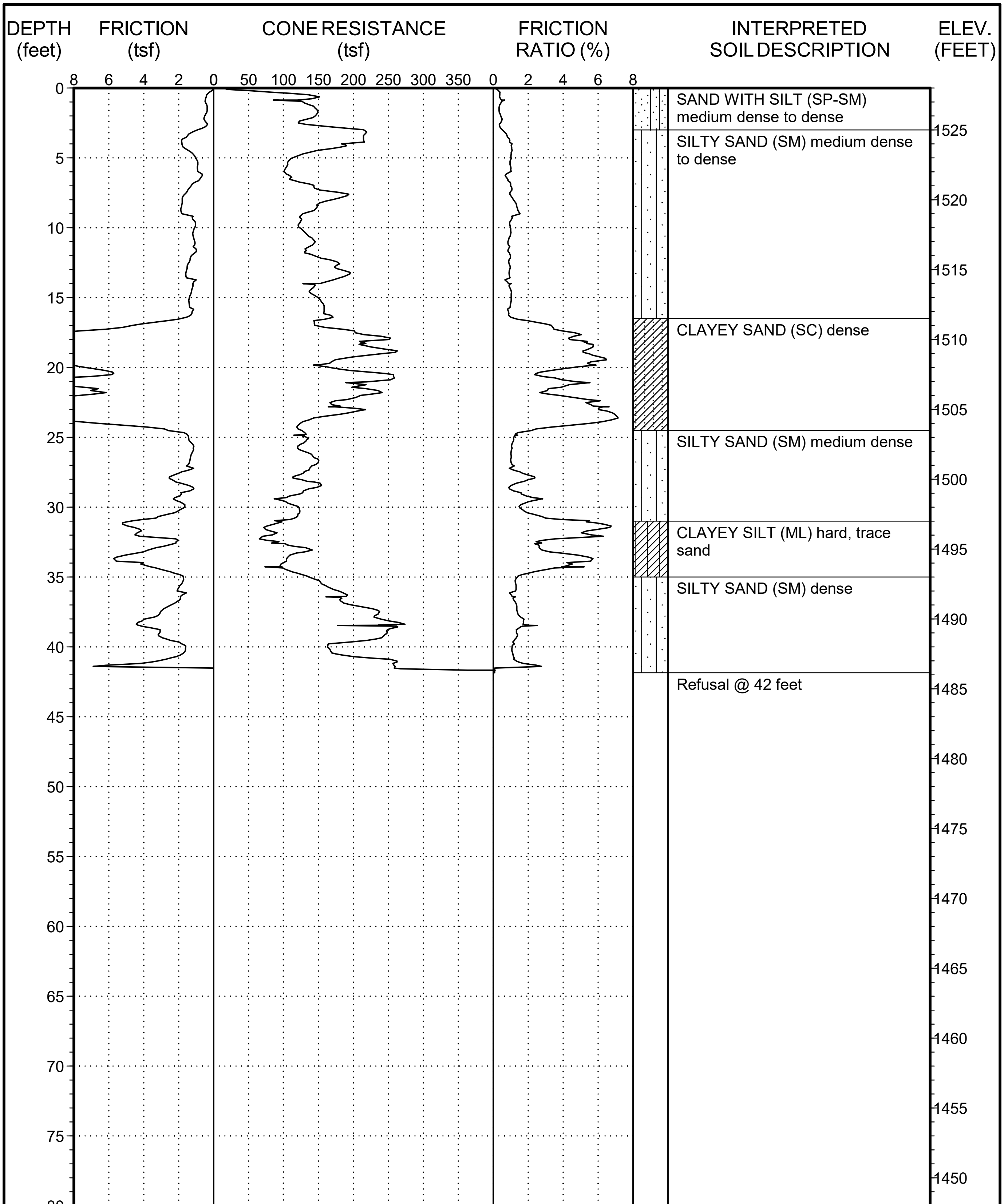
LOG OF CPT NO. C-14

FIGURE A-15



Date performed:7-25-23

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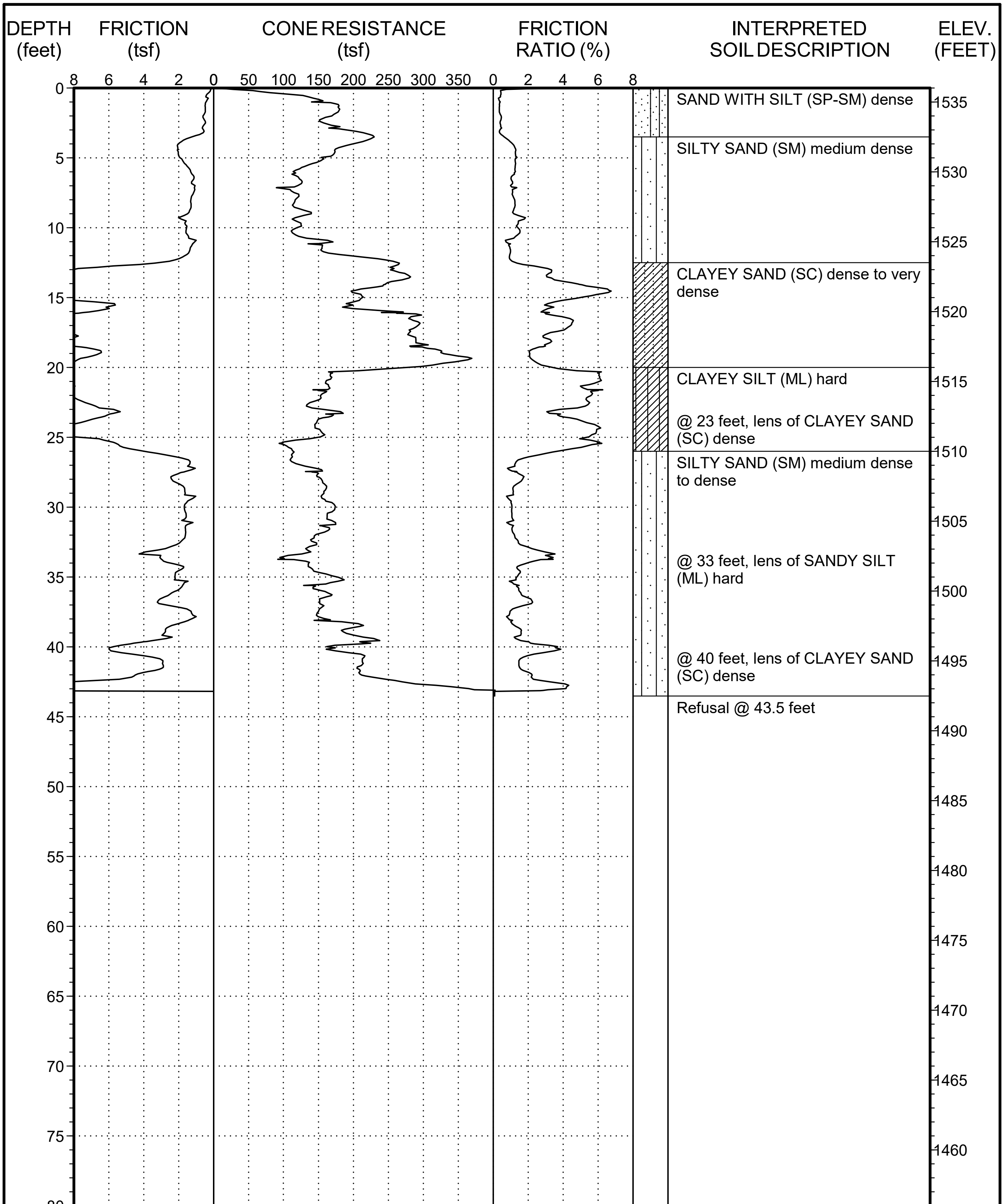
LOG OF CPT NO. C-15

FIGURE A-16



Date performed: 7-25-23

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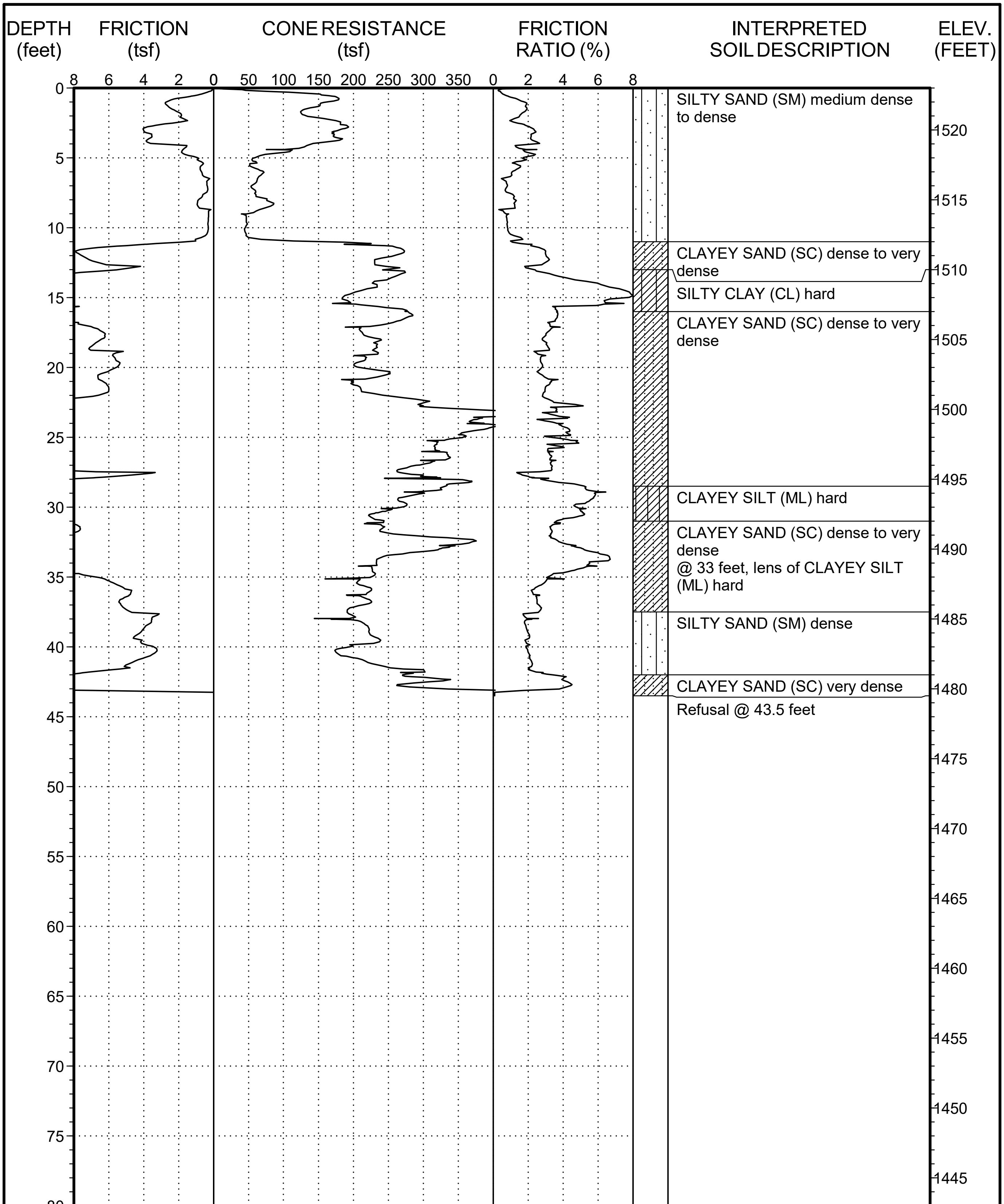
LOG OF CPT NO. C-16

