GEOTECHNICAL
PROFESSIONALS INC.

GEOTECHNICAL INVESTIGATION
PROPOSED COACHELLA AIRPORT BUSINESS PARK
NWC STATE HIGHWAY 86 AND AIRPORT BOULEVARD
COACHELLA, CALIFORNIA

Prepared for:
Haagen Co., LLC
12302 Exposition Boulevard
Los Angeles, California 90064

Prepared by:
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Project No. 2884.I

September 25, 2018

GEOTECHNICAL
PROFESSIONALS INC.

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Haagen Co., LLC
12302 Exposition Boulevard
Los Angeles, California 90064

Attention: Mr. Chris Fahey

Subject: Report of Geotechnical Investigation
Proposed Coachella Airport Business Park
NWC State Highway 86 and Airport Boulevard
Coachella, California
GPI Project No. 2884.I

Dear Mr. Fahey:

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

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

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

Very truly yours,
Geotechnical Professionals Inc.

James E. Harris, G.E.
Principal

2884-I-01L (09/18)

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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 business park in Coachella, California. The geographical site location is shown on the Site Location Map, Figure 1.

1.2 PROJECT DESCRIPTION

We understand that the proposed improvements at the site will consist of a new business park with single-story buildings of various sizes on a 43-acre parcel. The buildings will include large warehouses, small warehouses, small business, self-storage buildings, a service station, and a drive-thru coffee shop. Preliminary plans indicate the footprint of the buildings will range from approximately 103,300 square feet (sf) for the large warehouse to 4,000 sf for the coffee shop. Currently, thirty-two buildings are planned for the site plus 14 self-storage buildings. The proposed buildings will cover a footprint of approximately 677,000 sf. Additional improvements will include paved vehicular drives and parking as well as landscaping. The preliminary layout of the proposed development is shown on Figure 2.

We have assumed that the buildings will be tilt-up, masonry block, or wood construction. Based on our experience with similar projects, we expect that the structures will have maximum column and wall loads on the order of 30 to 100 kips and 2 to 5 kips per lineal foot, respectively.

Information regarding proposed finish grades for the development is not known at this time. We assume that finish grades will be found at or near existing grades and no changes of grade not more than 3 to 4 feet from existing grades.

Since structural loads or grades can significantly impact the performance of the proposed development, we should perform additional evaluations if the final grades and/or loads vary significantly from those discussed herein.

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 and use of existing geotechnical data, field exploration, laboratory testing, engineering analysis, and the preparation of this report.

The field exploration program consisted of 23 Cone Penetration Tests (CPT's) and 11 exploratory borings. The locations of the explorations are shown on the Site Plan, Figure 2.

The CPT's were advanced to depths ranging from 50 to 80 feet below existing site grades. Detailed logs of the CPT's and a summary of the equipment used are presented in Appendix A. The borings were drilled using hollow-stem auger equipment to depths of 6 to 81½ 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, Atterberg Limits, grain size, compressibility (consolidation), shear strength (direct shear), collapse, R-value, and corrosion. Laboratory testing procedures and results are summarized in Appendix C.

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

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 located on an undeveloped parcel located directly between State Route 86 and an unlined storm water channel (Whitewater River). We observed no evidence of previous development at the site. Historic aerials (historicaerials.com) indicate the land has been undeveloped since prior the 1950's. Minor grading may have been performed along the property lines associated with the channelization of Whitewater River and the roadway construction.

The site is bounded by Airport Road to the south, State Route 86 to the east, undeveloped land to the north, and the storm water channel to the west.

The site is relatively flat sloping very gently to the south. In general, the north side of the site is approximately 8 feet higher than the southern side over a distance of approximately 3,000 feet. Existing ground surface elevations ranged from about -112 to -120 feet MSL based on a topographic map. The Civil Engineer is using a project datum that is 500 feet greater than actual MSL elevations to avoid negative elevations. The elevations on our exploration logs reflect the project datum.

Along the property limits, there are minor slopes adjacent to the site. State Route 86 is, in general, a few feet higher than the site with a minor descending slope. Directly adjacent to the western side of the site, an unpaved maintenance road is located at the top of the storm channel on a berm, which is approximately 2 to 3 feet higher than the project site at the southern end of the site and approximately 8 to 10 feet higher than the project site at the northern end of the site. The berm appears to have been constructed as a levee for the storm water channel. The bottom of the storm channel appears to be on the order of 6 to 8 feet lower than project site.

3.2 SUBSURFACE SOILS

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

Though significant fill soils were not encountered, some fills are expected at the top of the slope immediately adjacent to the storm water channel.

The natural soils consist of interbedded layers of sands, silts, and clays and their mixtures. The consistencies of the sandy soils ranged typically from loose to medium dense in the upper 30 feet and medium dense to dense at greater depths. The sandy soils in the upper 30 feet exhibit moderate strength and moderate to low compressibility characteristics. Very dense sand layers were encountered at depths greater than approximately 55 to 60 feet.

The fine grained soils (silts and clays) are generally firm to stiff with some very stiff to hard layers in the upper 20 feet. In general, the fine-grained soils within the upper 20 to

30 feet varied from firm to stiff and moderately compressible. The underlying fine-grained soils become predominantly stiffer with depth, exhibiting moderate strength and moderate to low compressibility characteristics.

Clay soils were not observed in the near surface soils. The near surface soils can be anticipated to have very low expansion characteristics.

3.3 GROUNDWATER AND CAVING

In the borings, groundwater was measured at depths of 14 to 20 feet immediately after drilling. Due to the method of drilling, accurate depths to groundwater and the potential for caving were very difficult to determine. Groundwater may rise from the deeper measured levels if allow to stabilize with time. Based on the moisture content of the soil samples, we anticipate a stabilized groundwater level at a depth of 10 to 15 feet below existing grade. The historical high groundwater has not been determined in the area by the State of California. We recommend a design groundwater depth of 10 feet for the project

The sandy soils are expected to cave in dry loose soils in the upper 10 feet of the soil profile and severely cave below the groundwater.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 GENERAL

Based on the results of our investigation, it is our opinion that from a geotechnical 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 site is located in an area mapped by the City of Coachella as having a potential for soil liquefaction. Some of the sandy soils underlying the site at depths from 10 to 55 feet below existing grade exhibit a potential for liquefaction in the event of a design earthquake. We estimate that the maximum settlements at the site in the event of a design earthquake would range from approximately 2½ to 3 inches. See Section 4.2 for methods to mitigate settlement.

- To help limit total and differential settlements of the proposed buildings to the magnitudes described above either mat foundations, pile foundations and pile supported structural floor slabs, or ground improvement will be required. If ground improvement is performed to limit settlements to an acceptable magnitude, the buildings can be supported on conventional spread footings.

- Prior to construction of the building foundations (conventional or mat), disturbed soils and a portion of dry, compressible soils should be removed and replaced as properly compacted fill. Deeper removals will be required if conventional footings tied together with grade beams are used for buildings. 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 and under minor structures, in order to provide a consistent, moist layer of soils for uniform support. The depth of removals and details regarding grading are provided in “Earthwork” section of this report.

- The near surface soils exhibit soluble sulfate contents that are detrimental to concrete. The foundation concrete should conform to the requirements for severe sulfate exposure as outlined in ACI 318, Section 4.3.

- The on-site soils should be considered severely 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 MITIGATION OF SETTLEMENT

The maximum allowable total and differential settlements for shallow foundations and slabs on grade, from all sources, is typically on the order of 1½ inches and ¾-inch, respectively in Southern California. For mat foundations, the maximum allowable total and differential settlements, from all sources, is typically on the order of 4 inches and 2 inches, respectively. Sources include static (gravity) and seismic causes.

The site soil profile includes compressible and potentially liquefiable soils in the upper 55 feet. The potential building settlement under both static and earthquake loads could be mitigated by specially designed spread footings, mat foundations, pile foundations, or in-place ground modification methods (ground improvement) supporting conventional shallow foundations.

Structural mitigation measures for the impacts of the seismic settlements of shallow foundations could be implemented by the Structural Engineer. The risk associated with not mitigating seismic settlement by the methods in the above paragraph should be fully understood. With proper structural mitigation measures, the risk would include the building not being fully functional after a design seismic event causing the predicted seismic settlement. The floor slab and footings of the building may need to be re-leveled by compaction grouting or underpinning following a seismic event. The utility connections may also need to be repaired. The structural mitigation must be designed such that the structure would not collapse during a design seismic event causing a life and safety issue. On past, similar projects the footings were tied together with grade beams to help mitigate the impacts of seismic settlement and supported on a relatively thick layer of properly compacted soil. The details of the structural mitigation should be determined by the Project Structural Engineer.

Other potential structural mitigation methods are also provided in “Foundation Type” section of this report.

Pile foundations should be designed to resist both static loads and downdrag loads caused from seismic settlement by embedding the pile to sufficient depths below the liquefiable soil layers.

We reviewed typical methods used in Southern California such as vibro-replacement (stone columns), deep soil mixing (soil-cement columns), and rammed aggregate piers.

Vibro-replacement utilizes a large vibrating probe (mandrel) to create a cavity which is filled with gravel or crushed stone, and compacted as the mandrel is removed. The result is a stone column with the stone pushed laterally into the soil. Based on past discussions with a geotechnical specialty contractor, stone columns would not be effective to reduce the total settlements (static and seismic) due high silt or clay content of liquefiable soils, and relatively thin layers of liquefiable soils at the site. Stone columns are effective for densifying thicker, clean, loose sand layers, which are not prevalent at the site.

Rammed aggregate piers consist of drilled holes that are filled with aggregate base that is mechanically compacted as it is placed and were considered. Rammed aggregate piers are not effective in densifying surrounding soils and typically do not extend to the depth of soils exhibiting a potential for liquefaction.

Deep soil mixing involves the creation of soil/cement mixed columns extending through the soft compressible soil deposits and portion of the liquefiable soils. The resultant is similar to that of stone columns in that the method results in lower compressibility and increased shear strengths of soils below slabs and foundations. Deep soil mixing can reduce both anticipated static and seismic settlement in both the siltier sands and the significant layers of cohesive soils at the site. The soil mixing would have to reduce the static and seismic settlements to a magnitude acceptable to the Structural Engineer (typically 1½ inches or less) in order to utilize conventional spread foundations.

The proposed structures can be supported on deep foundations. Because of the anticipated seismic settlement, a pile supported structural slab would also be needed, if the previously described settlements are not tolerable and risk of floor slab damage is not acceptable. In order to limit settlement to an acceptable value, pile foundations would need to resist the downdrag of soils from liquefaction occurring above a depth of 45 feet. The total length of the piles to support this downdrag load as well as the building loads would likely be on the order of 65 to 75 feet. For the single story buildings proposed at the site, pile foundations are not likely to be economically feasible. If pile foundations were to be selected for the project, it is our opinion that the most feasible type of deep foundation would be an Augercast Pile. This type of foundation consists of a pressure-grouted pile constructed in a hollow-stem auger. The pile is especially suited for construction below groundwater. If desired, supplemental recommendations can be provided.

If mat foundations or shallow foundations with structural mitigation are not acceptable for any of the buildings, an evaluation should be made if pile foundations or deep soil mixing are economically feasible for the single story buildings planned for the site. Our report can be provided to specialty design-build contractors experienced in deep soil mixing and/or augercast piles to determine which of these methods appear to be the most cost effective to sufficiently reduce settlement of the buildings.

4.3 SEISMIC CONSIDERATIONS

4.3.1 General

The site is located in a seismically active area 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, 2016 edition. For the 2016 CBC, a Soil Class D may be used. The seismic code values can be obtained directly from the tables in the building code using the above values and appropriate United States Geological Survey web site (earthquake.usgs.gov). The Project Structural Engineer should determine the seismic design method.

4.3.2 Strong Ground Motion Potential

Based on published information (earthquake.usgs.gov), the most significant fault in the proximity of the site is the San Andreas Fault, which is located about 2½ 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 0.80g for a magnitude 6.9 earthquake. This acceleration has been computed using the mapped Maximum Considered Geometric Mean peak ground acceleration from ASCE 7-10 (ASCE, 2010) and a site coefficient (F_{PGA}) based on site class. The predominant earthquake magnitude was determined using a 2-percent probability of exceedance in a 50-year period, or an average return period of 2,475 years. The structural design will need to incorporate measures to mitigate the effects of strong ground motion.

4.3.3 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.

4.3.4 Liquefaction

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

The site is located within an area mapped by the City of Coachella as having a potential for soil liquefaction (City of Coachella, 2014). The State of California has not determined a historical high groundwater depth in the project area. Groundwater was encountered at depths of 14 to 20 feet below existing grades immediately after drilling in our recent explorations.

Revisions to the 2016 California Building Code, ASCE 7-10, and Special Publication 117A (CGS, 2008) require that the ground motion used for this evaluation be based on the Peak Ground Acceleration (PGA_M) adjusted for site class effects. This value is computed using the mapped Maximum Considered Geometric Mean (MCE_G) peak ground acceleration for a Site Class B and a site coefficient, F_{PGA}. In accordance with the 2016 CBC, we considered a ground acceleration of 0.80g for a magnitude 6.9 earthquake for our analyses, which corresponds to the PGA_M obtained using the methods described above.

The potential for liquefaction was evaluated using the methods presented by the NCEER and updated by Robertson (Robertson, 2009) and modifications provided in Special Publication 117A. Criterion for liquefaction susceptibility of the fine-grained soils was based on methods presented in Bray and Sancio (2006). We used a groundwater depth of 10 feet for our evaluations.

The soils encountered in our CPT's below the groundwater level are predominantly layers of medium dense to dense silty sands interbedded with layers of firm to very stiff layers of silts and clays. At depths of approximately 35 to 45 feet, the layers of silty sands generally become dense to very dense and silts and clays become very stiff to hard.

In general, the clays below foundation and groundwater level are resistant to liquefaction based on criteria in Bray and Sancio (2006). This conclusion is based upon the plasticity indices of soils below design water level being greater than 12. A portion of the clays have plasticity indices between 12 and 18, which are more resistant to liquefaction but susceptible to cyclic mobility.

Based on our evaluation of the field data, generally isolated and thin layers of silty sands occurring at depths of approximately 10 to 55 feet exhibit a potential for liquefaction. Based on our analyses, we computed an overall potential seismic-induced liquefaction settlement of 2½ to 3 inches. Differential seismic settlement is estimated to be 1¼- to 2-inches across a span of 40 feet.

4.3.5 Lateral Spreading

A potential result of soil liquefaction at the site is lateral spreading. Lateral spreading is defined as the horizontal movement of soils resulting from the loss of shear strength during liquefaction combined with either a sloping ground surface or a nearby free face condition. Conditions contributing to the potential for lateral spreading include the extent and severity of liquefaction, grain size of liquefiable materials, distance to the causative fault, and extent of surficial grade changes.