FIGURE A-17



Date performed:7-25-23

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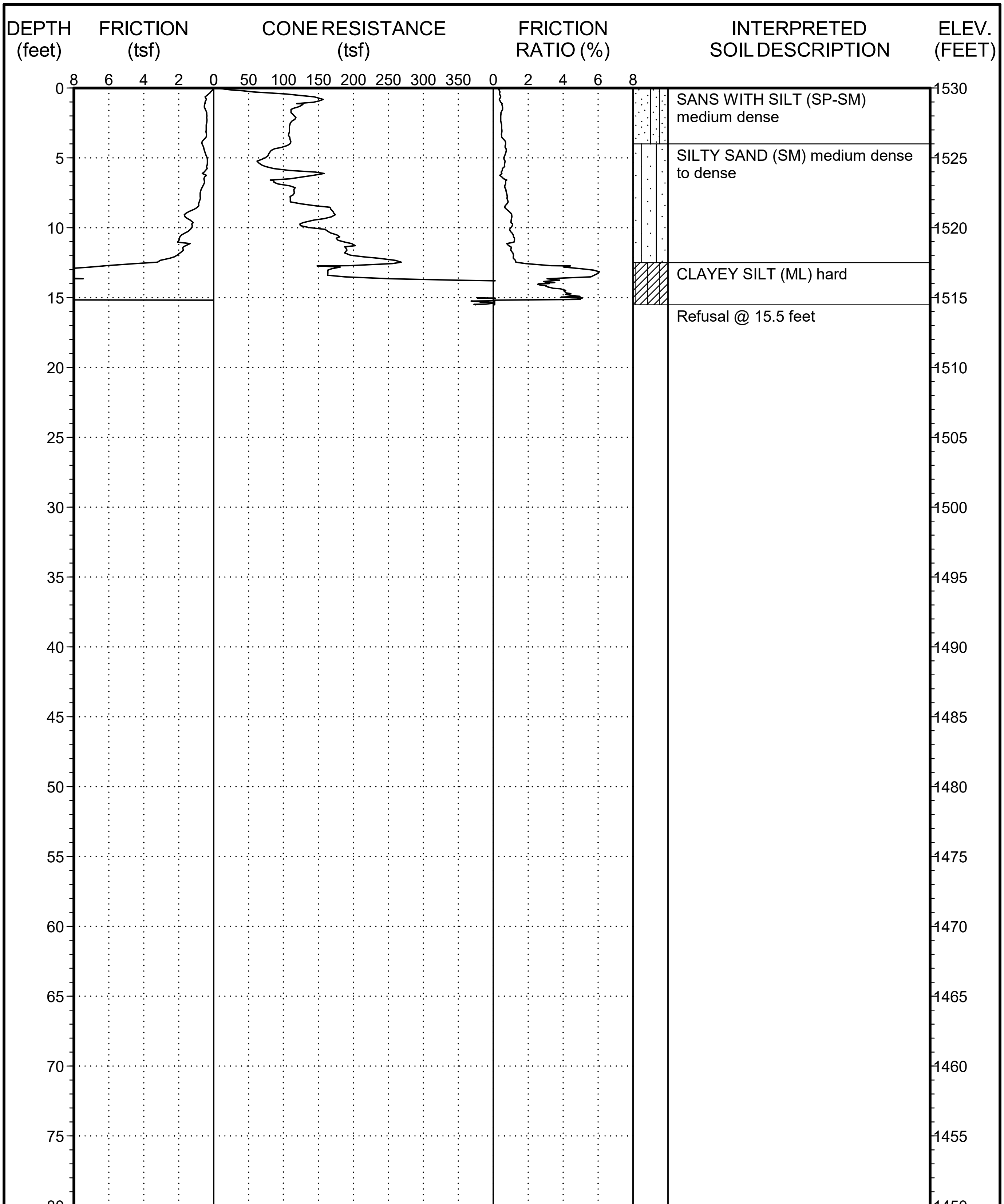
LOG OF CPT NO. C-17

FIGURE A-18



Date performed:7-25-23

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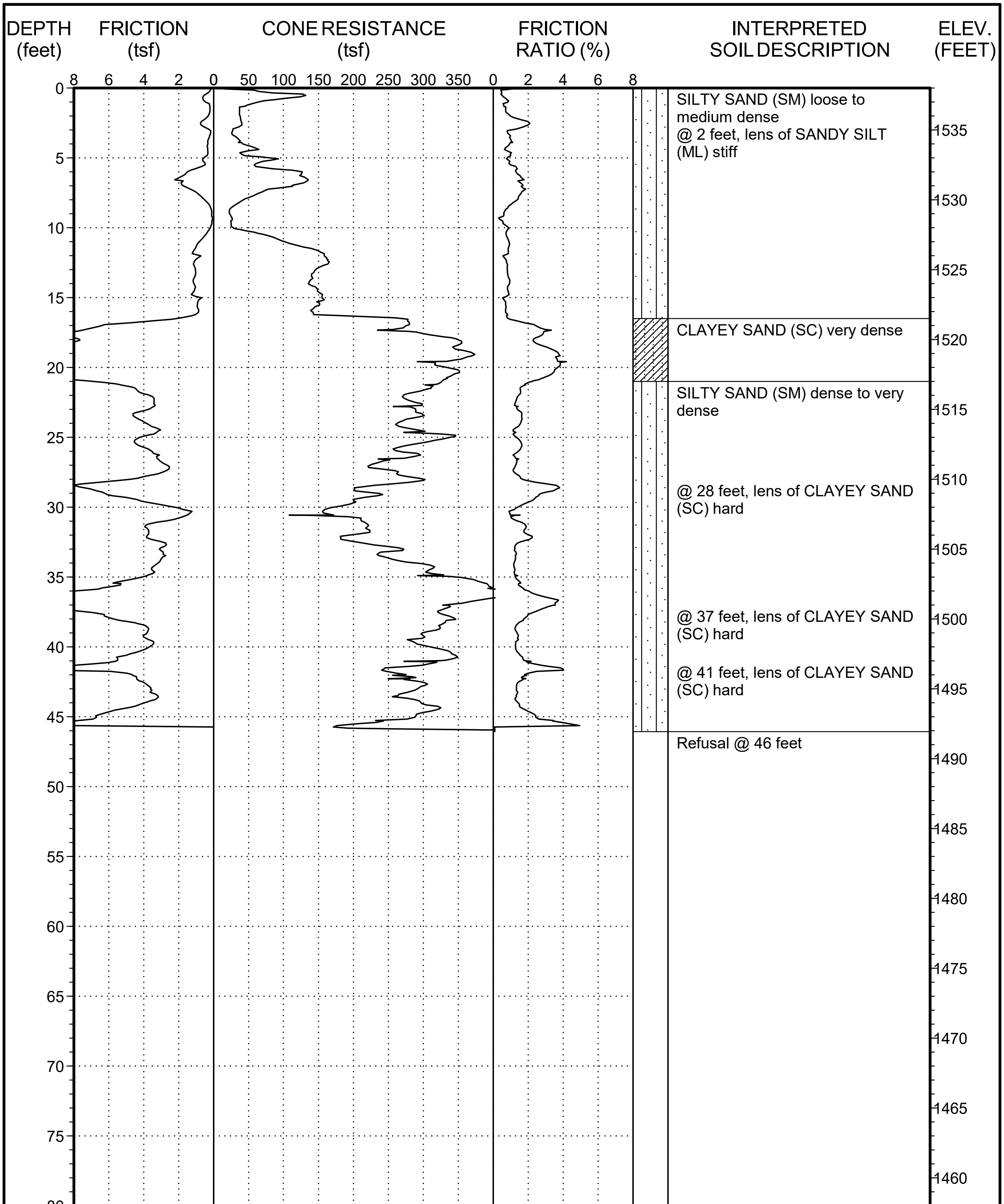
LOG OF CPT NO. C-18

FIGURE A-19



Date performed:7-25-23

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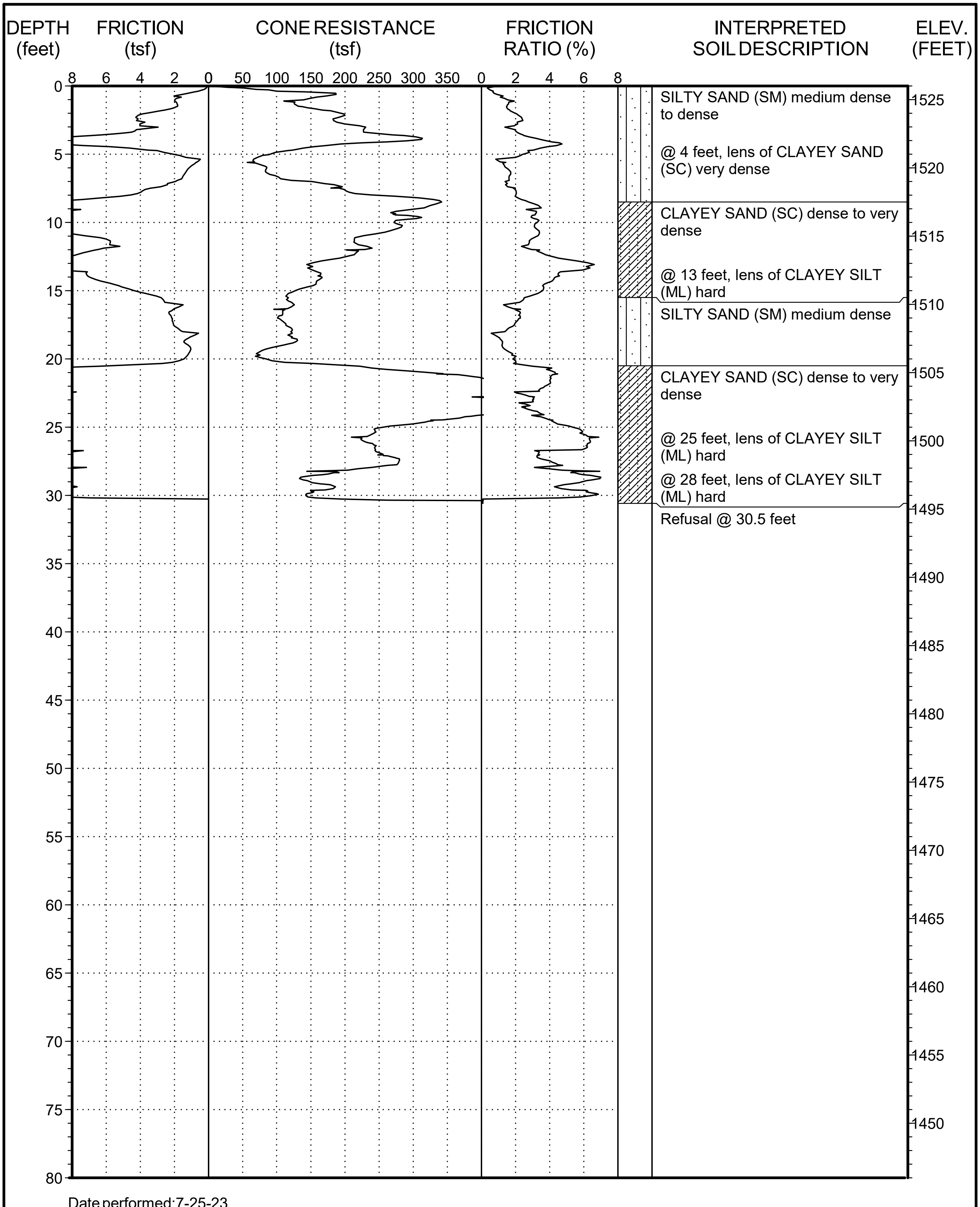
LOG OF CPT NO. C-19

FIGURE A-20



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LOG OF CPT NO. C-20

FIGURE A-21

APPENDIX B

APPENDIX B

EXPLORATORY BORINGS

The subsurface conditions at the site were investigated by drilling and sampling sixteen exploratory borings. The borings were advanced to depths of 6 to 41 feet below the existing ground surface. The locations of the explorations are shown on the Site Plan, Figure 2.