The unlined storm water channel on the east side of the site is an open face excavation (free face condition) with an estimated depth on the order of approximately 6 to 8 feet. The slope to the storm water channel is approximately 100 to 150 feet from the western property line at the site. The project site is essentially flat with a very minor ground slope of about 0.3 percent towards the southeast paralleling the storm water channel.

These conditions along with the liquefaction potential of underlying soils are consistent with areas that may be subject to lateral spreading.

We evaluated the potential for lateral spreading towards the open face excavation of the storm water channel. A lateral displacement was determined using the calculated Lateral Displacement Index (LDI) as described by Zhang et. al. (2004) for the site geometry. The analyses evaluate the topographic and subsurface information to determine the potential lateral displacement induced by the movement of the site towards the free face caused by severe liquefaction of a continuous layer beneath the site.

The LDI was calculated for soil layers having the potential for liquefaction utilizing the CPT data for the site, we calculated LDI for the CPT's within the western boundary of the project site. Utilizing this geometry and the analytical method described above, we determined the potential total lateral-spreading induced displacement from approximately 3 to 12 inches could occur at the western portion of the site.

As the discussed above, lateral spreading requires continuous liquefiable layers across the site in a westerly direction to the drainage channel. We reviewed 9 cross sections of CPT data toward the channel. Evidence of distinct and consistent liquefiable layers across the site toward the channel could only be identified in a few of the cross sections. Based on this data, lateral spreading has a moderate potential to adversely impact the site in limited areas of the site with displacements on the order discussed above.

Other empirical methods (Youd,1997) indicate that for lateral spreading to occur, the layers subject to liquefaction should be continuous across the site and have an overburden-normalized standard penetration test blowcount (sandy soils) of less than 15. Our data did not indicate continuous layers across the site with these blowcounts.

If mat foundations or footings tied together with grades beams are used to support the buildings, minor amounts of lateral spreading as discussed above is not expected to adversely impact the building from a life and safety standpoint. Some minor displacement of the buildings, utility connections, and parking lot along the west side of the site due to lateral spreading in the event of a design earthquake may occur but repairing the structures, pavements and other site improvements would likely be more cost-effective than ground improvement methods. Ground improvement required to resist the potential impacts of lateral spreading would likely consist of a deep barrier wall with multiple rows of soil-cement columns along the entire western boundary of the property.

4.4 EARTHWORK

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

4.4.1 Clearing

Prior to grading, the areas to be developed should be stripped of vegetation, pavements, foundations, and cleared of all 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 any further grading.

4.4.2 Excavations

Excavations at the site will include removal of unsuitable soils, foundation excavations and trenching for utility lines.

Prior to placement of fills or construction of the buildings, existing disturbed soils and a portion of the dry, compressible 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 buildings supported on spread footings extend to 7 feet below existing grades or 5 feet below footings, whichever is deeper. We recommend that removals within the footprints of buildings supported on mat foundations extend to 4 feet below existing grades or 2 feet below foundations, whichever is deeper. The purpose of these removals is to remove and recompact the dry, low-density natural soils near the ground surface and disturbed soils, if encountered. If undocumented fills are encountered within the building footprints, we also recommend removal and replacement as properly compacted fill.

In proposed pavement areas, removals should extend to 2-feet below existing grades. Existing grade refers to elevations at locations of explorations.

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

The depth of removals may be reduced by 2-feet if the exposed subgrade soils in the building and parking areas are moisture conditioned and densified in-place using heavy vibratory equipment as discussed in “Subgrade Preparation”. The contractor will need to demonstrate that the recommended compaction has been achieved by provided test pits for access for density testing.

The removals should extend laterally beyond the edge of footing a minimum distance equal to the depth of overexcavation/compaction below finish grade (i.e. a 1:1 projection below the edge of footings).

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 4 feet below adjacent grade. For deeper cuts up to 10 feet, the slopes should be properly shored or sloped back to at least 1:1 or flatter. Caving should be anticipated in excavations attempted in dry sands or below the groundwater level. As such, dewatering, shoring, excavation, and backfill methods should be developed by the contractor for structures or utilities that are anticipated to extend below the groundwater. 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. All excavations and shoring systems should meet the minimum requirements given in the most current State of California Occupational Safety and Health Standards.

4.4.3 Subgrade Preparation

After the recommended cuts and removals are performed and prior to placing fills or construction of the proposed improvements, the subgrade soils should be scarified to a depth of 12 inches, moisture conditioned, and compacted to at least 95 percent (90 percent cohesive soils) of the maximum dry density, determined in accordance with ASTM D1557. Moistening of the dry sandy soils anticipated at the site can usually be accomplished by deep ripping and liberal watering (including “rainbirds” or flooding) prior to compaction.

If the removals are reduced by 2-feet, as provided as an option in “Excavations” section of this report, the exposed subgrade soils in building and parking areas should be moisture-conditioned and proofrolled a minimum of six passes with a heavy vibratory pad-foot-roller (minimum 40,000 pounds dynamic force) until the soils have been compacted to at least 95 percent (90 percent cohesive soils) of maximum dry density. Proofrolling should continue until the required compaction has been achieved to a depth of at least 2 feet below the exposed subgrade, as measured by in-place density testing.

The fill soils within the upper 12 inches below building floor slabs and the pavement base should be compacted to dry densities equal to at least 95 percent (90 percent cohesive soils) of maximum dry density (ASTM D-1557).

4.4.4 Material for Fill

The surficial on-site soils are, in general, suitable for use as compacted fill. On-site clays, if encountered, should not be used where non-expansive fill is specified or recommended. Imported fill material should be predominately granular (containing no more than 40 percent fines - portion passing No. 200 sieve) and non-expansive (Expansion Index of 20 or less). The import should also exhibit a minimum R-value of 40, consistent with the existing near surface soils. 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.

While not anticipated at the site, on-site inert demolition debris, such as concrete and asphalt, may be reused in the compacted fills provided approval is provided by 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.

4.4.5 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 cohesive) for of the maximum dry density in building and pavement areas, in accordance with ASTM D-1557. In pavement areas, including the parking structure pavements on grade, the upper 12 inches should be compacted to 95 percent (90 percent for cohesive soils). 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.

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. Granular fills should be placed at a moisture content of 0 to 2 percent over the optimum moisture content. The moisture content of the soils encountered in the upper 5 to 10 feet of the explorations was generally well below the optimum moisture content. As such, significant moisture conditioning (wetting) may 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.

4.4.6 Shrinkage and Subsidence

Shrinkage is the loss of soil volume caused by compaction of fills to a higher density than before grading. Subsidence is the settlement of in-place subgrade soils caused by loads generated by large earthmoving equipment. For earthwork volume estimating purposes, an average shrinkage value of about 15 to 20 percent and subsidence of 0.1 to 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.

4.4.7 Trench/Wall Backfill

Utility trench and wall backfill consisting of the on-site material or imported sand should be mechanically compacted in lifts. Letting or flooding should not be permitted. The on-site silts (or clays if encountered) should not be used in retaining wall backfill. 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 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.

4.4.8 Observation and Testing

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

4.5 SHALLOW FOUNDATIONS

4.5.1 General

On similar projects, proposed buildings have been supported on spread footings tied together laterally with grade beams provided the static and seismic settlements as designed by the Project Structural Engineer.

In order to help mitigate the seismic settlements (total and differential) at the site after remedial grading, the Structural Engineer should also consider additional structural mitigation beyond connecting the footings with grade beams. The actual method of structural mitigation should be determined by the Project Structural Engineer.

As discussed in Section 4.2 “Mitigation of Settlement” of the report, mat foundations, pile foundations, or ground improvement may also be used to mitigate the potential liquefaction settlements. Recommendations for a mat foundation are provided in Section 4.5 of this report. GPI can provide recommendations for the other mitigation methods, if the static and seismic settlements (total and differential) are beyond the structural mitigation methods provided above and mat foundations are not feasible for the building type.

The subsurface soils should be prepared in accordance with the recommendations given in this report.

4.5.2 Allowable Bearing Pressures – Spread Footings

Based on the shear strength and elastic settlement characteristics of the natural and recompacted on-site soils, static allowable net bearing pressures of up to 3,000 pounds per square foot (psf) may be used for both continuous footings and isolated column footings for the proposed building addition or other lightly-loaded structures. These bearing pressures are for dead-load-plus-live-load, any may be increased one-third for short-term, transient, wind and seismic loading. The actual bearing pressure used may be less than the value presented above and can be based on economics and structural loads to determine the minimum width for footings as discussed below. The maximum edge pressures induced by eccentric loading or overturning moments should not be allowed to exceed these recommended values.

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

STATIC BEARING PRESSURE (psf)	MINIMUM FOOTING WIDTH (inches)	MINIMUM FOOTING* EMBEDMENT (inches)
3,000	48	24
2,500	24	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.

Total static settlement of the column footings (100 kips maximum load) is expected to be on the order of 1-inch or less. Total static settlement of the wall footings (2 to 4 kips per lineal foot maximum load) is expected to be on the order of ¾-inch or less. Maximum differential settlements between similarly loaded adjacent footings or along a 40-foot span are expected to be on the order of ½-inch or less. Similar settlements are anticipated for lightly loaded structures supported on 2 feet of properly compacted fill.

The above settlements should be included with the anticipated seismic settlement caused by liquefaction when evaluating the total settlement of the building or other lightly loaded structures.

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

4.5.3 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 0.35 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 foundations are poured tight against the compacted fill. These values may be used in combination without reduction.

4.5.4 Footing Excavation Observation

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

4.6 MAT FOUNDATIONS

The sizes and foundation pressures for mat foundations may vary significantly for the different buildings planned for the project. We evaluated mat foundations for a warehouse building with a footprint of 160 feet by 400 feet and for an office building with a footprint of 300 feet by 75 feet. We assumed that the mat pressure for the warehouse building may be on the order of 300 psf and 150 psf for the office building. Other building sizes and mat pressure can be evaluated as the project develops.

The bearing pressure near the center of a mat (approximately 400 feet length and 160 feet width in dimension) is assumed to be on the order of 300 psf for the warehouse building. We estimate the ground surface under the center portions of the loaded area having the above dimensions and the aforementioned applied pressure will settle approximately ¾-inch. The outside edge of this area under the same loading conditions is expected to settle approximately $^{3}/_{8}$-inch. The outside corner of this area under the same loading conditions is expected to settle less than ¼-inch.

The bearing pressure near the center of a mat (approximately 300 feet length and 75 feet width in dimension) is assumed to be on the order of 150 psf for the office building. We estimate the ground surface under the center portions of the loaded area having the above dimensions and the aforementioned applied pressure will settle approximately ½-inch. The outside edge of this area under the same loading conditions is expected to settle approximately ¼-inch. The outside corner of this area under the same loading conditions is expected to settle less than ¼-inch.

The static settlements assume a uniformly applied pressure and do not include the effects (stiffness) of the mat. The actual settlement of the mat will depend on the stiffness of the mat, its ability to distribute the loads and should be determined by the Structural Engineer.

The above settlements should be included with the anticipated seismic settlement caused by liquefaction when evaluating the total settlement of the building.

For the structural analysis of the mat foundation, we recommend using an uncorrected modulus of subgrade reaction of 180 pci. This value is based on a 1-foot square bearing area and medium dense sands and stiff clays. We recommend this modulus be reduced by 75 percent to a value of 45 pci to account for the size of the mat foundation.

The allowable soil bearing pressure will be significantly greater than the average bearing pressures required for the mat foundation as discussed above. At localized thickened areas of the mat, such as columns and point of load applications, a static allowable net bearing pressure of 2,000 pounds per square foot may be used subject to the dimensions provided for spread footings. These allowable bearing pressures are for dead-plus-live loads, and may be increased one-third for short-term, transient, wind and seismic loading.

We should review the final mat design to confirm the estimated values.

4.7 FOUNDATION CONCRETE

Laboratory testing by HDR (Appendix C) indicates that the near surface soils exhibit a soluble sulfate content of 137 to 4,080 mg/kg (0.01 to 0.44 percent by weight). For the 2016 CBC, foundation concrete should conform to the requirements for severe sulfate exposure as outlined in ACI 318, Section 4.3.

4.8 BUILDING FLOOR SLABS

Slab-on-grade floors should be supported on non-expansive, granular compacted soils (Expansion Index less than 20) as discussed in the “Placement and Compaction of Fill” section. On-site clayey soils, if encountered, should not be placed within 2 feet of the finished grade in building floor slab area.

Settlement of the slab-on-grade floors should be anticipated in the event of liquefaction from a seismic event. Distress to the floor slabs may need to be repaired and/or the floor slabs may need to be releveled.

A vapor/moisture retarder should be placed under slabs that are to be covered with moisture-sensitive floor coverings (wood, vinyl, tile, etc.). Currently, common practice is to use a 10 or 15 mil polyethylene product or a 15-mil polyolefin product such as Stego Wrap for this purpose. Whether the concrete slab is placed directly on the vapor barrier or on a clean sand layer between the slab and vapor retarder is a decision for the Project Architect and General Contractor, as it is not a geotechnical issue. If covered by sand, the sand layer should be about 2 inches thick and contain less than 5 percent by weight passing the No. 200 sieve. Based on our explorations and laboratory testing, the near-surface soils at the site are not suitable for this purpose. The sand layer should be nominally compacted using light equipment. The sand placed over the vapor retarder should only be slightly moist. If the sand gets wet (for example as a result of rainfall or excessive moistening) it must be allowed to dry prior to placing concrete. Care should be taken to avoid infiltration of water into the sand layer after placement of the concrete slab, such as at slab cut-outs and other exposures. A sand layer is not required beneath the vapor retarder, but we take no exception if one is provided.

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

For lateral resistance design, a coefficient of friction value of 0.35 between aggregate base or select fill and concrete may be used. For a slab on a visqueen moisture barrier, a coefficient of 0.1 should be used. For a concrete slab on Stego Wrap, a coefficient of 0.3 may be used, which is consistent with recommendations provided by the American Concrete Institute (ACI).

For elastic design of slabs-on-grade supporting sustained concentrated loads, a modulus of subgrade reaction (k) of 180 pounds per cubic inch (pounds per square inch per inch of deflection) may be used. This value is for a 1-foot by 1-foot square loaded area and should be adjusted by the structural designer for the area of the proposed building slab using appropriate elastic theory.

Although not tested, the upper silty sands and sandy silts are anticipated to have a low potential for expansion. As such, there are no geotechnical requirements for minimum floor slab thickness or reinforcing.

4.9 LATERAL EARTH PRESSURES

Based on information available to us at the time this report was prepared, no major retaining walls or basements were planned on the site. The following recommendations are provided for walls less than 8 feet in height. We recommend that non-expansive, granular soils be used as wall backfill.

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

At-rest pressures should be used for restrained walls that remain rigid enough to be essentially non-yielding. At-rest pressures imposed by a fluid weighing 52 pounds per cubic foot should be used for <u>granular</u> backfill.

If the design of retaining walls requires seismic earth pressures to be included, a lateral pressure equivalent to a fluid with a unit weight of 25 pcf may be used. This pressure should be combined with the active earth pressure presented above for a total lateral earth pressure (active plus seismic) equal to a fluid weighing 60 pcf. If walls are designed using at-rest pressures, a total lateral earth pressure may be limited to 60 pcf.

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 wall backfill should be well-drained to relieve possible hydrostatic pressure or designed to withstand these pressures. A drain consisting of perforated pipe and gravel wrapped in filter fabric should be used. One cubic foot of rock should be used for each lineal foot of pipe. The fabric (non-woven filter fabric, Mirafi 140N or equivalent) should be lapped at the top.

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 HDR indicate that the soils are severely corrosive to ferrous metals (resistivity measurements of 160 to 1,040 ohm-cm). GPI does not practice corrosion engineering. Should the use of buried metal pipe be proposed, a corrosion engineer, such as HDR, 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. The introduction of water into the existing fill soils can result in subsidence. 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. The use of the clayey soils, if encountered, within 2 feet of the slab subgrade should not be permitted unless differential heave is tolerable. This includes exterior sidewalks, stamped concrete, non-traffic pavement, pavers, etc. Prior to placement of concrete, the subgrade should be prepared as recommended in the "Subgrade Preparation" section of this report.

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. Based on our subsurface investigation, groundwater was encountered within 14 feet of the existing ground surface at portions of the site and the upper 15 feet of the soil profile consists predominantly of loose to medium dense silty sands and firm to stiff sandy silts.

Our analysis indicate that the silty sands and sandy silts in the upper 15 feet of the soil profile exhibit a potential for settlement from liquefaction upon saturation. Storm water infiltration into the underlying soils may adversely impact the proposed buildings and improvements as well as the adjacent public roadways. We do not recommend storm water infiltration for the subject site unless the risk is acceptable for potential liquefaction settlement of soils underlying infiltration areas.

If on-site infiltration of storm water is used, we recommend that infiltration areas adjacent to the building and property lines should be avoided. We recommend any infiltration device be located at least 40 feet from the proposed building and property lines. Storm water infiltration should also not be allowed within 10 feet vertically from the current groundwater level which excludes most buried chamber systems.

If infiltration devices are proposed for the project, the rate of infiltration should be determined by on-site percolation tests at the location and depth of the proposed infiltration devise. Infiltration tests should be performed in accordance to Riverside County guidelines (Riverside, 2011).

4.14 PAVED AREAS

Preliminary pavement design has been based on an assumed R-value of 40. 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 SUBGRADE

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	7
		Portland Cement Concrete	**Aggregate Base Course**
Auto Parking	4	6	---
Circulation Drives	5	6	---
Truck Drives	6	6.5	---

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 Portland cement concrete used for paving should have a modulus of rupture of at least 550 psi (equivalent to an approximate compressive strength of 3,700 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 Haagen Co., LLC and their consultants in designing the proposed development. The report is not intended to be suitable for reuse on extensions or modifications of the project or for use on 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.

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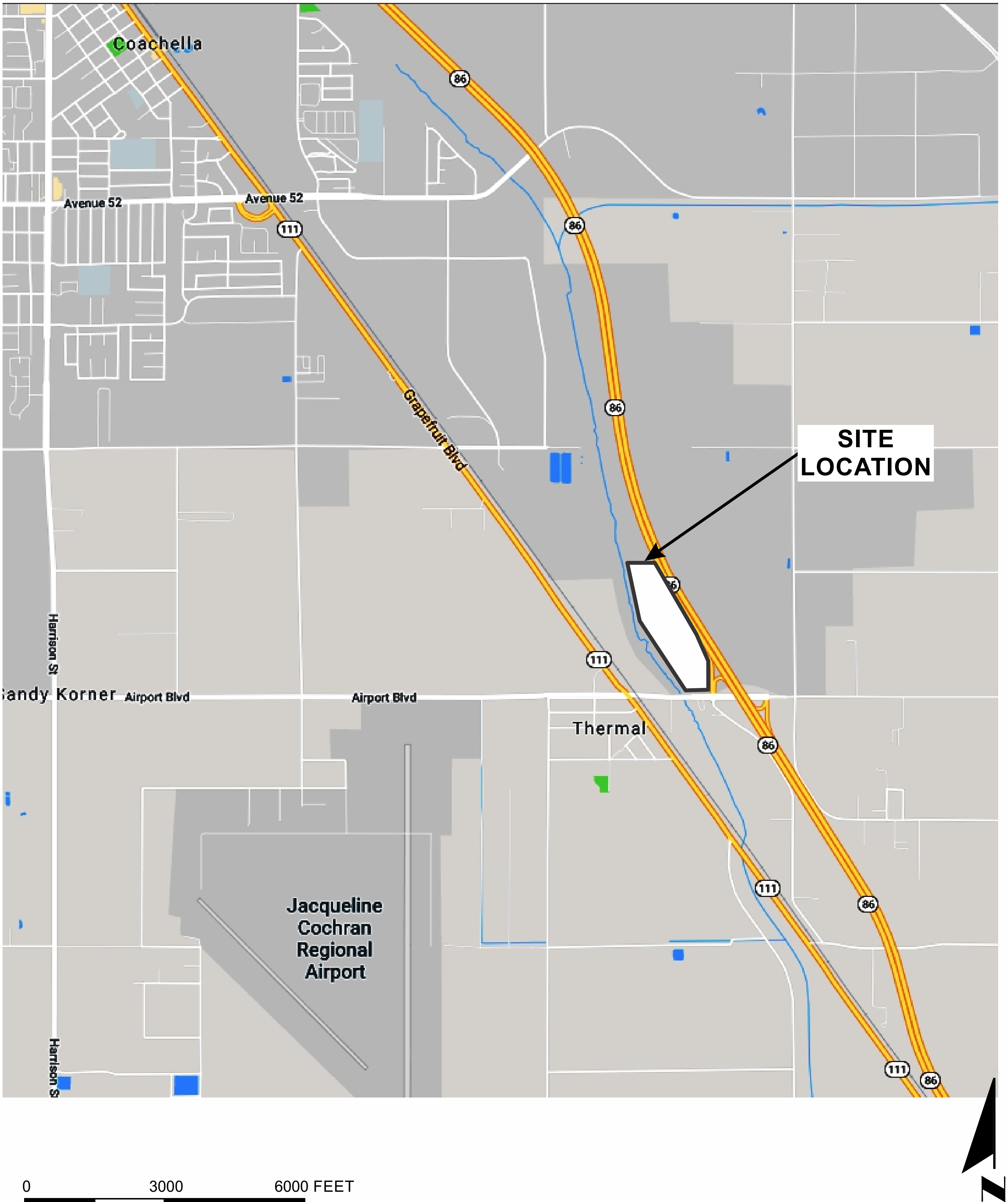


BASE MAP REPRODUCED FROM GOOGLE MAPS © 2018

GPI GEOTECHNICAL PROFESSIONALS, INC.

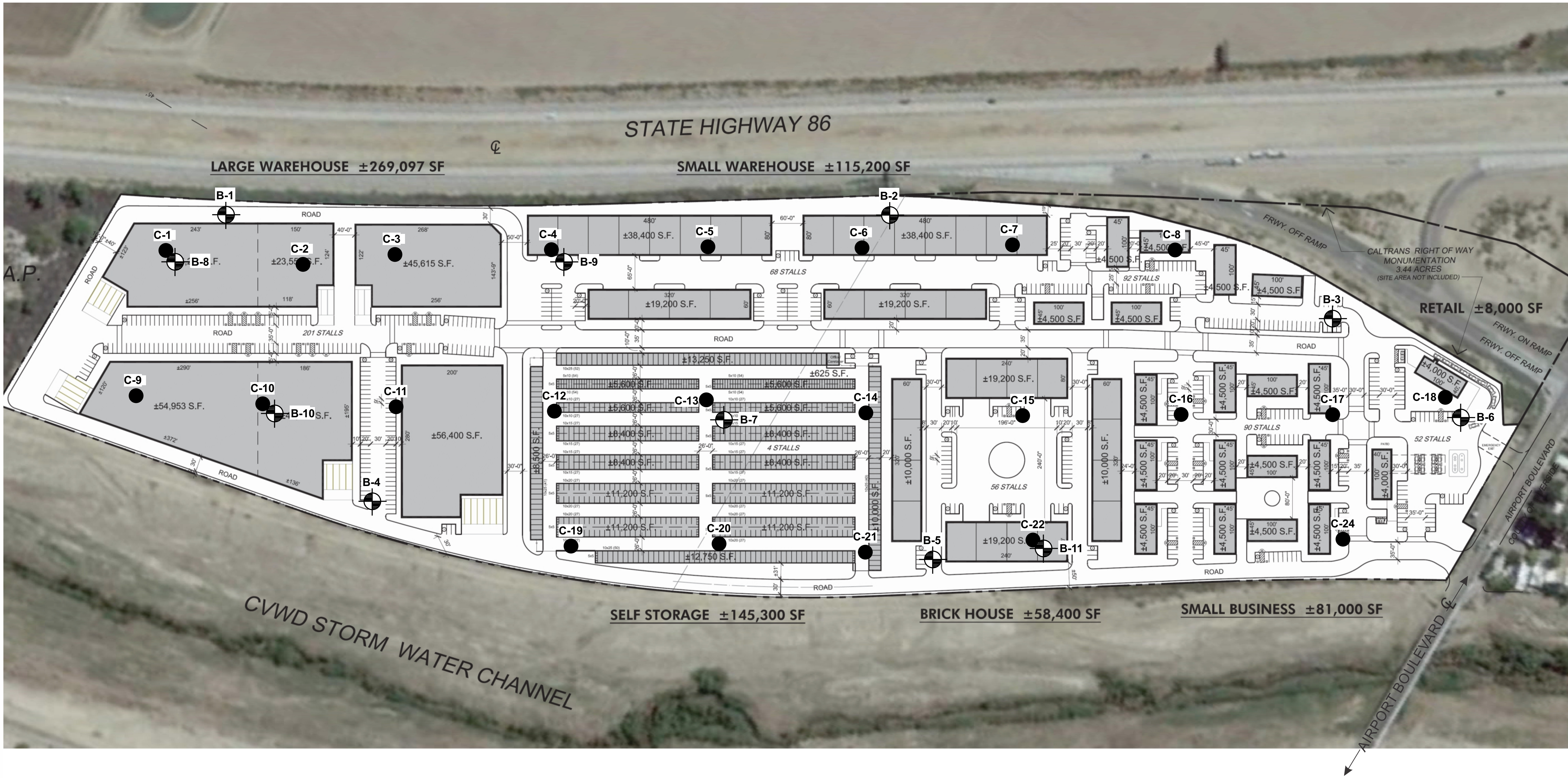
COACHELLA BUSINESS PARK

GPI PROJECT NO.: 2884.I

SCALE: 1" = 3000'

SITE LOCATION MAP

FIGURE 1



EXPLANATION

B-11 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING

C-24 APPROXIMATE LOCATION AND NUMBER OF CONE PENETRATION TEST

0 200 400 FEET

BASE MAP REPRODUCED FROM PROPOSED SITE PLAN BY MCKENTLY MALAK ARCHITECTS DATED 5/1/18

GPI GEOTECHNICAL PROFESSIONALS, INC.

COACHELLA BUSINESS PARK

GPI PROJECT NO.: 2884.I

SCALE: 1" = 200'

SITE PLAN

FIGURE 2

APPENDIX A

APPENDIX A

CONE PENETRATION TESTS

Twenty-three Cone Penetration Tests (CPT's) were performed at the site. The soundings were advanced to depths of 50 to 80 feet below existing grades. One proposed CPT was not performed due to the location being inaccessible due to soft sands. The locations of the CPT's are shown on the Site Plan, Figure 2.

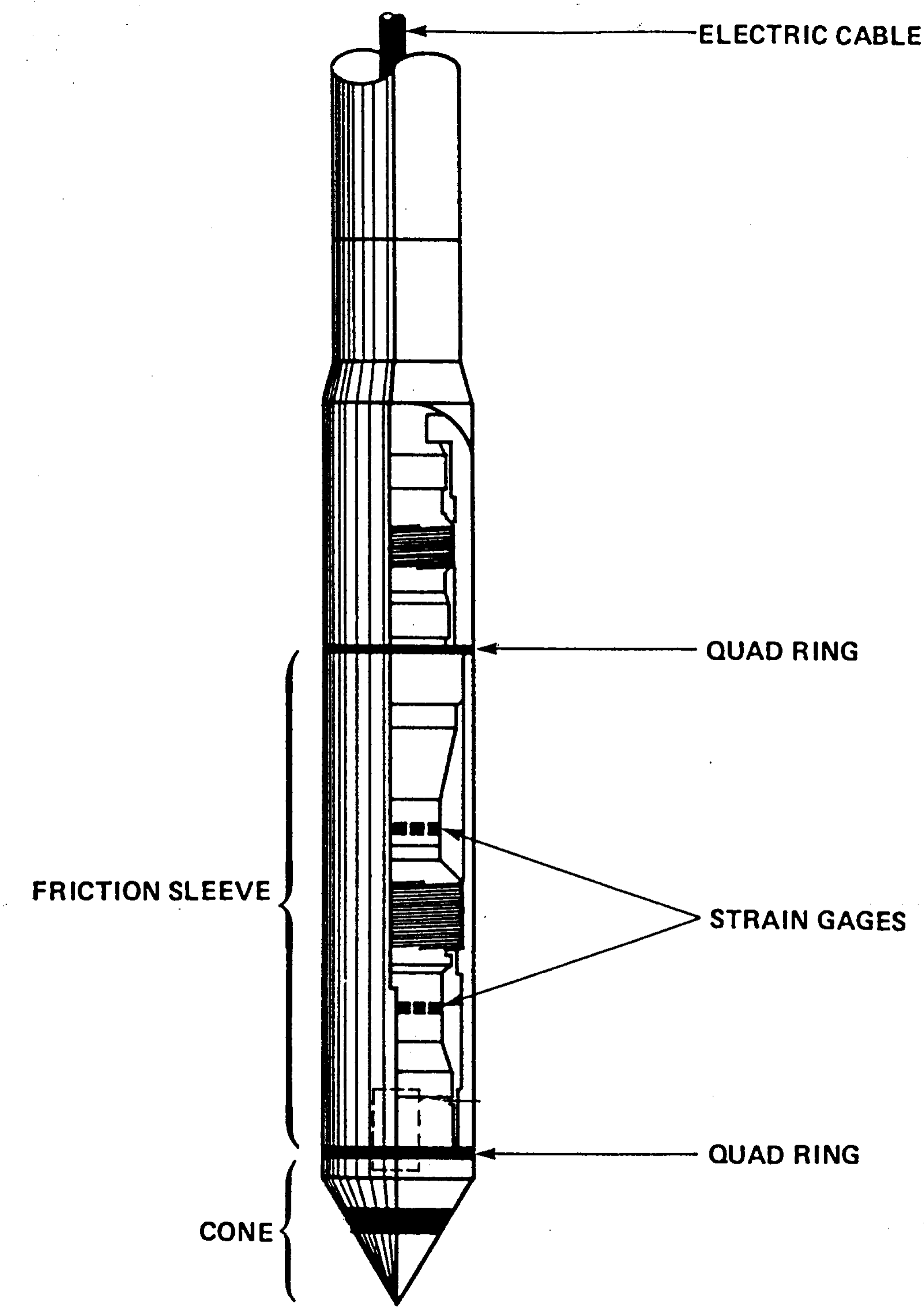
The Cone Penetration Test consists of pushing a cone-tipped probe into the soil deposit while simultaneously recording the cone tip resistance and side friction resistance of the soil to penetration (refer to Figure A-1). The CPT described in this report was conducted in general accordance with ASTM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

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

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 are presented in Figures A-2 through A-24 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 soil descriptions were prepared by GPI.