The exploratory borings were drilled using truck-mounted hollow-stem auger drill equipment. Relatively undisturbed samples were obtained using a brass-ring lined sampler (ASTM 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.

The field explorations for the investigation were performed under the continuous technical supervision of GPI's representative, who visually inspected the site, maintained detailed logs of the borings, classified the soils encountered, and obtained relatively undisturbed samples for examination and laboratory testing. The soils encountered in the borings were classified in the field and through further examination in the laboratory in accordance with the Unified Soils Classification System. Detailed logs of the borings are presented in Figures B-1 and B-16 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 Google Earth and should be considered very approximate.

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
				0	Fill: **SILTY SAND (SM)** brown, dry	
2.7	108	18	D		Natural: **SILTY SAND (SM)** brown, dry, medium dense	1520
5.0	110	16	D	5	@ 5 feet, slightly moist	
					Total Depth 6 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.

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

DATE DRILLED:
10-5-23

EQUIPMENT USED:
8 " HOLLOW STEM AUGER

GROUNDWATER LEVEL (ft):
NOT ENCOUNTERED

GPI

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LOG OF BORING NO. 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)
				0	Fill: **SILTY SAND (SM)** brown, slightly moist	
4.8	103	22	D		Natural: **SILTY SAND (SM)** brown, slightly moist, medium dense	
						1515
3.6	106	15	D	5	@ 5 feet, dry, loose to medium dense	
6.3	110	27	D		@ 7 feet, slightly moist, medium dense	
						1510
6.7	111	30	D	10		
						1505
10.2	124	70	D	15	**CLAYEY SAND (SC)** brown, moist, dense	
						1500
7.4	117	50/5"	D	20	**SILTY SAND (SM)** brown, slightly moist, very dense	
					Total Depth 21 feet	

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

DATE DRILLED:
10-5-23
EQUIPMENT USED:
8 " HOLLOW STEM AUGER
GROUNDWATER LEVEL (ft):
NOT ENCOUNTERED

GPI

PROJECT NO.: 3194.2I
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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	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
			B	0	Fill: **CLAYEY SAND (SC)** brown, moist	1520
8.6	114	12	D		Natural: **CLAYEY SAND (SC)** brown, moist, loose	
6.4	117	25	D	5	**SILTY SAND (SM)** brown, slightly moist, medium dense	1515
6.3	118	40	D		@ 7 feet, trace clayey sand	
6.3	103	28	D	10		1510
7.1	110	29	D	15		1505
10.0	118	53	D	20	@ 20 feet, moist, dense **CLAYEY SAND (SC)** brown, moist, dense	1500
10.0	123	50/5"	D	25	**SILTY SAND (SM)** brown , moist, very dense, trace clayey sand	1495
8.2	118	48	D	30		1490
					Total Depth 31 feet	

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

DATE DRILLED:
10-5-23

EQUIPMENT USED:
8 " HOLLOW STEM AUGER

GROUNDWATER LEVEL (ft):
NOT ENCOUNTERED

GPI

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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)
						This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
				B	0	Fill: **SILTY SAND (SM)** brown, dry	1525
	3.8	101	34	D		Natural: **SILTY SAND (SM)** brown, dry, medium dense	
	3.5	98	22	D	5		1520
	5.0	117	71	D		@ 7 feet, slightly moist, dense	
	4.7	114	41	D	10	@ 10 feet, medium dense	1515
	6.6	121	60	D	15	@ 15 feet, dense	1510
	6.9	109	50/3"	D	20	@ 20 feet, very dense	1505
						Total Depth 21 feet	

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

DATE DRILLED:
10-5-23

EQUIPMENT USED:
8 " HOLLOW STEM AUGER

GROUNDWATER LEVEL (ft):
NOT ENCOUNTERED

GPI

PROJECT NO.: 3194.2I

PMB RUHS PERRIS

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	Fill: **SILTY SAND (SM)** brown, dry	1530
	2.6	114	50/5"	D		Natural: **SILTY SAND (SM)** brown, dry, very dense	
	2.8	116	58	D	5	@ 5 feet, dense	1525
	5.7	165	59	D		@ 7 feet, slightly moist	
	5.4	107	25	D	10	@ 10 feet, medium dense	1520
	6.5	116	55	D	15	@ 15 feet, dense	1515
	9.1	119	50/3"	D	20	@ 20 feet, moist, very dense	1510
	9.1	121	50/4"	D	25		1505
	3.3	111	50/4"	D	30	@ 30 feet, dry	1500
	8.5	119	58	D	35	@ 35 feet, moist, dense	1495
	5.9	108	40	D	40	@ 40 feet, slightly moist, medium dense	1490
						Total Depth 41 feet	

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

DATE DRILLED:
10-5-23

EQUIPMENT USED:
8 " HOLLOW STEM AUGER

GROUNDWATER LEVEL (ft):
NOT ENCOUNTERED

GPI

PROJECT NO.: 3194.2I

PMB RUHS PERRIS

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, dry	
	3.9	107	36	D		Natural: **SILTY SAND (SM)** brown, dry, medium dense	
							1525
	6.7	118	32	D	5	@ 5 feet, slightly moist	
	6.7	122	32	D			
							1520
	5.4	110	21	D	10		
							1515
	5.7	115	34	D	15		
							1510
	9.9	121	55	D	20	@ 20 feet, moist, dense	
						Total Depth 21 feet	

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

DATE DRILLED:
10-5-23

EQUIPMENT USED:
8 " HOLLOW STEM AUGER

GROUNDWATER LEVEL (ft):
NOT ENCOUNTERED

GPI

PROJECT NO.: 3194.2I

PMB RUHS PERRIS

LOG OF BORING NO. B-6

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
				B	0	Fill: **SILTY SAND (SM)** brown, dry	
	4.2	106	55	D		Natural: **SILTY SAND (SM)** brown, dry, dense	1535
	3.9		28	D	5	@ 5 feet, medium dense	
	4.2	107	24	D			1530
	8.7	92	31	D	10	@ 10 feet, moist	
							1525
	9.5	121	50/4"	D	15	@ 15 feet, very dense	
							1520
	11.3	118	50/5"	D	20		
						Total Depth 21 feet	

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

DATE DRILLED:
10-5-23
EQUIPMENT USED:
8 " HOLLOW STEM AUGER
GROUNDWATER LEVEL (ft):
NOT ENCOUNTERED

GPI

PROJECT NO.: 3194.2I
PMB RUHS PERRIS

LOG OF BORING NO. B-7

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					0	Fill: **SILTY SAND (SM)** brown, dry	
	4.3	117	57	D		Natural: **SILTY SAND (SM)** brown, dry, dense	
							1530
	3.3	104	10	D	5	@ 5 feet, loose	
	4.9	106	39	D		@ 7 feet, slightly moist, medium dense	
							1525
	2.5	112	41	D	10	@ 10 feet, dry	
							1520
	4.1	110	46	D	15	@ 15 feet, dense	
							1515
	5.8	121	58	D	20	@ 20 feet, slightly moist	
							1510
	5.1	118	50/5"	D	25	@ 25 feet, very dense	
							1505
	4.3	122	50/3"	D	30	@ 30 feet, dry	
							1500
	6.4	114	50/4"	D	35	@ 35 feet, slightly moist	
							1495
	2.2		50/3"	D	40	@ 40 feet, sample disturbed	
						Total Depth 41 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:
10-5-23

EQUIPMENT USED:
8 " HOLLOW STEM AUGER

GROUNDWATER LEVEL (ft):
NOT ENCOUNTERED

GPI

PROJECT NO.: 3194.2I
PMB RUHS PERRIS

LOG OF BORING NO. B-8

FIGURE B-8

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
			B	0	Fill: **SILTY SAND (SM)** brown, dry	
4.0	113	67	D		Natural: **SILTY SAND (SM)** brown, dry, dense	1510
3.3	113	57	D	5	@ 5 feet, trace clayey sand	
7.2	122	43	D		**CLAYEY SAND (SC)** brown, slightly moist, medium dense	1505
5.4	105	24	D	10	**SILTY SAND (SM)** brown, slightly moist, medium dense, trace clayey sand	1500
7.4	117	50/3"	D	15	@ 15 feet, very dense	1495
7.2	99	29	D	20	@ 20 feet, medium dense	1490
7.7	120	50/4"	D	25	@ 25 feet, moist, very dense	1485
10.1	118	50/3"	D	30		
					Total Depth 31 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:
10-5-23

EQUIPMENT USED:
8 " HOLLOW STEM AUGER

GROUNDWATER LEVEL (ft):
NOT ENCOUNTERED

GPI

PROJECT NO.: 3194.2I
PMB RUHS PERRIS

LOG OF BORING NO. B-9

FIGURE B-9

	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, dry	1515
	4.1	116	50/5"	D		Natural: **SILTY SAND (SM)** brown, dry, very dense	
	3.8	115	58	D	5	@ 5 feet, dense	1510
						Total Depth 6 feet	

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

DATE DRILLED:
10-5-23

EQUIPMENT USED:
8 " HOLLOW STEM AUGER

GROUNDWATER LEVEL (ft):
NOT ENCOUNTERED

GPI

PROJECT NO.: 3194.2I

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LOG OF BORING NO. B-10

FIGURE B-10

	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, dry	1520
	3.3	110	40	D		Natural: **SILTY SAND (SM)** brown, dry, medium dense	
	4.1	118	32	D	5		1515
	10.0	119	45	D		@ 7 feet, moist, medium dense to dense, trace clayey sand	
	5.5	112	31	D	10	@ 10 feet, slightly moist	1510
	3.0	110	38	D	15	@ 15 feet, dry	1505
	11.2	119	50/4"	D	20	@ 20 feet, moist, very dense	1500
	9.8	121	50/2"	D	25		1495
	7.5	116	73	D	30	@ 30 feet, dense	1490
	7.0	117	45	D	35	@ 35 feet, slightly moist, medium dense to dense	1485
	5.2		50/3"	D	40	**SAND WITH SILT (SP-SM)** white, slightly moist, very dense	1480
						Total Depth 41 feet	

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

DATE DRILLED:
10-5-23

EQUIPMENT USED:
8 " HOLLOW STEM AUGER

GROUNDWATER LEVEL (ft):
NOT ENCOUNTERED

GPI

PROJECT NO.: 3194.2I

PMB RUHS PERRIS

LOG OF BORING NO. B-11

FIGURE B-11

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					0	Fill: **SILTY SAND (SM)** brown, dry	1515
	4.2	108	33	D		Natural: **SILTY SAND (SM)** brown, dry, medium dense	
	3.7	112	53	D	5	@ 5 feet, dense	1510
	4.1	114	50/4"	D		@ 7 feet, very dense	
	4.9	104	40	D	10	@ 10 feet, slightly moist, medium dense	1505
	10.1	120	35	D	15		1500
	20.2	108	55	D	20	@ 20 feet, wet, dense	1495
						Total Depth 21 feet	

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

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

DATE DRILLED: 10-5-23

EQUIPMENT USED: 8 " HOLLOW STEM AUGER

GROUNDWATER LEVEL (ft): NOT ENCOUNTERED

GPI

PROJECT NO.: 3194.2I

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LOG OF BORING NO. B-12

FIGURE B-12

	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, dry	
	3.6	102	34	D		Natural: **SILTY SAND (SM)** brown, dry, medium dense	1520
	4.0	117	50/5"	D	5	@ 5 feet, very dense	
						Total Depth 6 feet	

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

DATE DRILLED:
10-5-23

EQUIPMENT USED:
8 " HOLLOW STEM AUGER

GROUNDWATER LEVEL (ft):
NOT ENCOUNTERED

GPI

PROJECT NO.: 3194.2I

PMB RUHS PERRIS

LOG OF BORING NO. B-13

FIGURE B-13

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
				0	**SILTY SAND (SM)** brown, dry	
4.0	108	39	D		Natural: **SILTY SAND (SM)** brown, dry, medium dense	
						1520
4.1	113	20	D	5		
3.3	103	15	D			
						1515
3.8	109	33	D	10		
						1510
9.0	1521	44	D	15	@ 15 feet, moist, trace clayey sand	
						1505
6.4	122	50/3"	D	20	@ 20 feet, slightly moist, very dense	
					Total Depth 21 feet	

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

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

DATE DRILLED: 10-5-23
EQUIPMENT USED: 8 " HOLLOW STEM AUGER
GROUNDWATER LEVEL (ft): NOT ENCOUNTERED

GPI

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LOG OF BORING NO. B-14

FIGURE B-14

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
						This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
				B	0	Fill: **SILTY SAND (SM)** brown, dry	1525
	3.7	112	50	D		Natural: **SILTY SAND (SM)** brown, dry, dense	
	2.8	106	32	D	5	@ 5 feet, medium dense	1520
	5.5	111	50/4"	D		@ 7 feet, slightly moist, very dense	
	5.5	111	64	D	10	@ 10 feet, dense	1515
	6.2	112	28	D	15	@ 15 feet, medium dense	1510
	5.4	108	71	D	20	@ 20 feet, dense	1505
	5.7	116	71	D	25		1500
	3.2	108	50/4"	D	30	**SAND (SP)** brown, dry, very dense	1495
	3.1	112	50/3"	D	35	**SAND WITH SILT (SP-SM)** brown, dry, very dense	1490
			50/3"	D	40		1485
						Total Depth 41 feet	

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

DATE DRILLED:
10-5-23

EQUIPMENT USED:
8 " HOLLOW STEM AUGER

GROUNDWATER LEVEL (ft):
NOT ENCOUNTERED

GPI

PROJECT NO.: 3194.2I

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LOG OF BORING NO. B-15

FIGURE B-15

	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, dry	
	3.9	114	48	D		Natural: **SILTY SAND (SM)** brown, dry, dense	
							1525
	3.2	109	26	D	5	@ 5 feet, medium dense	
						Total Depth 6 feet	

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

DATE DRILLED:
10-5-23

EQUIPMENT USED:
8 " HOLLOW STEM AUGER

GROUNDWATER LEVEL (ft):
NOT ENCOUNTERED

GPI

PROJECT NO.: 3194.2I

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LOG OF BORING NO. B-16

FIGURE B-16

APPENDIX C

APPENDIX C

LABORATORY TESTS

INTRODUCTION

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

MOISTURE CONTENT AND DRY DENSITY

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

GRAIN SIZE DISTRIBUTION

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

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	PERCENT PASSING No. 200 SIEVE
B-1	2	Silty Sand (SM)	19
B-3	0-5	Clayey Sand (SC)	29
B-7	0-5	Silty Sand (SM)	40
B-7	7	Silty Sand (SM)	17
B-9	0-5	Silty Sand (SM)	33
B-10	2	Silty Sand (SM)	39
B-12	2	Silty Sand (SM)	41
B-13	2	Silty Sand (SM)	35
B-14	7	Silty Sand (SM)	20
B-15	0-5	Silty Sand (SM)	33

DIRECT SHEAR

Direct shear tests were performed on undisturbed and remolded bulk samples in accordance with ASTM D3080. The bulk samples were remolded to approximately 95 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-6.

HYDRO-CONSOLIDATION

Oedometer tests were performed on relatively undisturbed samples in accordance with ASTM D 5333. After trimming the ends, the sample was placed in the consolidometer and loaded to 0.4 ksf. Thereafter, the samples were incrementally loaded to 0.4, 0.8, 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	IN-SITU MOISTURE CONTENT (%)	TOTAL COMPRESSION (%)	
				BEFORE SATURATION	AFTER SATURATION
B-2	5	Silty Sand (SM)	5.0	2.9	9.8
B-7	5	Silty Sand (SM)	3.9	2.2	4.1
B-8	10	Silty Sand (SM)	2.5	2.3	4.3
B-14	5	Silty Sand (SM)	4.1	2.1	5.8
B-14	7	Silty Sand (SM)	3.3	2.2	5.6

COMPACTION TEST

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

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)
B-3	0-5	Clayey Sand (SC)	135	7.5
B-7	0-5	Silty Sand (SM)	133	8.5
B-15	0-5	Silty Sand (SM)	135	7

EXPANSION INDEX

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

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	EXPANSION INDEX
B-3	0 – 5	Clayey Sand (SC)	2

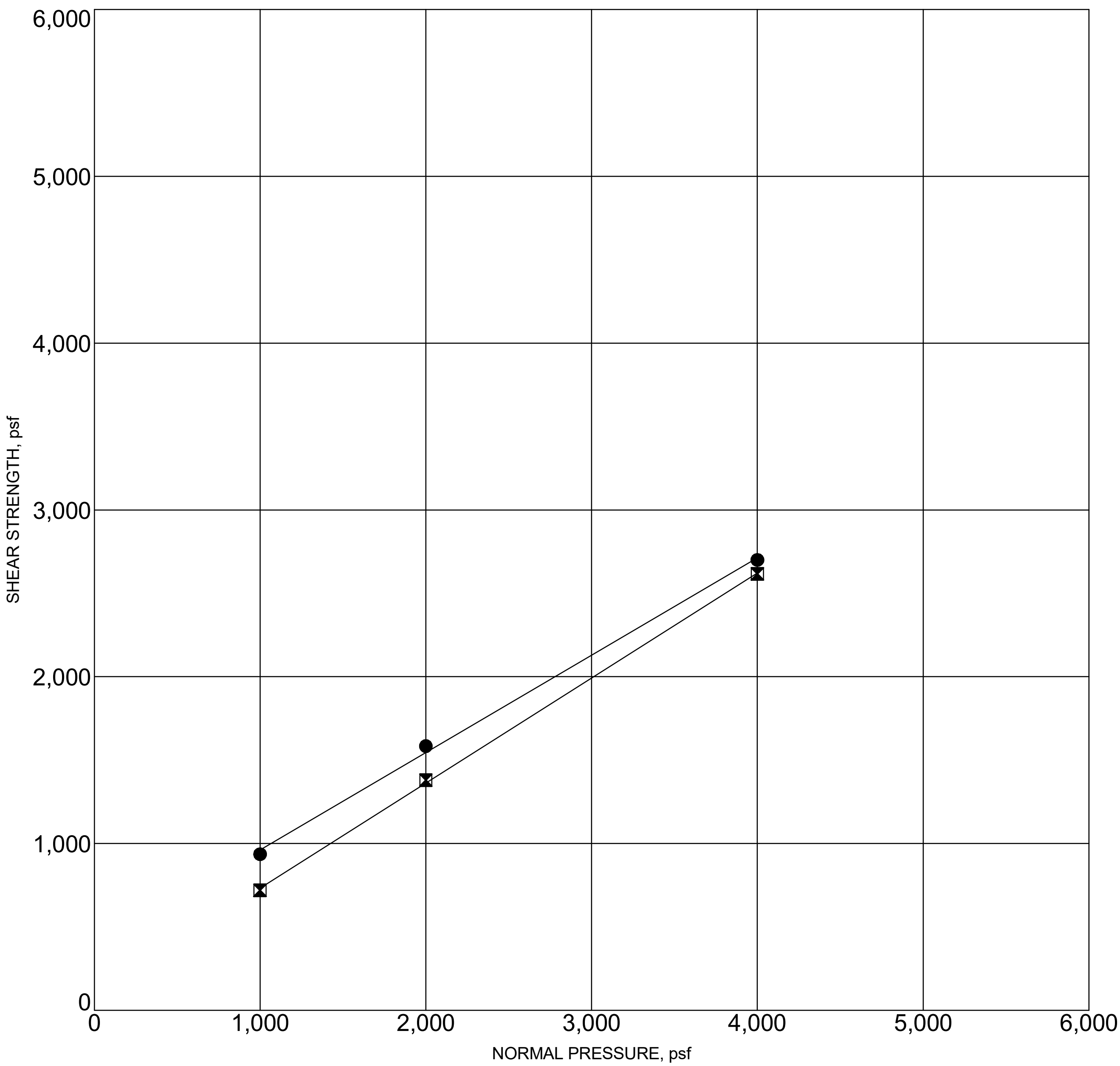
R-VALUE

Suitability of the near-surface soils for pavement was evaluated by conducting an R-value test. The test was performed in accordance with ASTM D 2844 by GeoLogic Associates (GLA) under subcontract to GPI. The result of the test is as follows:

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	R-VALUE BY EXUDATION
B-7	0 – 5	Silty Sand (SM)	45
B-9	0 – 5	Silty Sand (SM)	56

CORROSIVITY

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



● **PEAK STRENGTH**
Friction Angle= 30 degrees
Cohesion= 378 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 32 degrees
Cohesion= 102 psf

Note: Samples remolded to 95% of maximum dry density.

Sample Location		Classification	DD,pcf	MC,%
B-3	0-5	SILTY SAND (SM)	128	7.5

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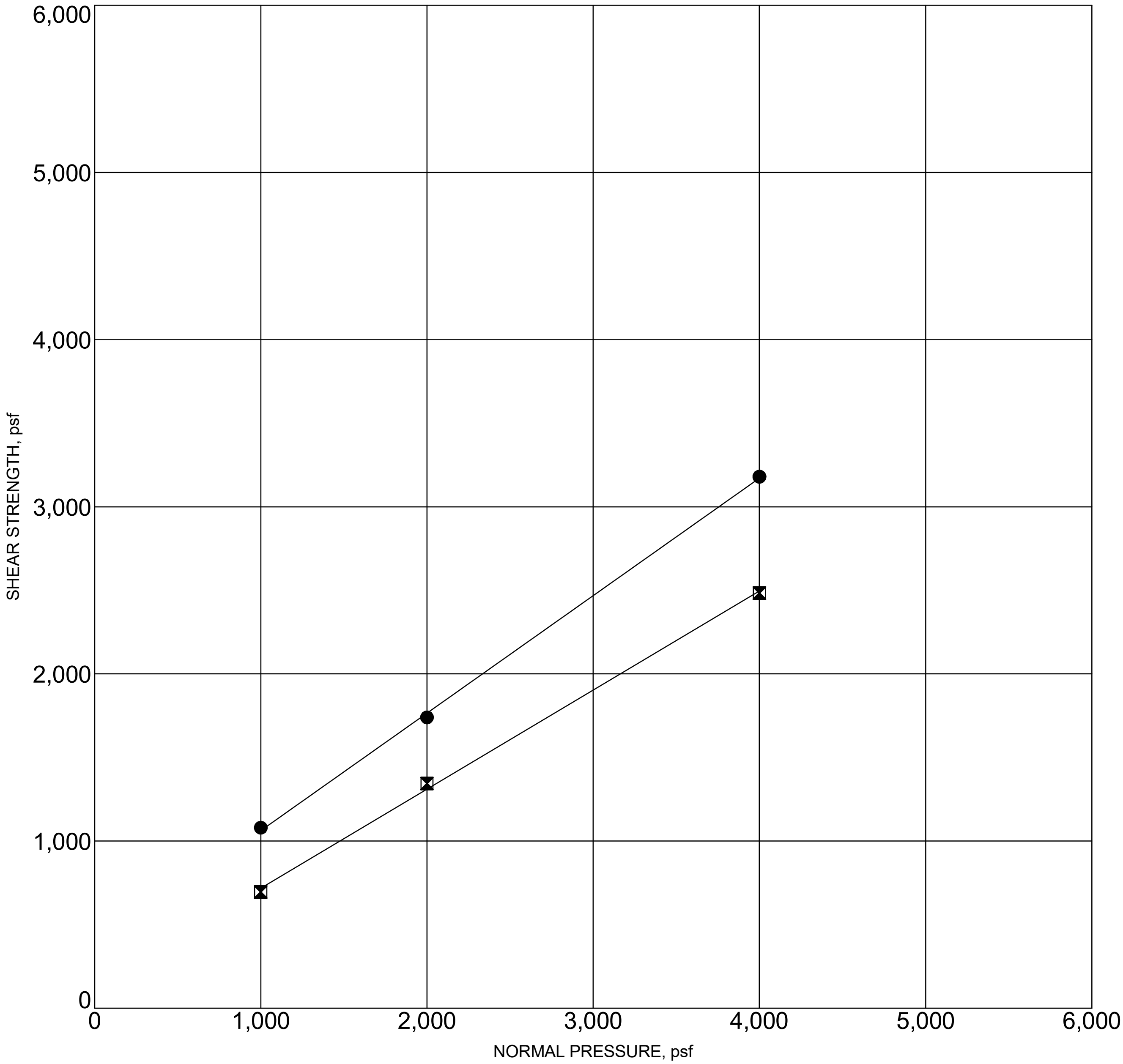
DIRECT SHEAR TEST RESULTS