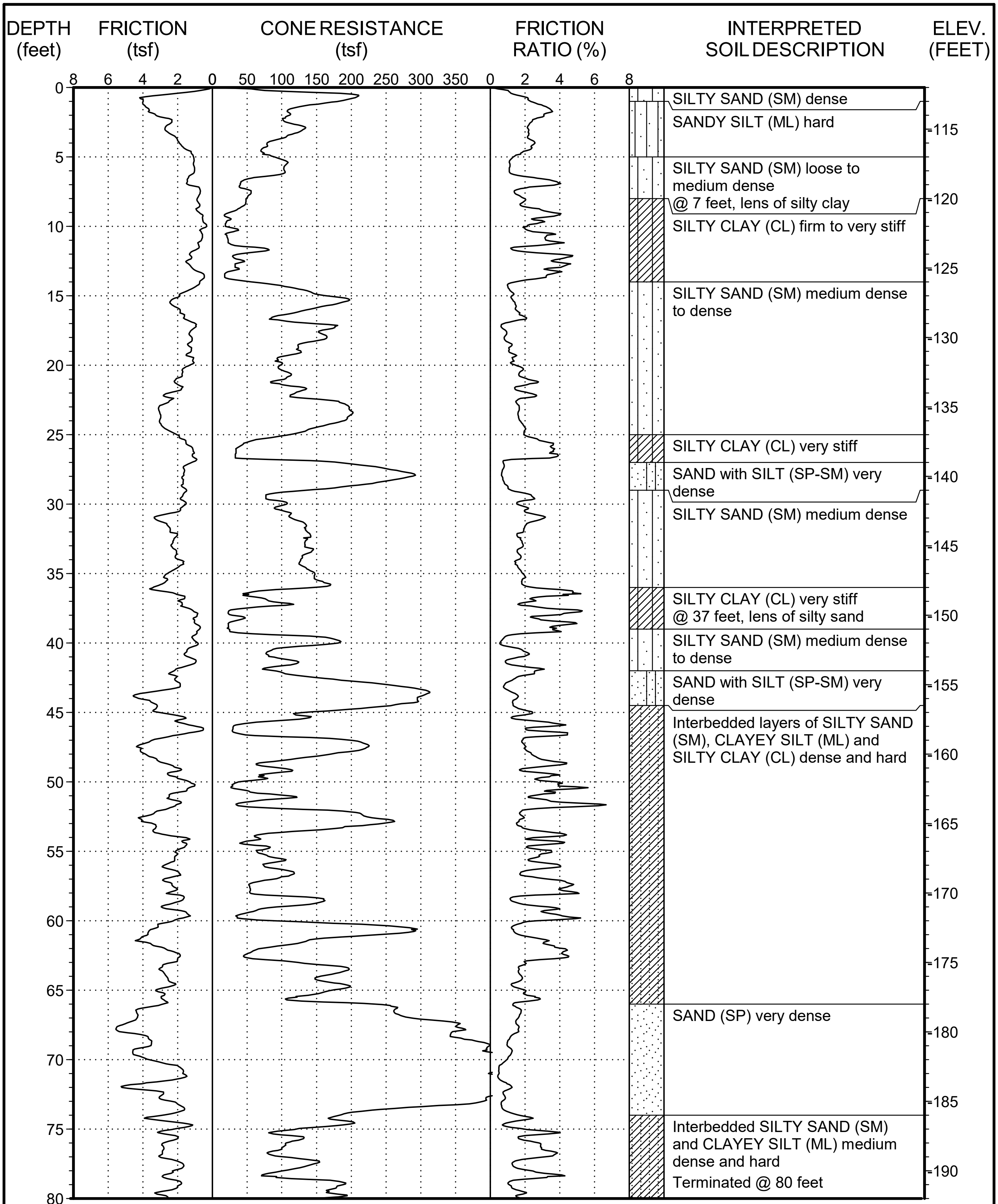
The CPT locations were laid out in the field by measuring from existing site features. Ground surface elevations at the CPT locations were estimated from topographic map dated July 5, 2018 by The Altum Group using a project datum and should be considered approximate. The project datum is 500 feet greater than actual MSL elevations to avoid negative elevations.



GEOTECHNICAL PROFESSIONALS, INC.

CONE PENETROMETER

FIGURE A-1



Date performed: 7-23-18

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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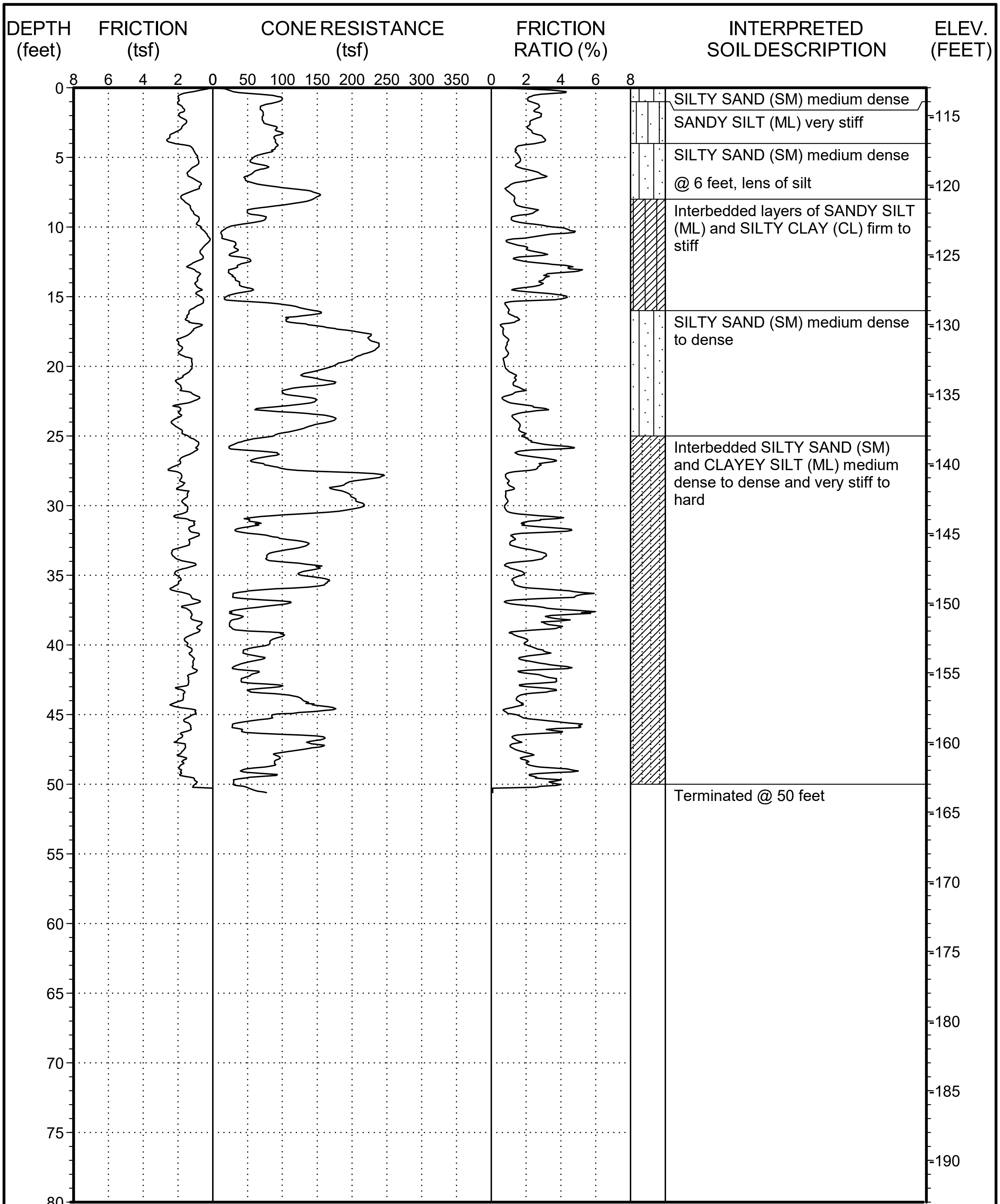
LOG OF CPT NO. C-1

FIGURE A-2



Date performed:7-23-18

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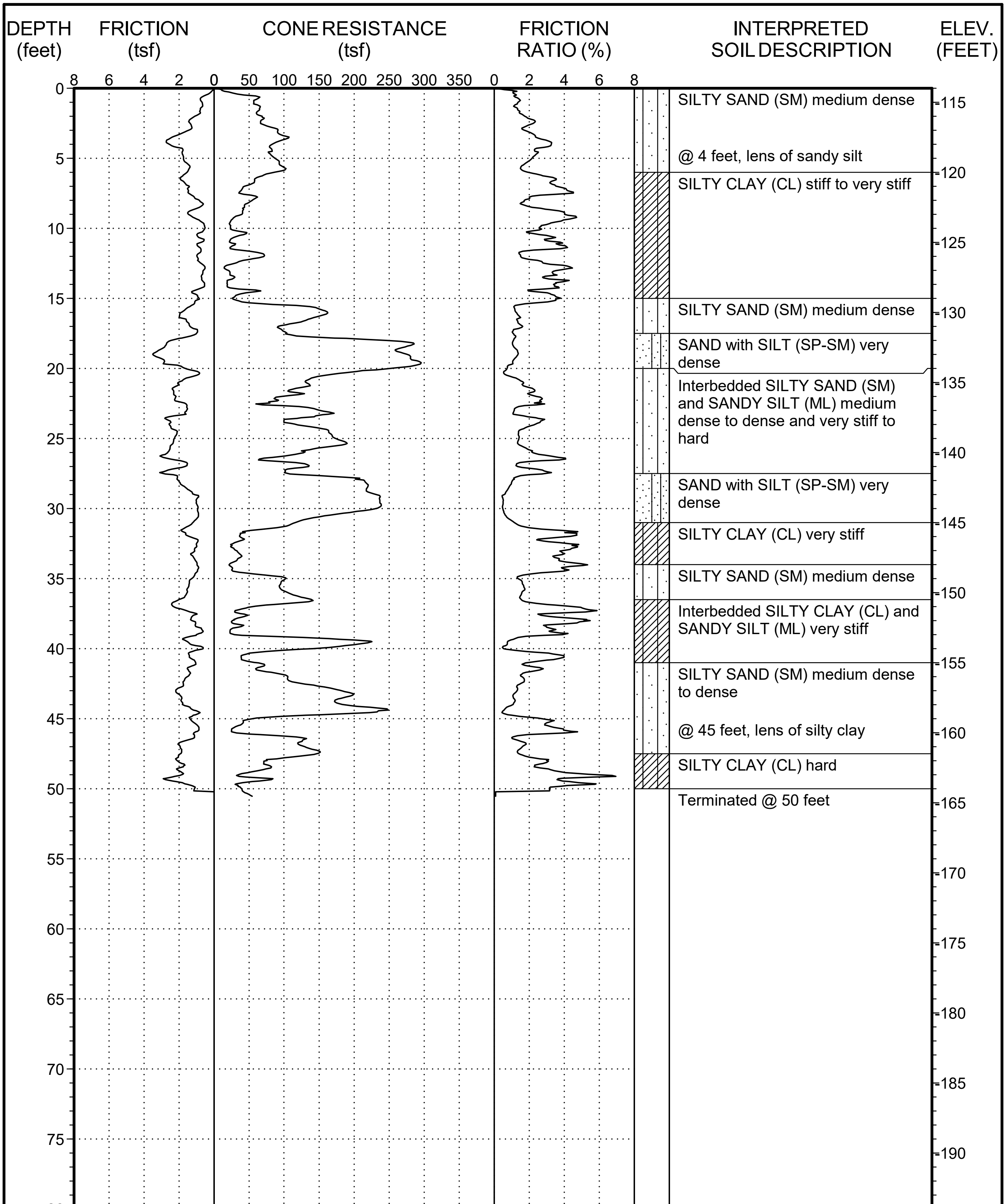
LOG OF CPT NO. C-2