FIGURE C-1



● **PEAK STRENGTH**
Friction Angle= 35 degrees
Cohesion= 360 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 31 degrees
Cohesion= 126 psf

Sample Location		Classification	DD,pcf	MC,%
B-6	7.0	SILTY SAND (SM)	122	6.7

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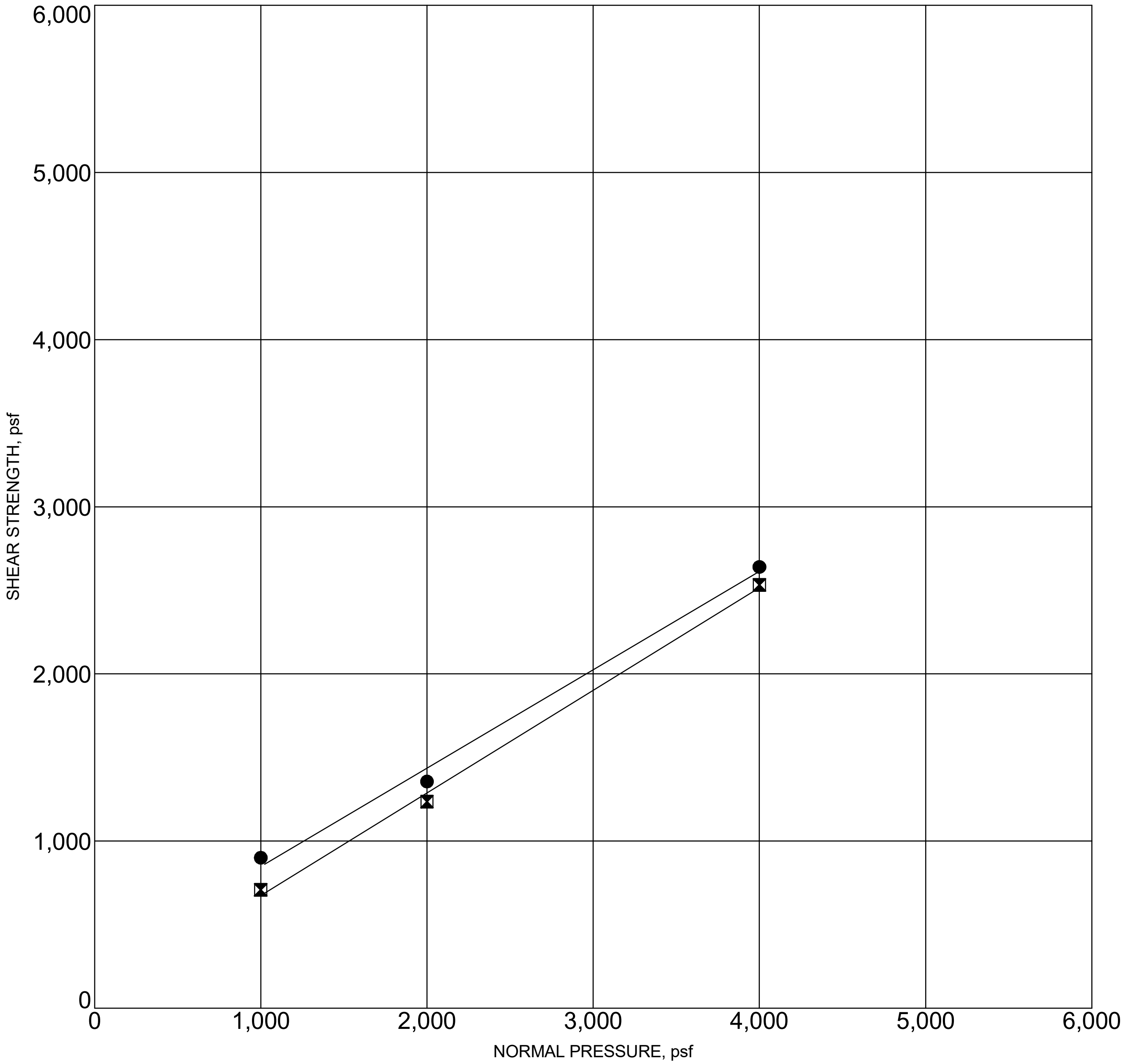
DIRECT SHEAR TEST RESULTS

FIGURE C-2



● **PEAK STRENGTH**
Friction Angle= 30 degrees
Cohesion= 258 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 32 degrees
Cohesion= 60 psf

Note: Samples remolded to 95% of maximum dry density.

Sample Location		Classification	DD,pcf	MC,%
B-7	0-5	SILTY SAND (SM)	126	8.5

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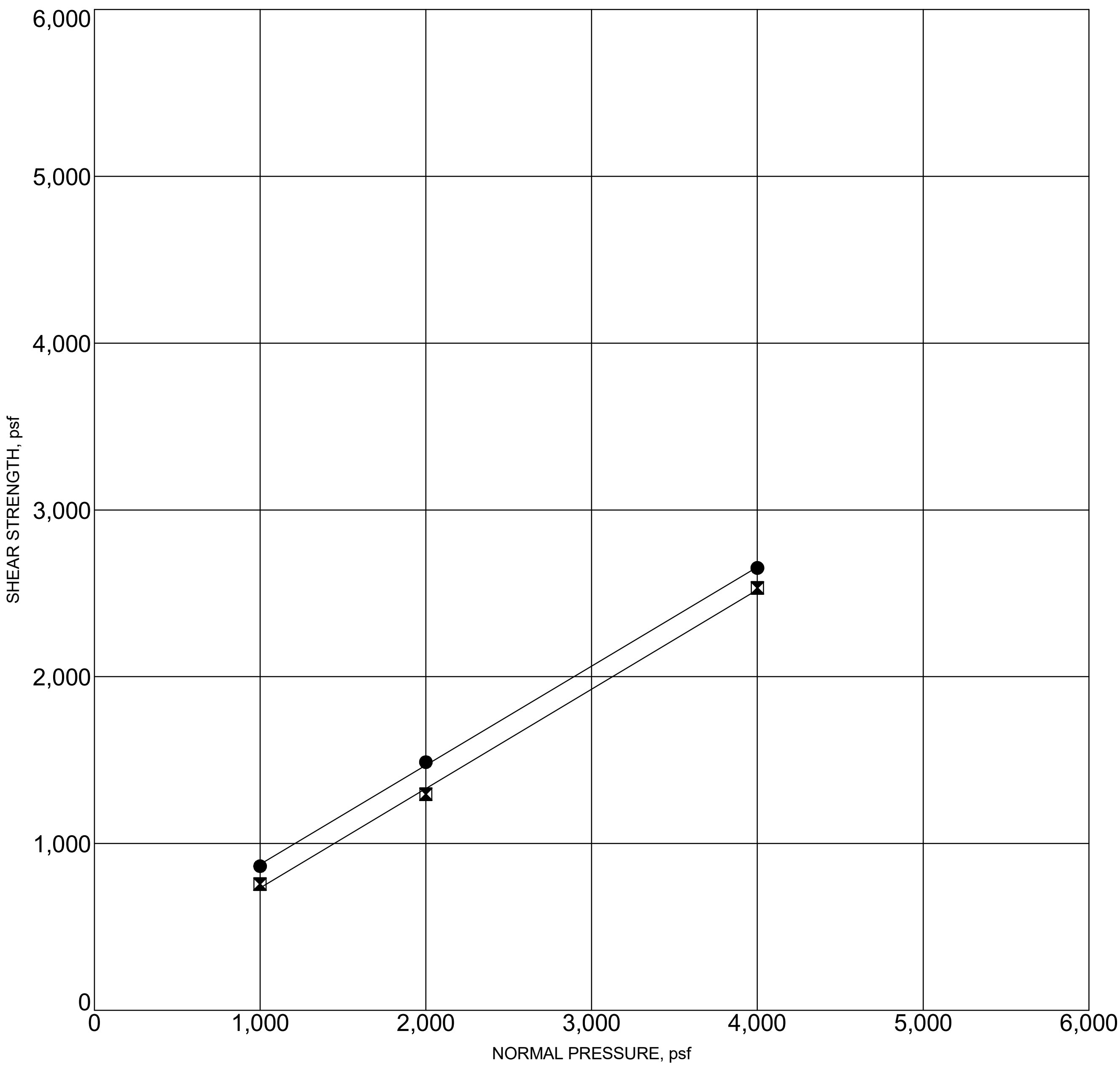
DIRECT SHEAR TEST RESULTS