FIGURE A-3



Date performed:7-20-18

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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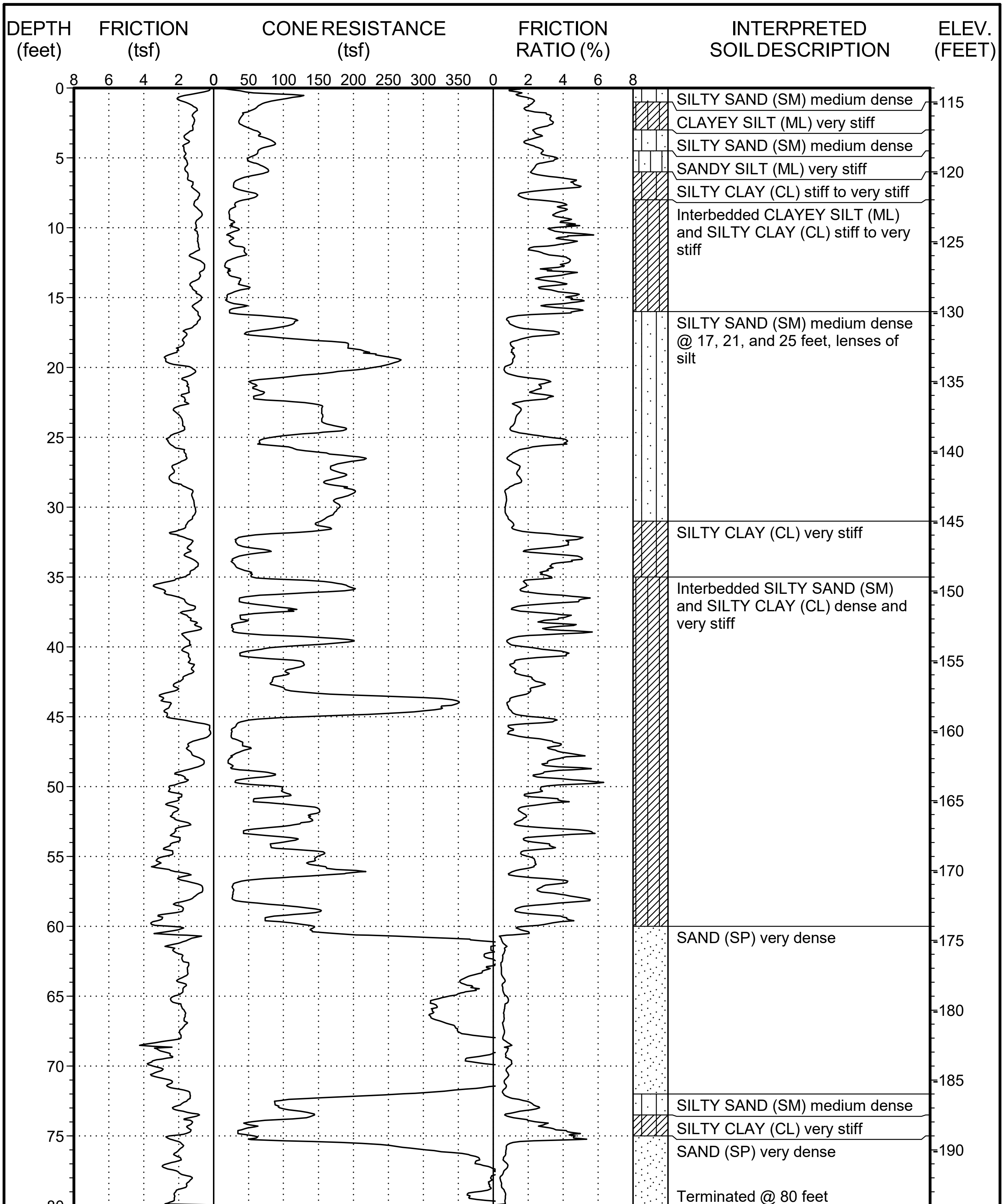
LOG OF CPT NO. C-3

FIGURE A-4



Date performed: 7-20-18

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

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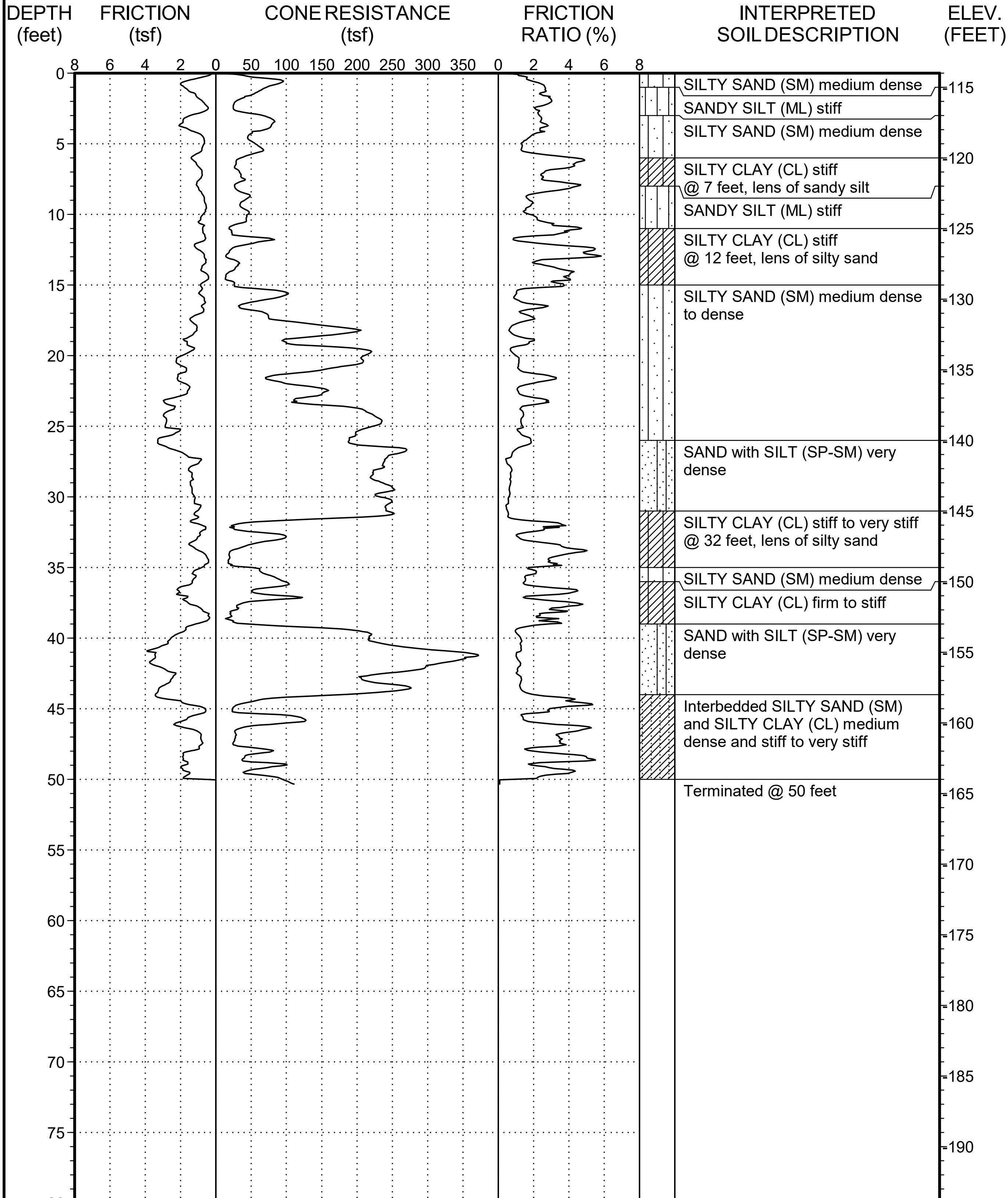
LOG OF CPT NO. C-4

FIGURE A-5



Date performed: 7-20-18

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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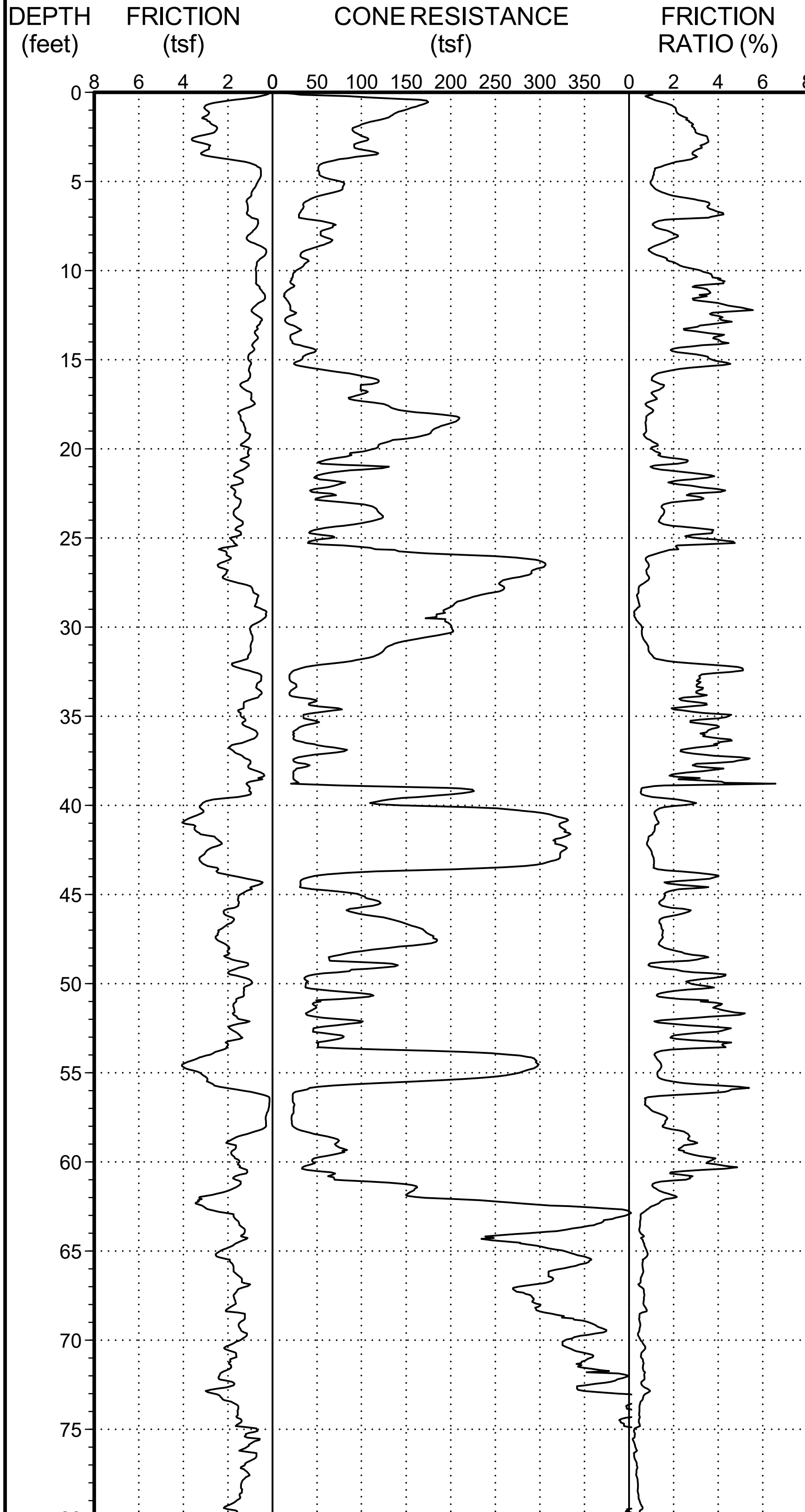
LOG OF CPT NO. C-5

FIGURE A-6



INTERPRETED SOIL DESCRIPTION

SILTY SAND (SM) dense

SANDY SILT (ML) hard

SILTY SAND (SM) medium dense

@ 6 feet, lens of silty clay

SILTY CLAY (CL) firm to stiff

SILTY SAND (SM) medium dense to dense

Interbedded SILTY SAND (SM) and CLAYEY SILT (ML) medium dense and very stiff

SAND (SP) dense to very dense

Interbedded CLAYEY SILT (ML) and SILTY CLAY (CL) stiff to very stiff

SAND (SP) very dense

SILTY CLAY (CL) stiff

SILTY SAND (SM) medium dense to dense

Interbedded SILTY SAND (SM) and CLAYEY SILT (ML) medium dense and very stiff

SAND (SP) very dense

SILTY CLAY (CL) hard

SILTY SAND (SM) to SANDY SILT (ML) loose and soft to firm

CLAYEY SILT (ML) very stiff

SAND (SP) very dense

Terminated @ 80 feet

ELEV. (FEET): 120, 125, 130, 135, 140, 145, 150, 155, 160, 165, 170, 175, 180, 185, 190, 195

Date performed: 7-23-18

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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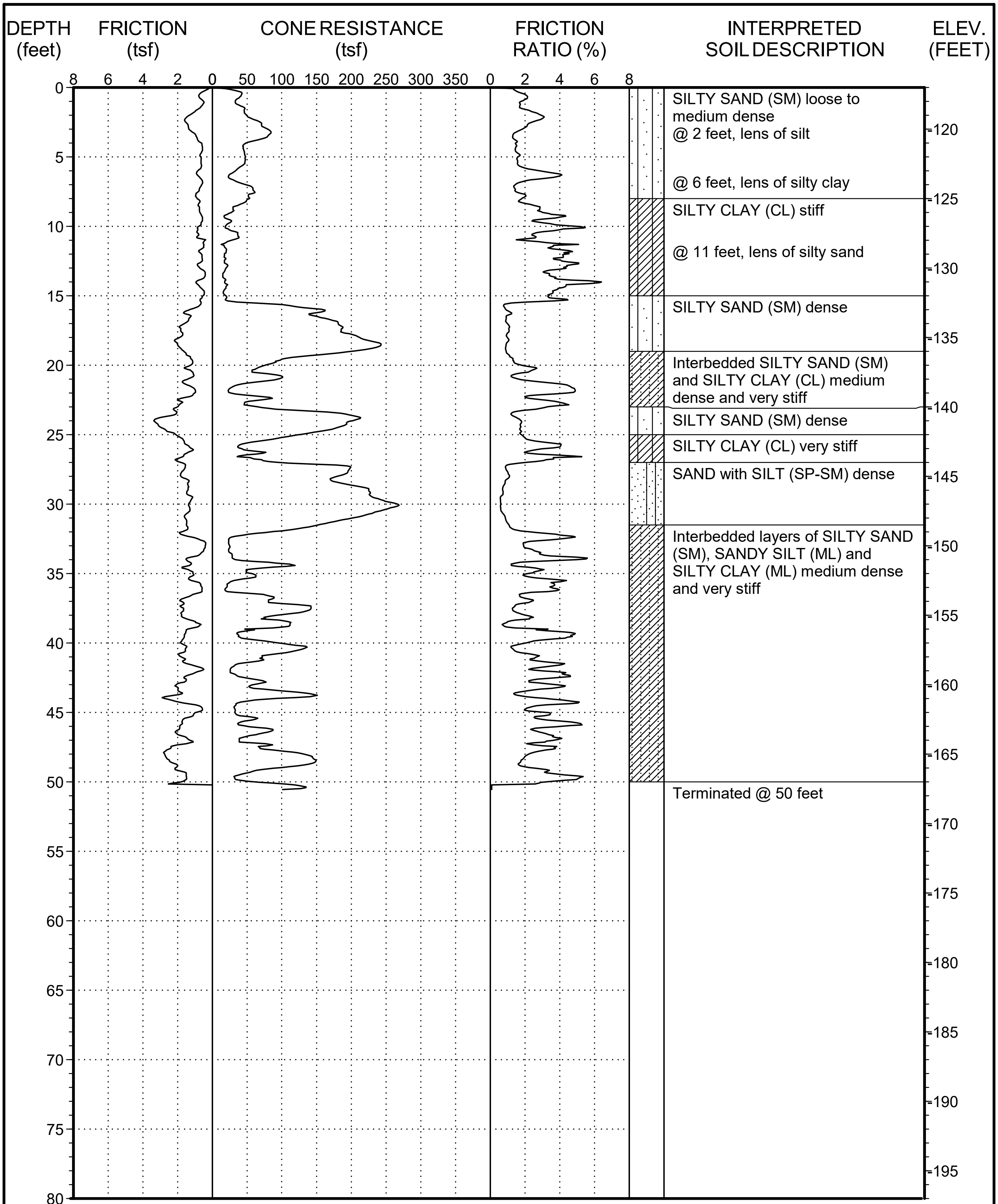
LOG OF CPT NO. C-6