FIGURE C-3



PEAK STRENGTH
Friction Angle= 31 degrees
Cohesion= 282 psf

ULTIMATE STRENGTH
Friction Angle= 31 degrees
Cohesion= 138 psf

Sample Location		Classification	DD,pcf	MC,%
B-7	7.0	SILTY SAND (SM)	107	4.2

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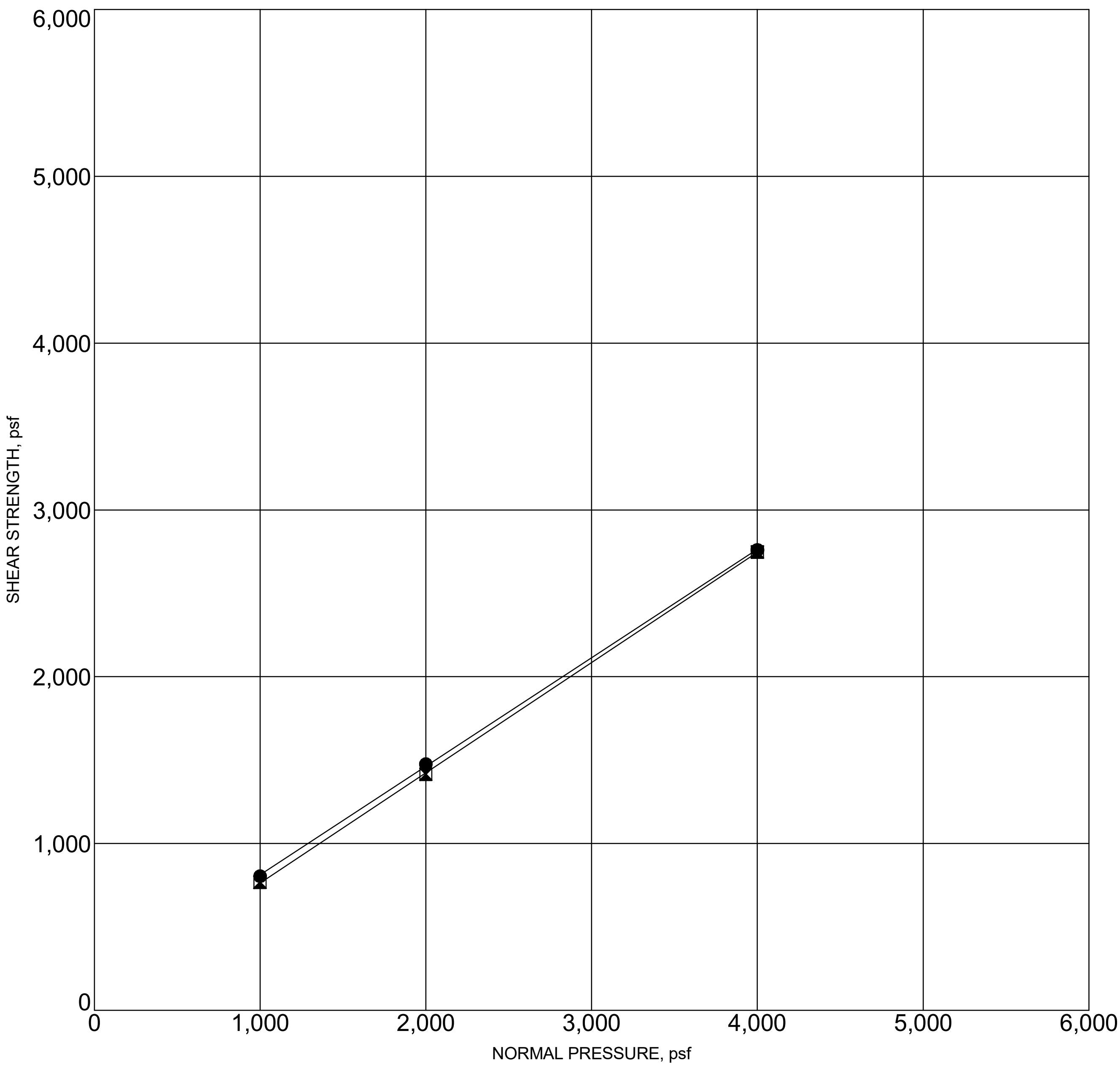
DIRECT SHEAR TEST RESULTS

FIGURE C-4



● **PEAK STRENGTH**
Friction Angle= 33 degrees
Cohesion= 162 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 33 degrees
Cohesion= 102 psf

Sample Location		Classification	DD,pcf	MC,%
B-14	7.0	SILTY SAND (SM)	103	3.3

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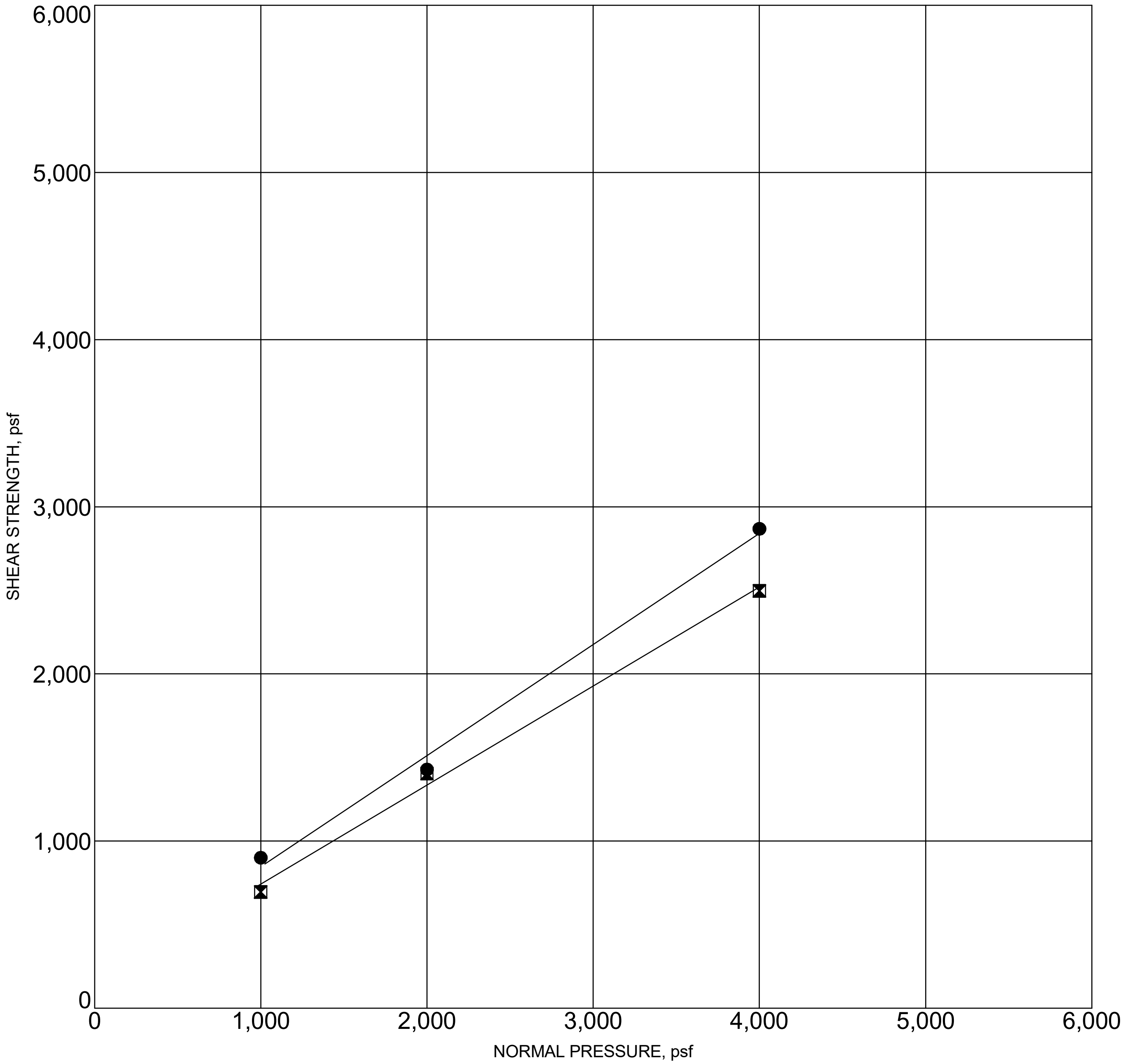
DIRECT SHEAR TEST RESULTS

FIGURE C-5



● **PEAK STRENGTH**
Friction Angle= 34 degrees
Cohesion= 180 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 31 degrees
Cohesion= 150 psf

Note: Samples remolded to 95% of maximum dry density.

Sample Location		Classification	DD,pcf	MC,%
B-15	0-5	SILTY SAND (SM)	128	7.0

PROJECT: PMB RUHS PERRIS PROJECT NO.:3194.2I

GPI

DIRECT SHEAR TEST RESULTS

FIGURE C-6

Soil Analysis Lab Results

Client: Geotechnical Professionals
Inc. Job Name: RUHS - Perris
Client Job Number: 3194.2I
Project X Job Number: S231017E
October 19, 2023

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM G51	ASTM G200	SM 4500-D	ASTM D4327	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D4327	ASTM D4327
Bore# / Description	**Depth**	**Sulfates** SO_4^{2-}		**Chlorides** Cl^-		**Resistivity** As Rec'd | Minimum		**pH**	**Redox**	**Sulfide** S^{2-}	**Nitrate** NO_3^-	**Ammonium** NH_4^+	**Lithium** Li^+	**Sodium** Na^+	**Potassium** K^+	**Magnesium** Mg^{2+}	**Calcium** Ca^{2+}	**Fluoride** F_2^-	**Phosphate** PO_4^{3-}
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ohm-cm)	(Ohm-cm)		(mV)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)
B-4	0-5	31.8	0.0032	17.2	0.0017	113,900	9,380	8.5	116	2.0	2.8	0.1	ND	30.3	4.6	9.6	95.8	4.5	4.9
B-7	0-5	68.3	0.0068	25.7	0.0026	93,800	2,479	8.0	162	1.0	322.7	4.4	ND	66.0	207.4	9.6	80.7	3.4	12.5
B-9	0-5	73.1	0.0073	11.9	0.0012	147,400	6,700	8.0	151	1.3	50.1	2.4	ND	35.6	8.8	13.1	88.1	4.8	2.9

Cations and Anions, except Sulfide and Bicarbonate, tested with Ion Chromatography
mg/kg = milligrams per kilogram (parts per million) of dry soil weight
ND = 0 = Not Detected | NT = Not Tested | Unk = Unknown
Chemical Analysis performed on 1:3 Soil-To-Water extract
PPM = mg/kg (soil) = mg/L (Liquid)

Note: Sometimes a bad sulfate hit is a contaminated spot. Typical fertilizers are Potassium chloride, ammonium sulfate or ammonium sulfate nitrate (ASN). So this is another reason why testing full corrosion series is good because we then have the data to see if those other ingredients are present meaning the soil sample is just fertilizer-contaminated soil. This can happen often when the soil samples collected are simply surface scoops which is why it's best to dig in a foot, throw away the top and test the deeper stuff. Dairy farms are also notorious for these items.

APPENDIX D

TERRA GEOSCIENCES

SEISMIC SHEAR-WAVE SURVEY

PMB-RUHS PERRIS PROJECT

SWC OF PLACENTIA AND HARVILL AVENUES

PERRIS, RIVERSIDE COUNTY, CALIFORNIA

Project No. 233970-1

August 1, 2023

Prepared for:

Geotechnical Professionals Inc.
5736 Corporate Avenue
Cypress, CA 90630

Geotechnical Professionals Inc.
5736 Corporate Avenue
Cypress, CA 90630

Attention Mr. James E. Harris V, P.E., Project Engineer

Regarding: Seismic Shear-Wave Survey
PMB-RUHS Perris Project
SWC of Placentia and Harvill Avenues
Perris, Riverside County, California
GPI Project No. 3170.I

INTRODUCTION

As requested, this firm has performed a seismic shear-wave survey using the multi-channel analysis of surface waves (MASW) and microtremor array measurements (MAM) methods for the above-referenced site. The purpose of this survey was to assess the one-dimensional average shear-wave velocity structure beneath the subject survey area to a depth of at least 100 feet. Geologic mapping of the local area by Morton (2003), indicates the site to be mantled by very old alluvial fan deposits (early Pleistocene age), generally described as being comprised of well-indurated reddish-brown sand deposits. Underlying these deposits at depth is Cretaceous age granitic bedrock, generally comprised of a massive to well-foliated, medium- to coarse-grained, hypautomorphic-granular biotite-hornblende tonalite.

The location of the seismic traverse has been approximated on a captured Google™ Earth (2023), which is presented as the Seismic Line Location Map, Plate 2. Additionally, photographic views of the survey line are presented on Plate 1 for visual and reference purposes. As authorized by you, the following services were performed during this study:

- ***Review of available pertinent published and unpublished geologic and geophysical data in our files pertaining to the site.***
- ***Performing a seismic surface-wave survey by a licensed State of California Professional Geophysicist that included one traverse for shear-wave velocity analysis purposes.***
- ***Preparation of this report, presenting the results of our findings with respect to the shear-wave velocities of the subsurface earth materials.***

Accompanying Map, Illustrations, and Appendices

Plate 1 - Site Photographs
Plate 2 - Seismic Line Location Map
Appendix A - Shear-Wave Model and Data
Appendix B - References

SUMMARY OF SHEAR-WAVE SURVEY

Methodology

The fundamental premise of this survey uses the fact that the Earth is always in motion at various seismic frequencies. These relatively constant vibrations of the Earth's surface are called microtremors, which are very small with respect to amplitude and are generally referred to as background "noise" that contain abundant surface waves. These microtremors are caused by both human activity (i.e., cultural noise, traffic, factories, etc.) and natural phenomenon (i.e., wind, wave motion, rain, atmospheric pressure, etc.) which have now become regarded as useful signal information. Although these signals are generally very weak, the recording, amplification, and processing of these surface waves has greatly improved by the use of technologically improved seismic recording instrumentation and recently developed computer software. For this application, we are mainly concerned with the Rayleigh wave portion of the seismic signals, which is also referred to as "ground roll" since the Rayleigh wave is the dominant component of ground roll.