FIGURE A-7



Date performed: 7-20-18

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

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PROJECT NO.: 2884.I

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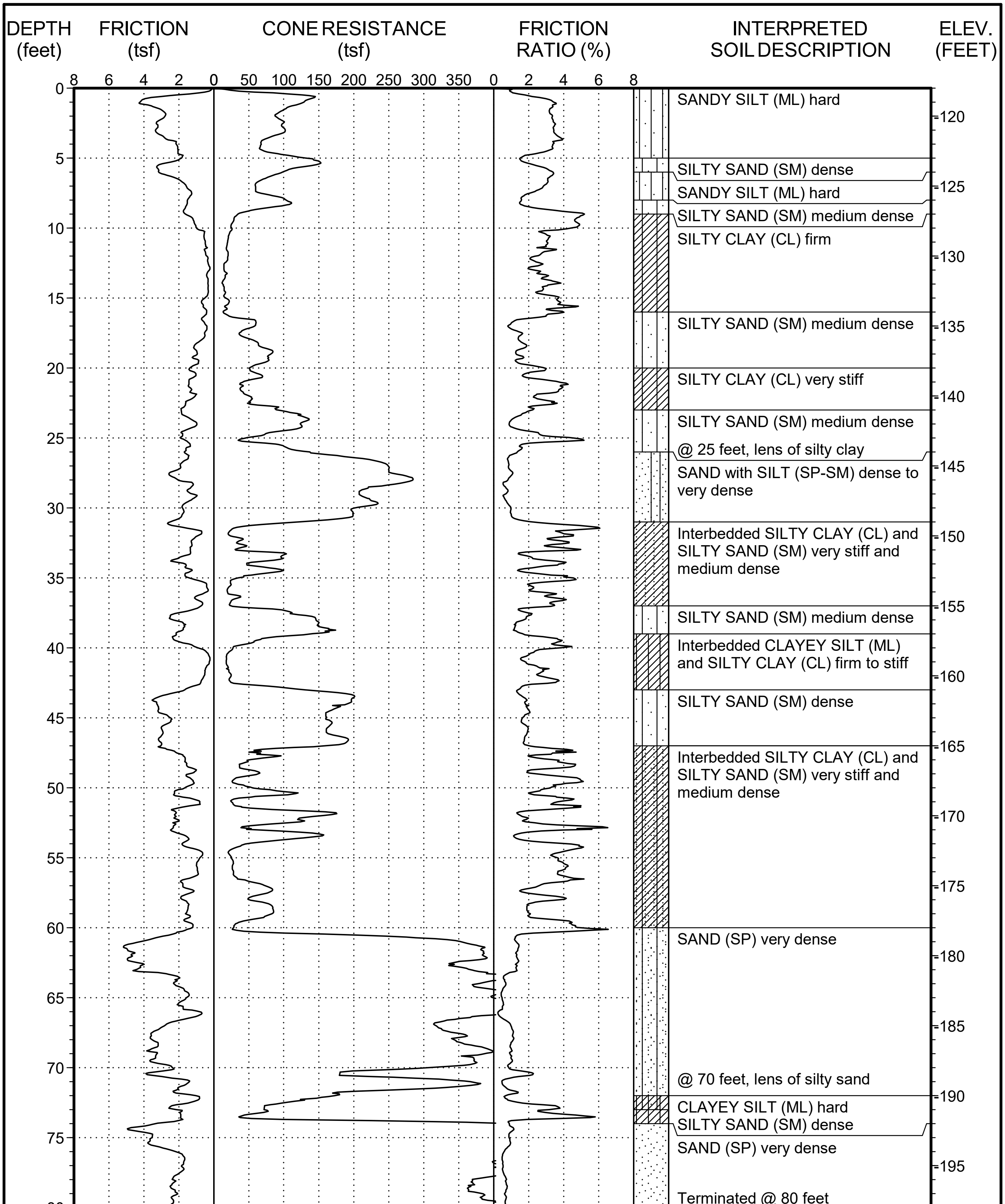
LOG OF CPT NO. C-7

FIGURE A-8



Date performed: 7-20-18

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.

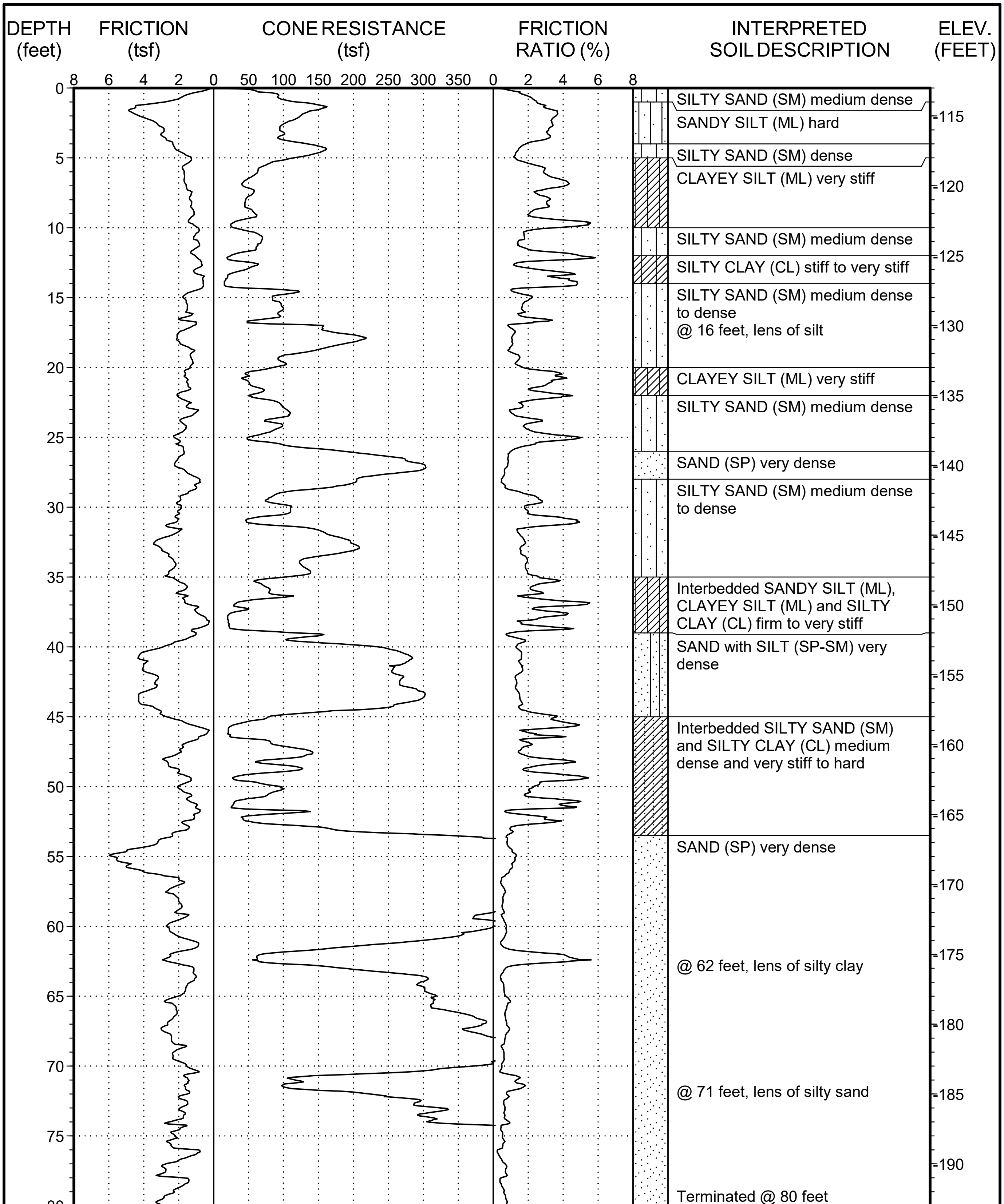
GPI

PROJECT NO.: 2884.I

COACHELLA

LOG OF CPT NO. C-8

FIGURE A-9



Date performed:7-23-18

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

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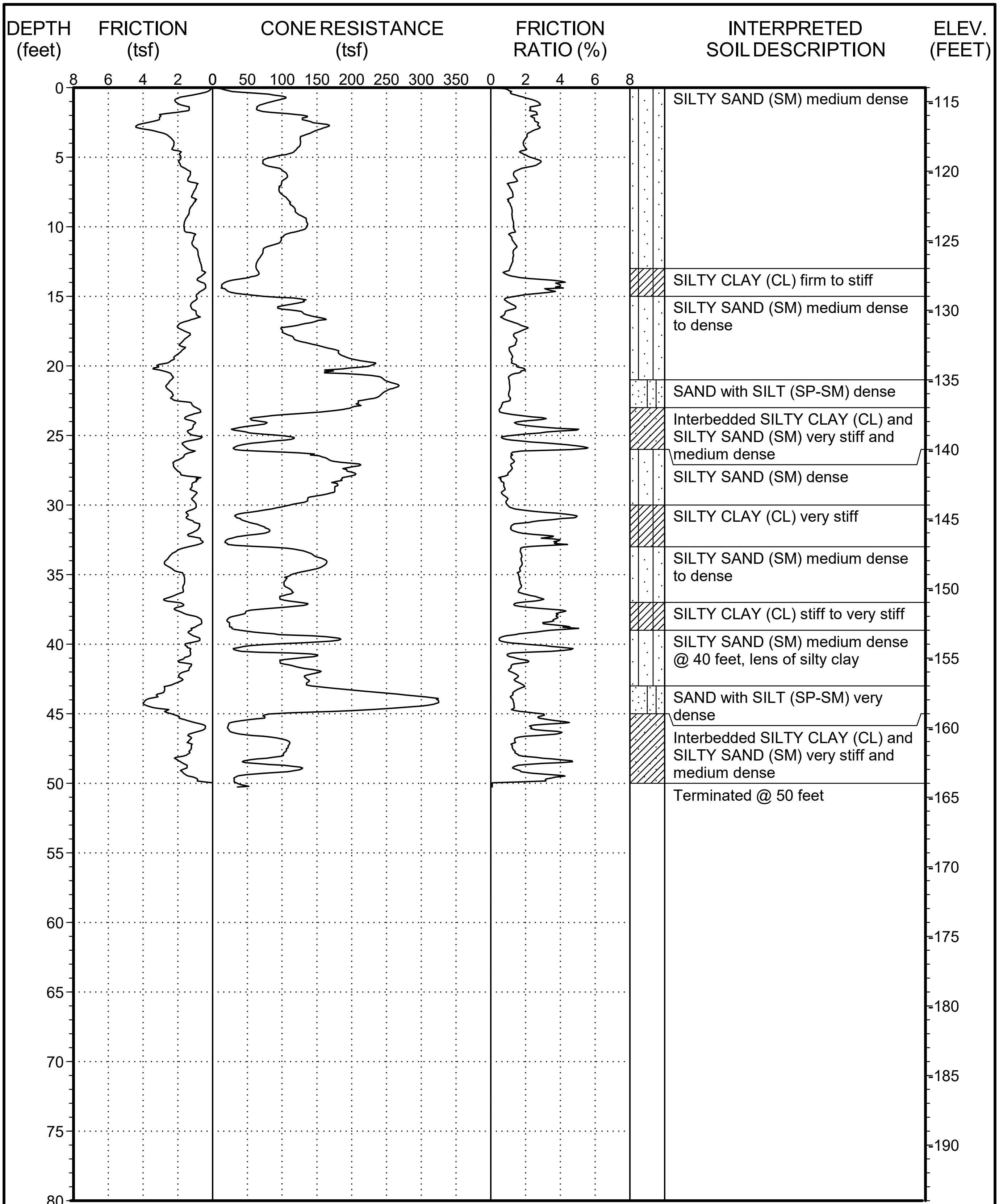
FIGURE A-10



Date performed:7-23-18

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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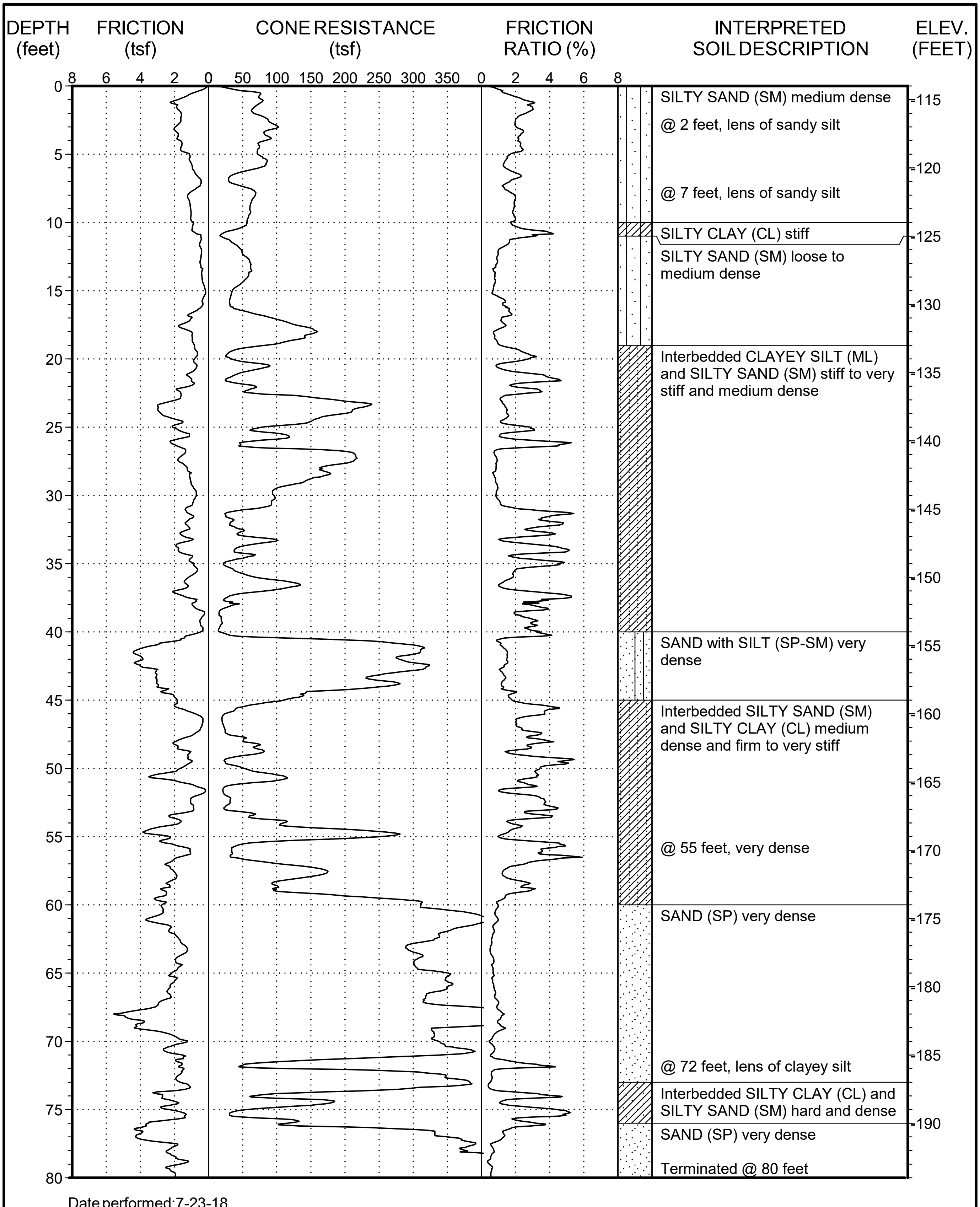
LOG OF CPT NO. C-10