For the purposes of this study, there are two ways that the surface waves were recorded, one being "active" and the other being "passive." Active means that seismic energy is intentionally generated at a specific location relative to the survey spread and recording begins when the source energy is imparted into the ground (i.e., MASW survey technique). Passive surveying, also called "microtremor surveying," is where the seismograph records ambient background vibrations (i.e., MAM survey technique), with the ideal vibration sources being at a constant level. Longer wavelength surface waves (longer-period and lower-frequency) travel deeper and thus contain more information about deeper velocity structure and are generally obtained with passive survey information. Shorter wavelength (shorter-period and higher-frequency) surface waves travel shallower and thus contain more information about shallower velocity structure and are generally collected with the use of active sources. For the most part, higher frequency active source surface waves will resolve the shallower velocity structure and lower frequency passive source surface waves will better resolve the deeper velocity structure. Therefore, the combination of both of these surveying techniques provides a more accurate depiction of the subsurface velocity structure.

The assemblage of the data that is gathered from these surface wave surveys results in development of a dispersion curve. Dispersion, or the change in phase velocity of the seismic waves with frequency, is the fundamental property utilized in the analysis of surface wave methods. The fundamental assumption of these survey methods is that the signal wavefront is planar, stable, and isotropic (coming from all directions) making it independent of source locations and for analytical purposes uses the spatial autocorrelation method (SPAC). The SPAC method is based on theories that are able to detect "signals" from background "noise" (Okada, 2003). The shear wave velocity (V_S) can then be calculated by mathematical inversion of the dispersive phase velocity of the surface waves which can be significant in the presence of velocity layering, which is common in the near-surface environment.

Field Procedures

One seismic shear-wave survey traverse (Seismic Line SW-1) was performed within the central portion of the subject property, which has been approximated on the Seismic Line Location Map, as presented on Plate 2, for reference. The traverse was located in the field by use of Google™ Earth imagery (2023) and GPS coordinates. For data collection, the field survey employed a twenty-four channel Geometrics StrataVisor™ NZXP model signal-enhancement refraction seismograph. This survey employed both active (MASW) and passive (MAM) source methods to ensure that both quality shallow and deeper shear-wave velocity information was recorded (Park et al., 2005).

Both the MASW and MAM surveys used the same linear geometry array that consisted of a 184-foot-long spread using a series of twenty-four 4.5-Hz geophones that were spaced at regular eight-foot intervals. For the MASW survey, the ground vibrations were recorded using a one second record length at a sampling rate of 0.5-milliseconds. Two seismic records were obtained using 30-foot offsets with respect to the beginning and end of the survey line, utilizing a 16-pound sledge-hammer as the energy source to produce the seismic waves. Each of these shot points used multiple hammer impacts (stacking) to improve the signal to noise ratio of the data.

The MAM survey did not require the introduction of any artificial seismic sources and only background ambient noise was recorded. The ambient ground vibrations were recorded using a thirty-two second record length at a two-millisecond sampling rate with 21 separate seismic records being obtained for quality control purposes. The seismic-wave forms and associated frequency spectrum that were displayed on the seismograph screen were used to assess the recorded seismic wave data for quality control purposes in the field. The acceptable records were digitally recorded on the in-board seismograph computer and subsequently transferred to a flash drive so that they could be subsequently transferred to our office computer for analysis.

Data Reduction

For analysis and presentation of the shear-wave profile and supportive illustrations, this study used the SeisImager/SW™ computer software program developed by Geometrics, Inc. (2004-2021). Both the active (MASW) and passive (MAM) survey results were combined for this analysis (Park et al., 2005). The combined results maximize the resolution and overall depth range in order to obtain one high resolution V_s curve over the entire sampled depth range. These methods economically and efficiently estimate one-dimensional subsurface shear-wave velocities using data collected from standard primary-wave (P-wave) refraction surveys, however, it should be noted that surface waves by their physical nature cannot resolve relatively abrupt or small-scale velocity anomalies. Processing of the data proceeded by calculating the dispersion curve from the input data which subsequently created an initial shear-wave model based on the observed data. This initial model was then inverted in order to converge on the best fit of the initial model and the observed data, creating the final shear-wave model (Seismic Line SW-1) as presented within Appendix A.

Summary of Data Analysis

Data acquisition went very smoothly and the quality was considered to be good. Analysis revealed that the average shear-wave velocity ("weighted average") in the upper 100 feet of the subject survey area is **1,628.1** feet per second as shown on the Shear-Wave Model SW-1, as presented within Appendix A. This average velocity classifies the underlying soils to that of Site Class "**C**" ("Very Dense Soil and Soft Rock" profile), which has a velocity ranging from 1,200 to 2,500 ft/sec (ASCE, 2017; Table 20.3-1).

The "weighted average" velocity is computed from a formula that is used by the ASCE (2017; Section 20.4, Equation 20.4-1) to determine the average shear-wave velocity for the upper 100 feet of the subsurface (V100). This formula is as follows:

V100' = 100/[(T1/V1) + (T2/V2) + ...+ (TN/VN)]

Where t1, t2, t3,...,tn, are the thicknesses for layers 1, 2, 3,...n, up to 100 feet, and v1, v2, v3,...,vn, are the seismic velocities (feet/second) for layers 1, 2, 3,...n.

The shear-wave model displays these calculated layers and associated velocities (feet/second) to the maximum obtained depth of 248 feet, where locally sampled (dark gray shaded area on shear-wave model represents the constrained data). The associated Dispersion Curves (for both the active and passive methods) which show the data quality and picks, along with the resultant combined dispersion curve model, are also included within Appendix A for visual and reference purposes.

It should be noted that when compared with traditional borehole shear-wave surveys, which use vertical body waves, the sources of error (if present) using horizontal surface waves for this project are not believed to be greater than 15 percent.

CLOSURE

The field survey was performed by the undersigned on July 31, 2023, using "state of the art" geophysical equipment and techniques along the selected portion of the subject study area as directed by you. It is important to note that the fundamental limitation for seismic surveys is known as nonuniqueness, wherein a specific seismic data set does not provide sufficient information to determine a single "true" earth model. Therefore, the interpretation of any seismic data set uses "best-fit" approximations along with the geologic models that appear to be most reasonable for the local area being surveyed. Client should also understand that when using the theoretical geophysical principles and techniques discussed in this report, sources of error are possible in both the data obtained and, in the interpretation, and that the results of this survey may not represent actual subsurface conditions.

These are all factors beyond **Terra Geosciences** control and no guarantees as to the results of this survey can be made. We make no warranty, either expressed or implied. If the client does not understand the limitations of this geophysical survey, additional input should be sought from the consultant.

Respectfully submitted,
TERRA GEOSCIENCES

Donn C. Schwartzkopf
Principal Geophysicist
PGP 1002

SITE PHOTOGRAPHS

View looking north along Seismic Line SW-1.

View looking south along Seismic Line SW-1.

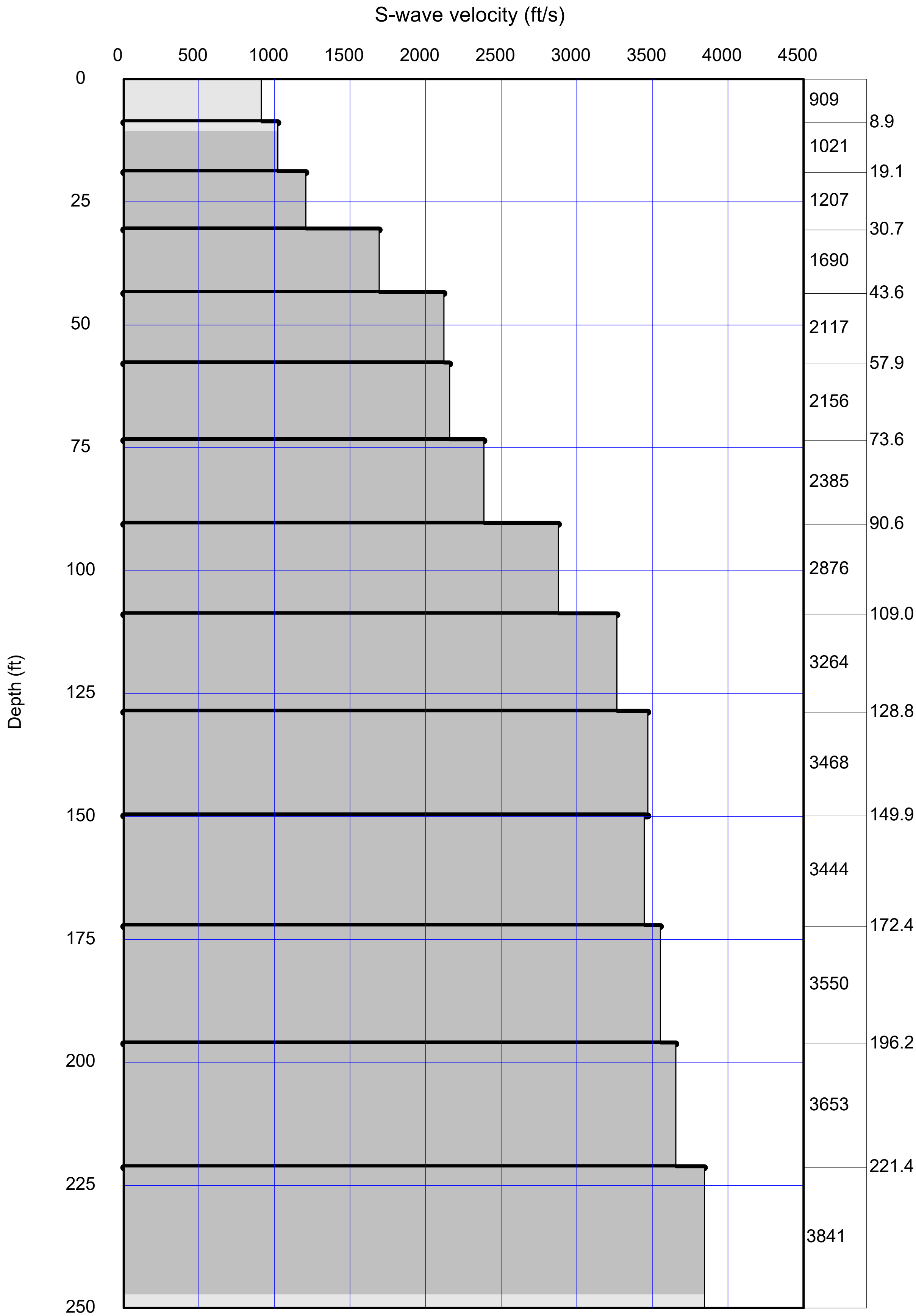
SEISMIC LINE LOCATION MAP

Base Map: Google™ Earth (2023); Seismic shear-wave traverse SW-1 shown as yellow line.

APPENDIX A

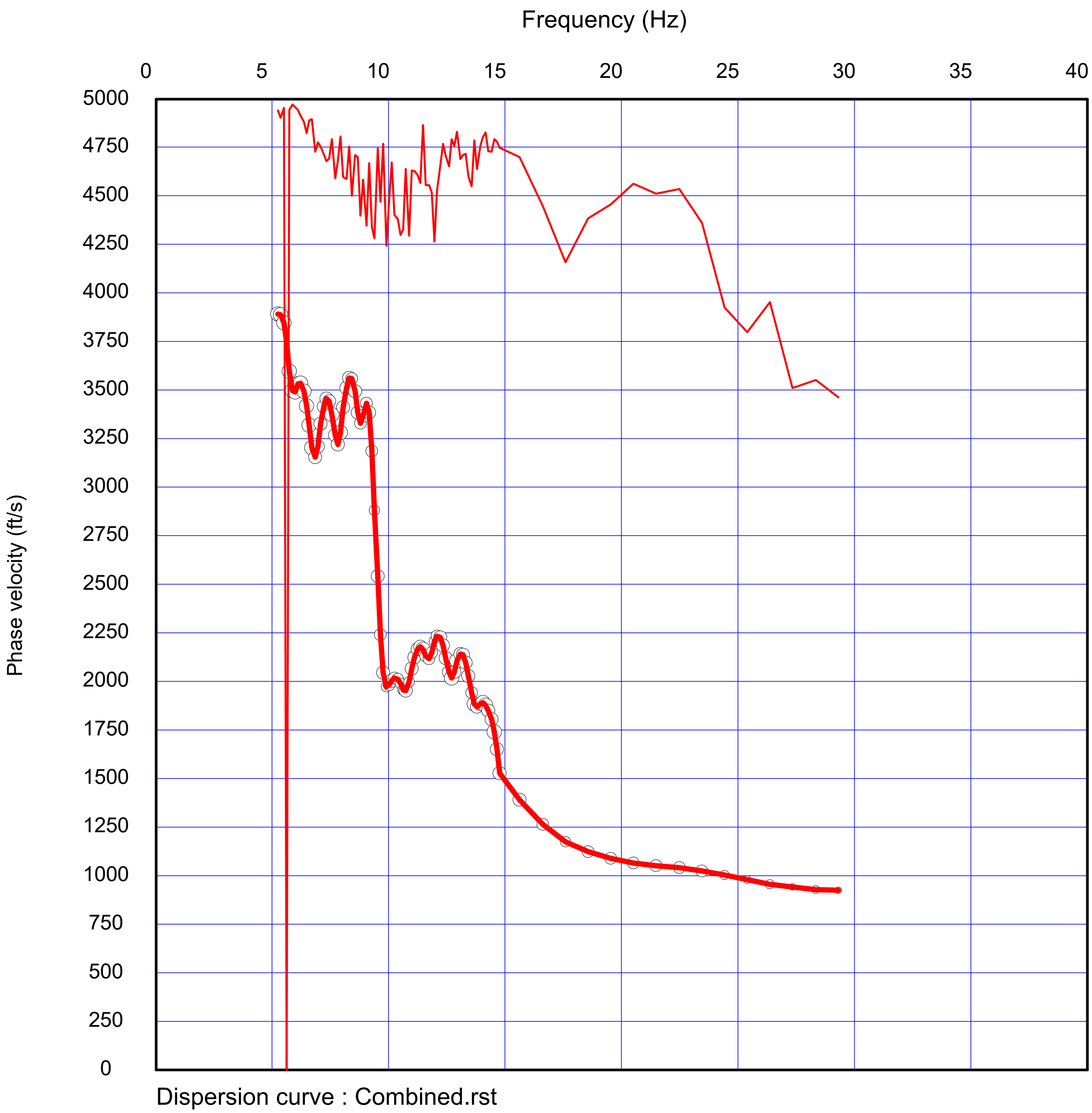
SHEAR-WAVE MODEL AND DATA

SEISMIC LINE SW-1
SHEAR-WAVE MODEL



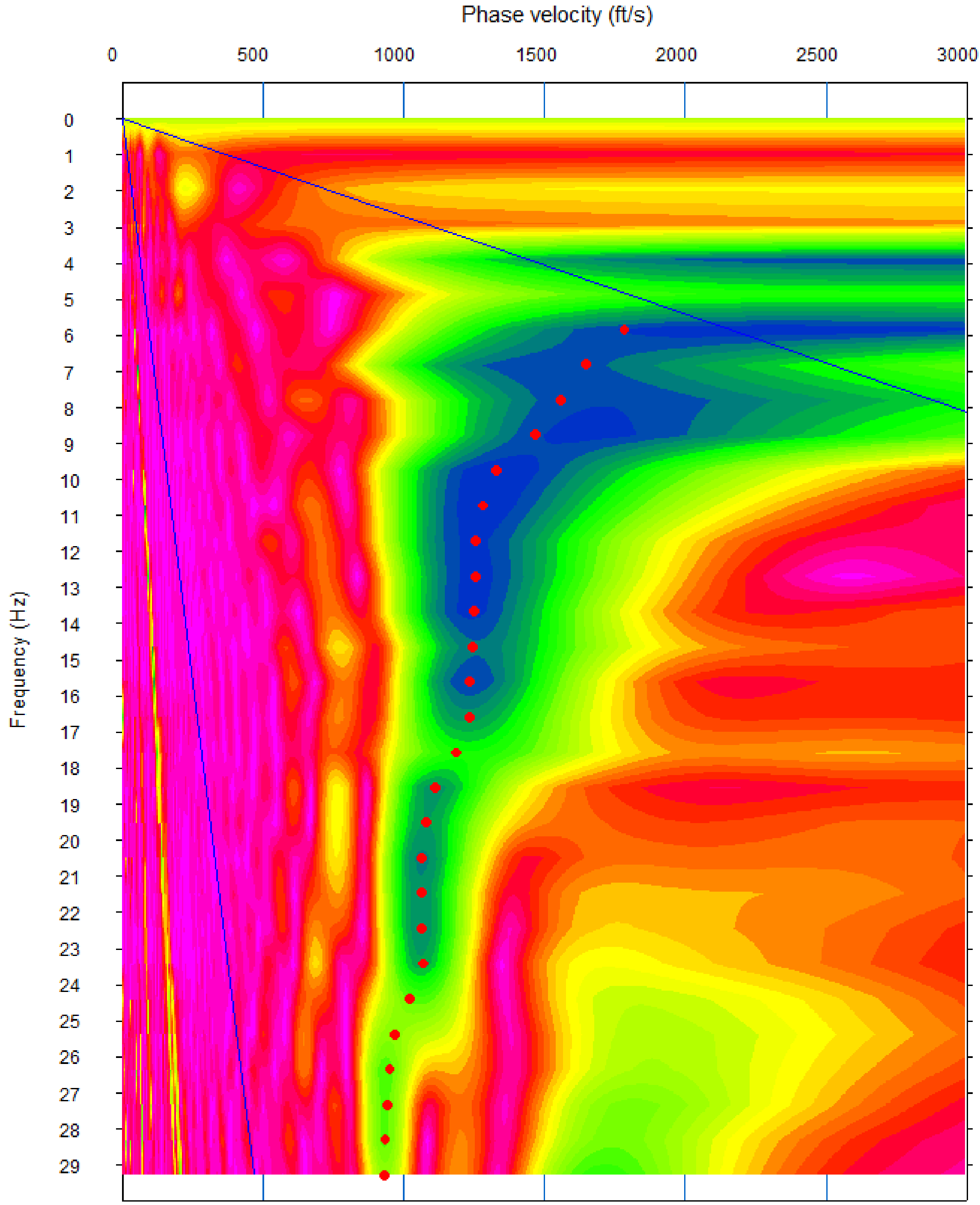
S-wave velocity model (inverted): Final.rst **Average Vs 100ft = 1628.1 ft/sec**

SEISMIC LINE SW-1



COMBINED DISPERSION CURVE

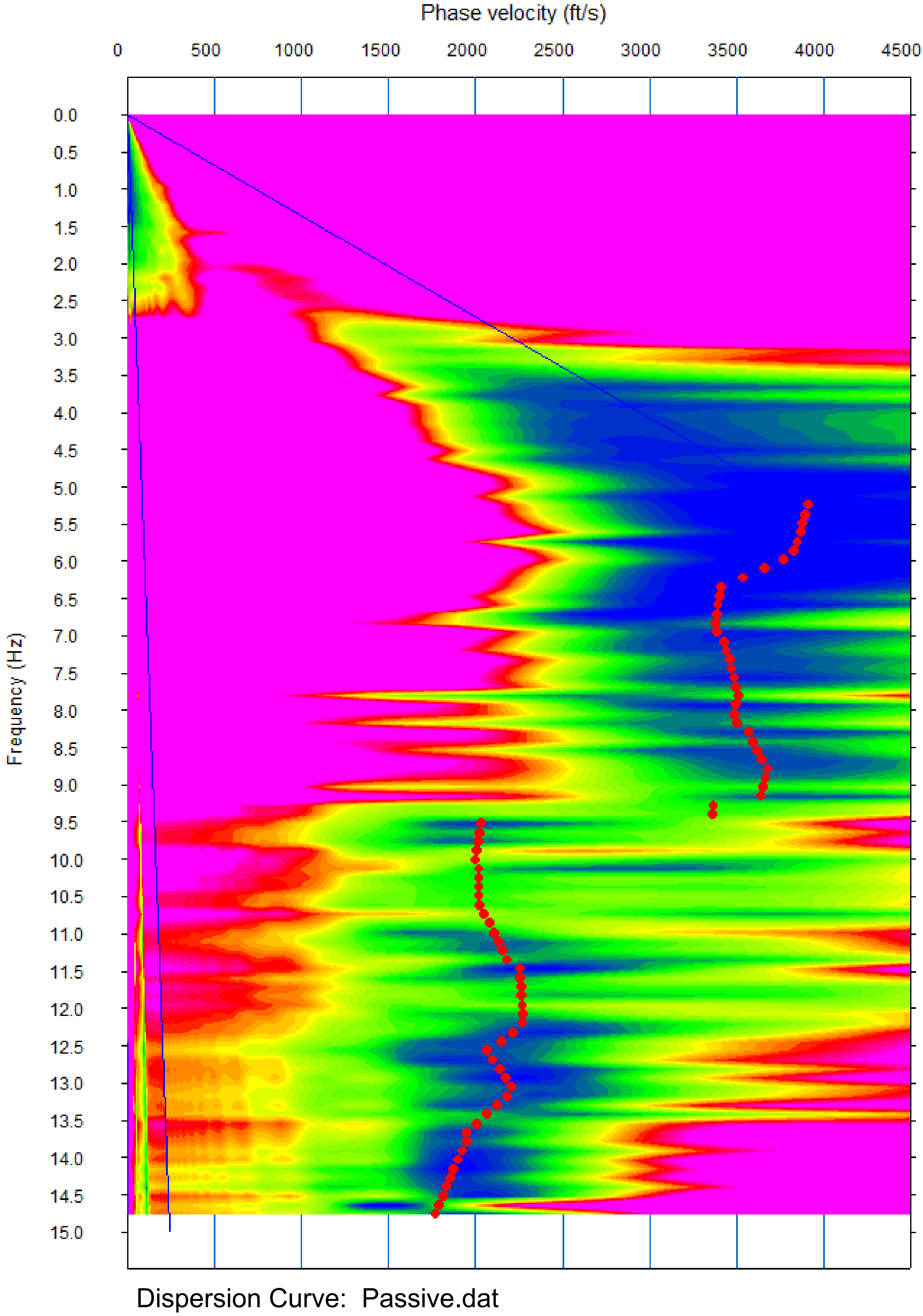
SEISMIC LINE SW-1



Dispersion Cure: Active.dat

ACTIVE DISPERSION CURVE

SEISMIC LINE SW-1



PASSIVE DISPERSION CURVE

APPENDIX B

REFERENCES

REFERENCES

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