FIGURE A-11



Date performed:7-23-18

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

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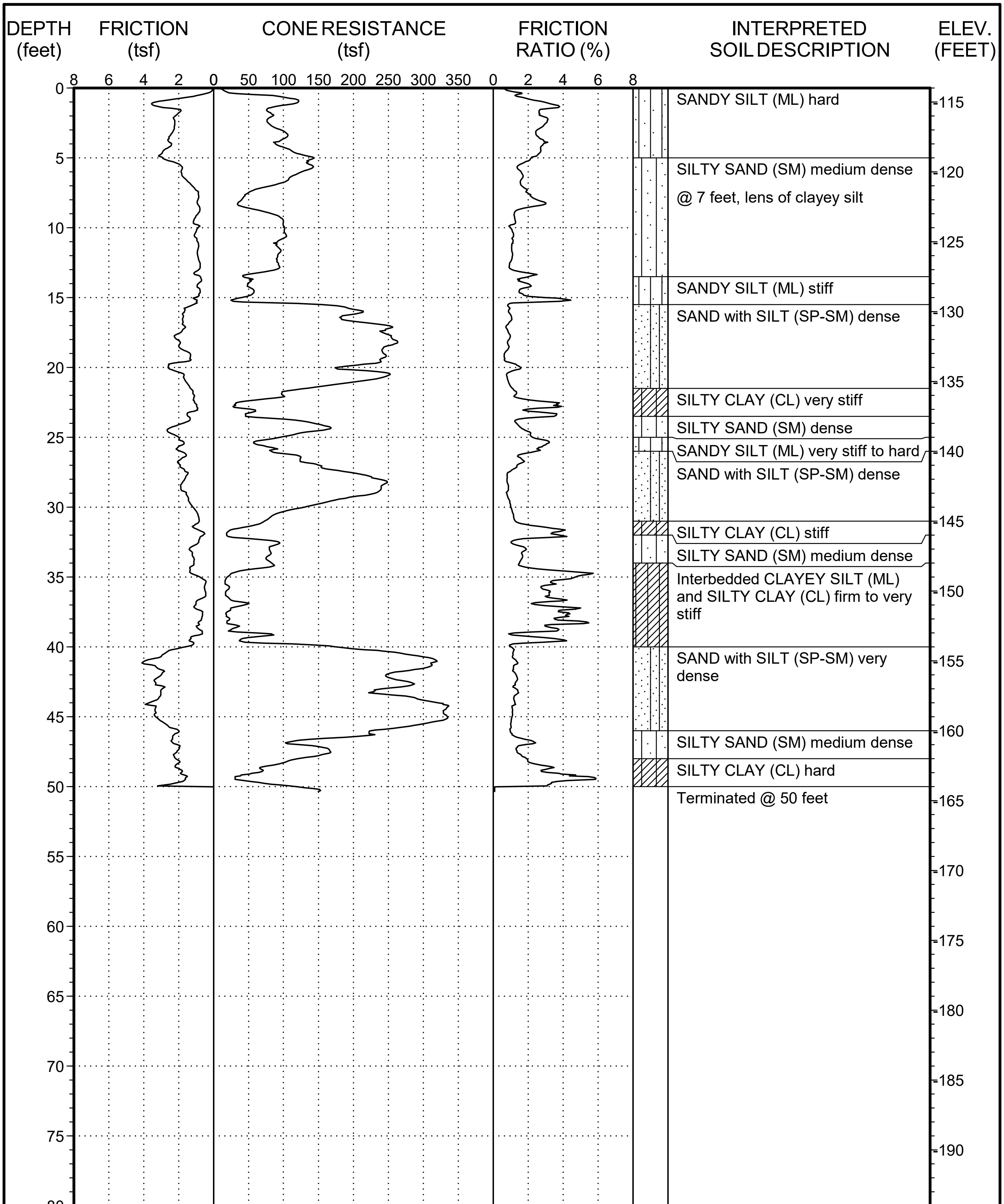
FIGURE A-12



Date performed:7-19-18

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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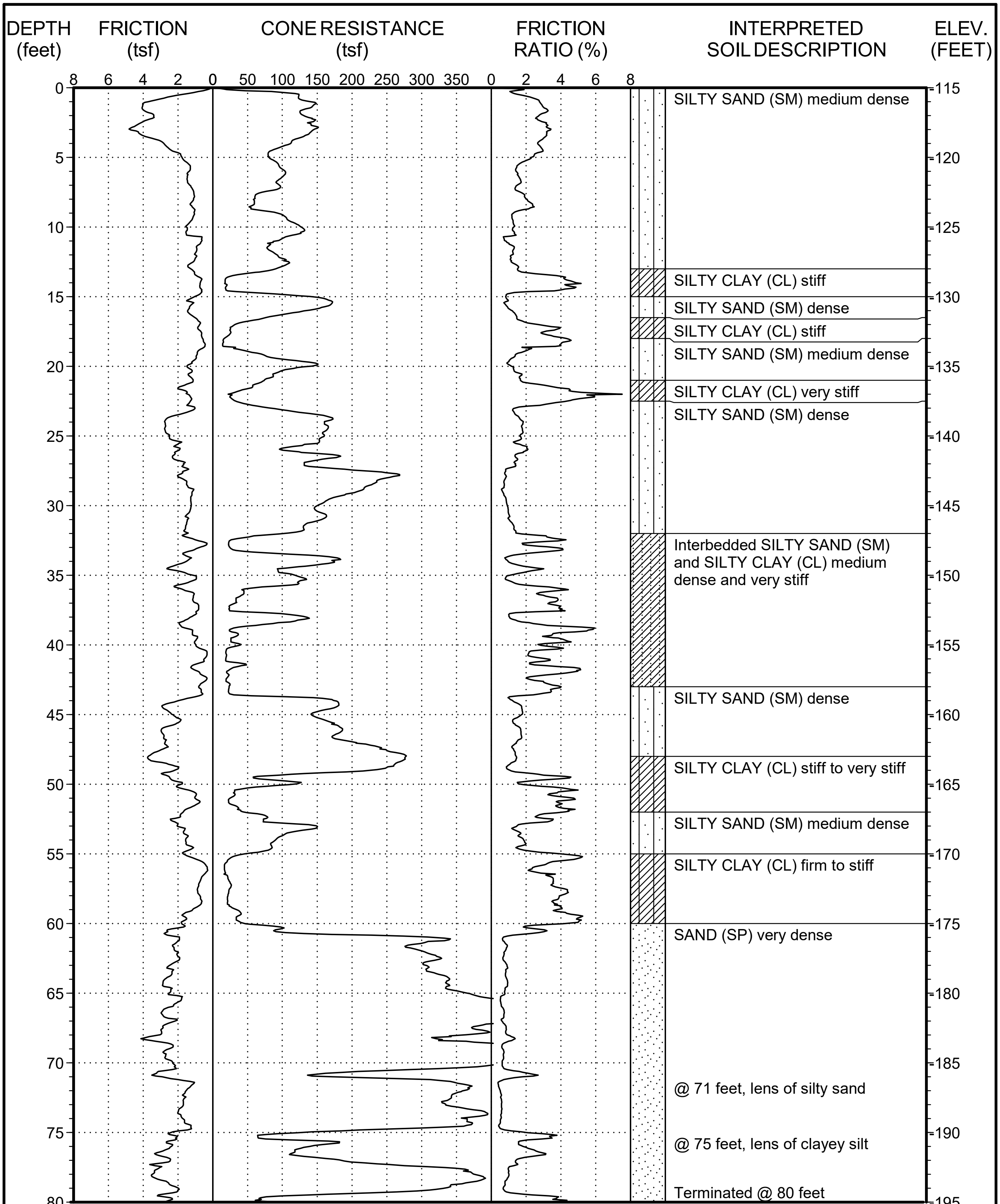
LOG OF CPT NO. C-12

FIGURE A-13



Date performed:7-19-18

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

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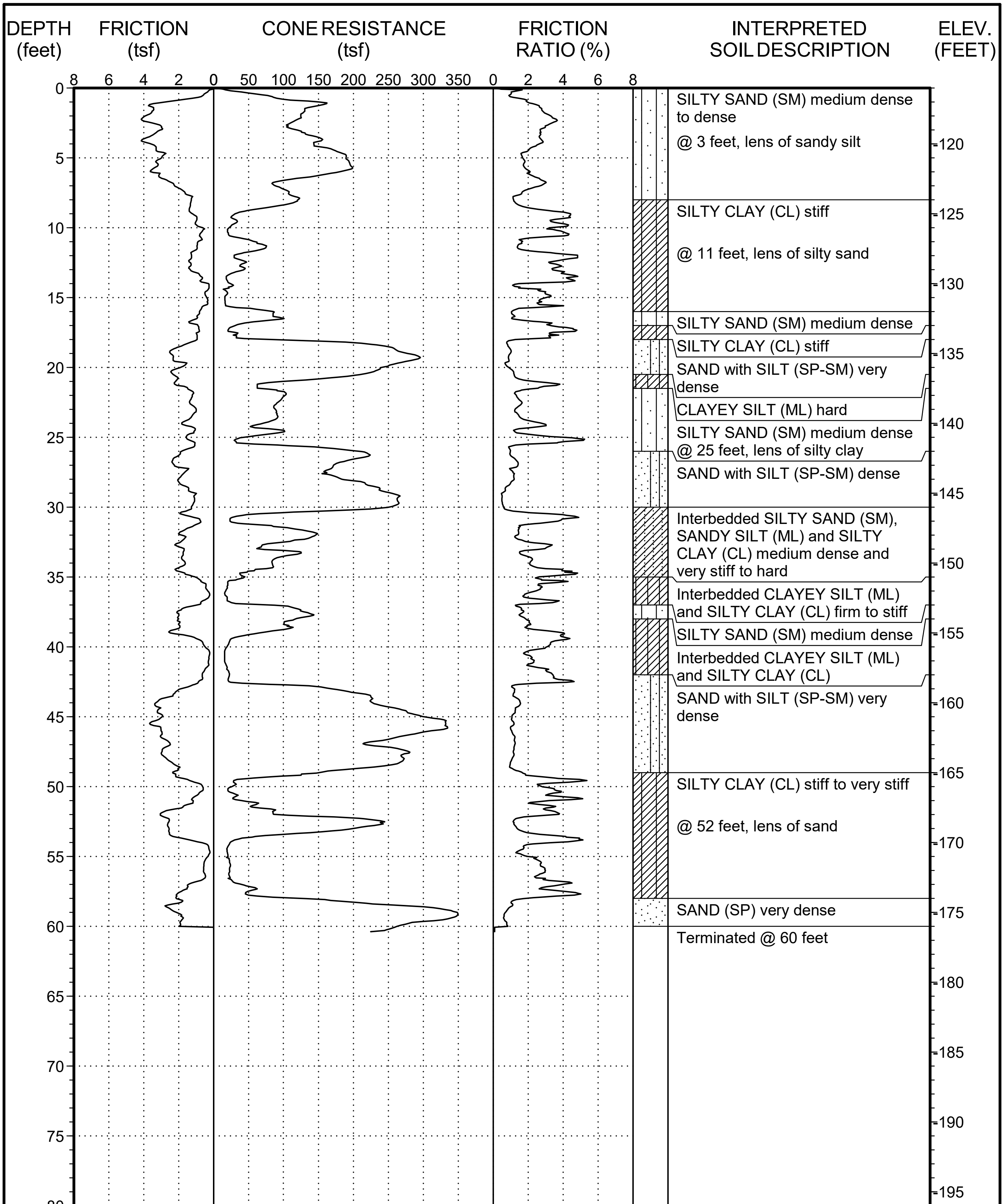
FIGURE A-14



Date performed: 7-19-18

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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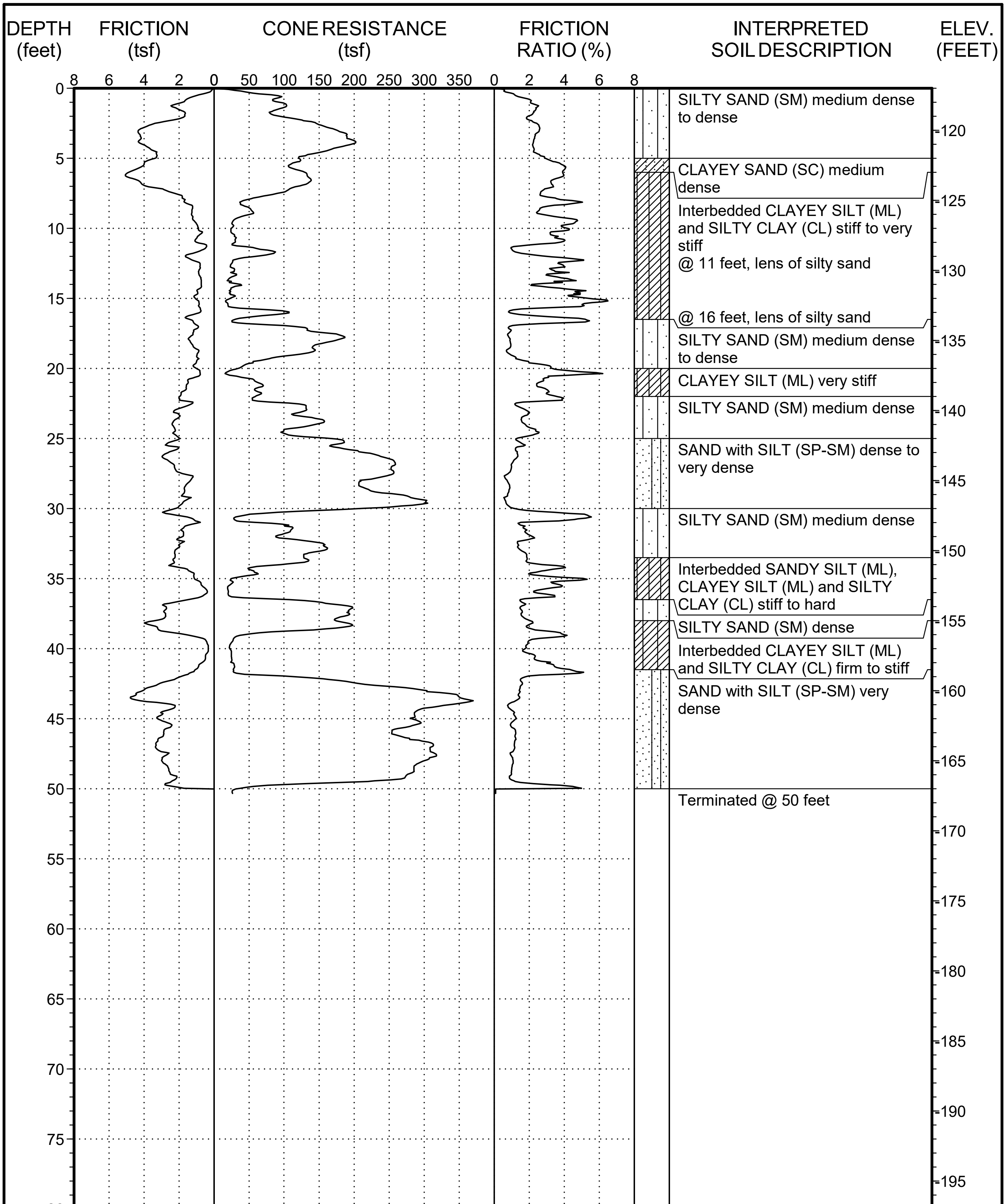
LOG OF CPT NO. C-14

FIGURE A-15



Date performed:7-19-18

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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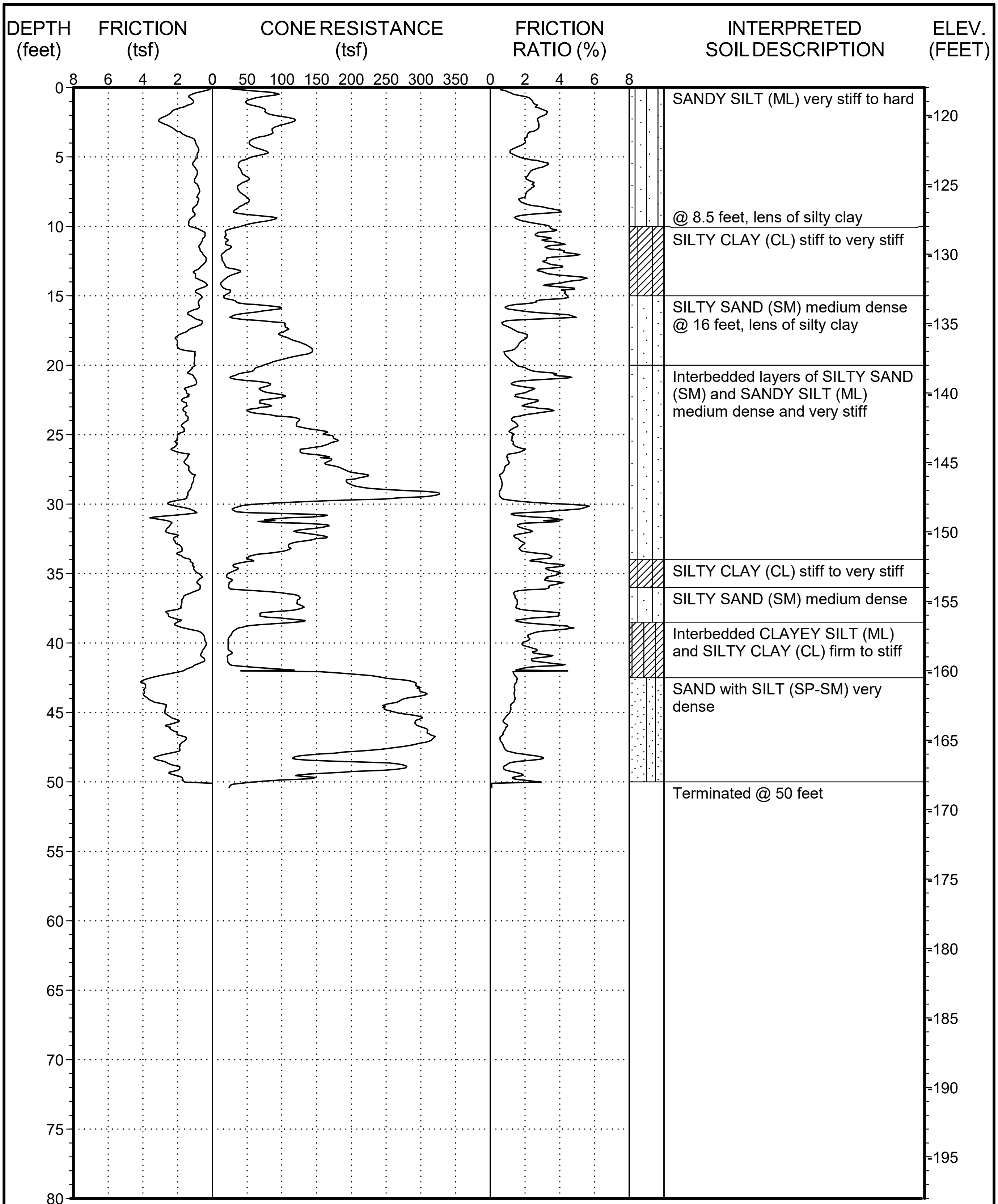
LOG OF CPT NO. C-15

FIGURE A-16



Date performed: 7-19-18

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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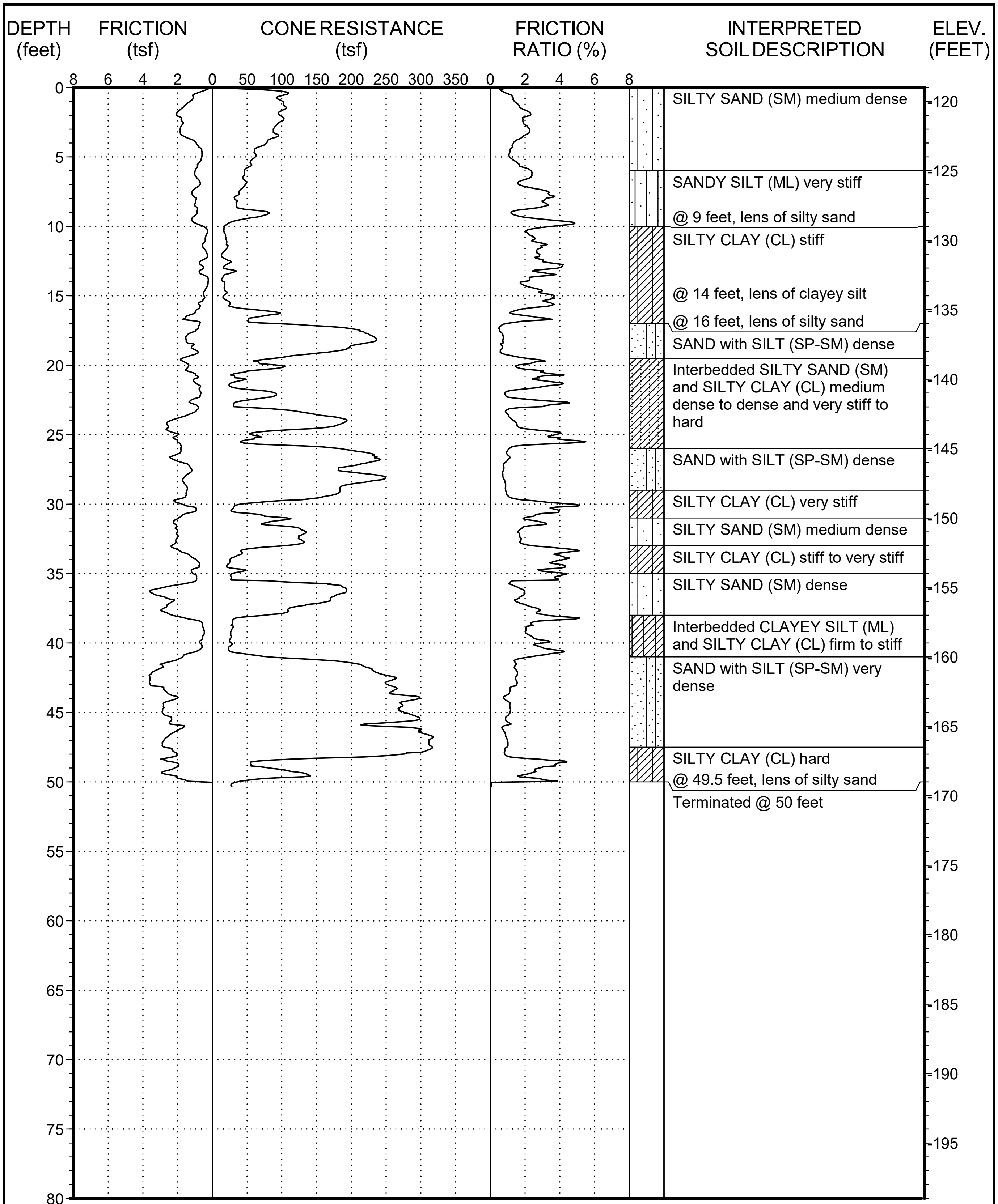
LOG OF CPT NO. C-16

FIGURE A-17



Date performed: 7-19-18

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

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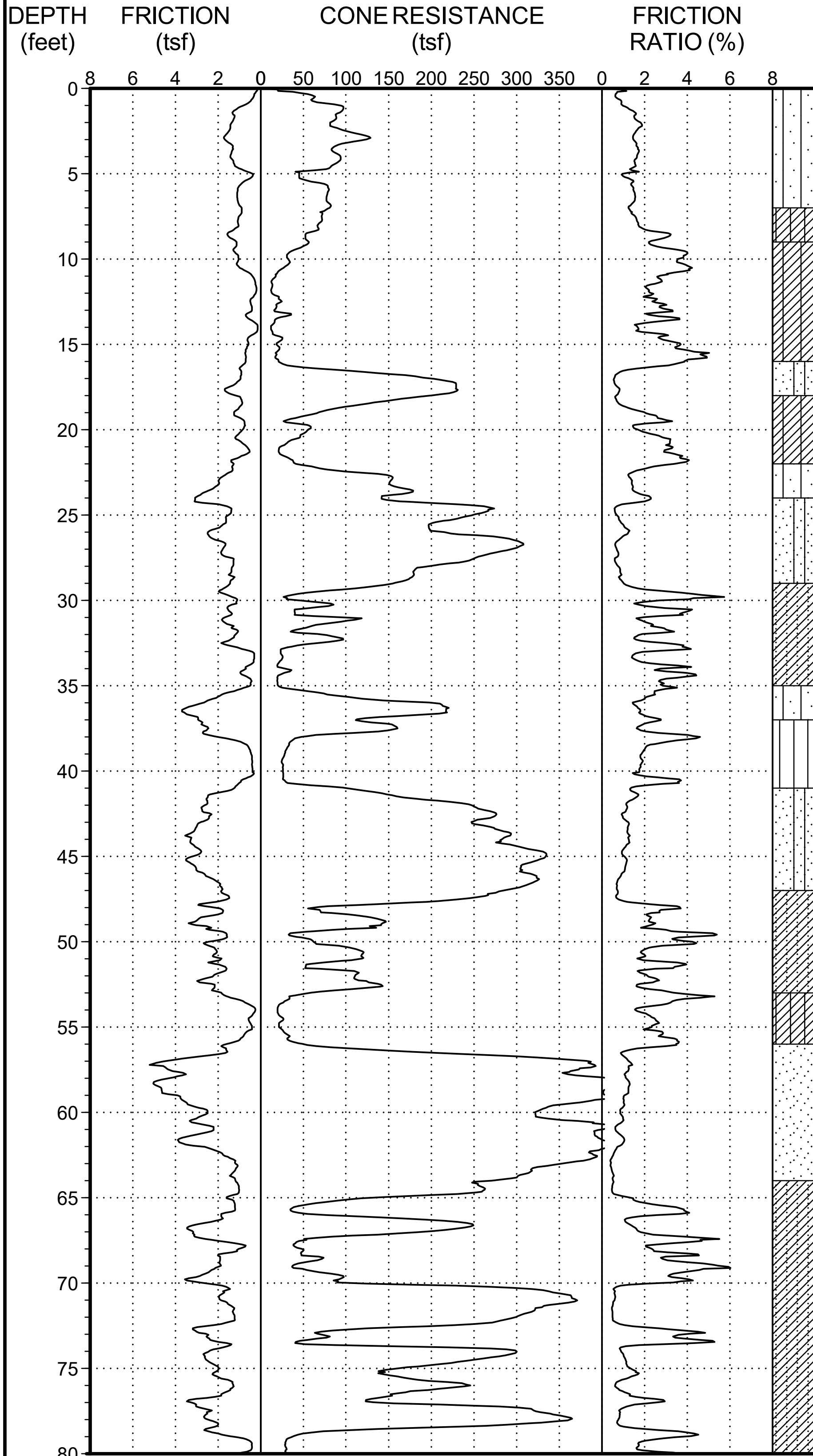
FIGURE A-18



Depth (feet)	INTERPRETED SOIL DESCRIPTION
0–7	SILTY SAND (SM) medium dense
7–9	CLAYEY SILT (ML) very stiff
9–16	Interbedded CLAYEY SILT (ML) to SILTY CLAY (CL) firm to stiff
16–18	SAND with SILT (SP-SM) dense
18–22	SILTY CLAY (CL) stiff to very stiff @ 20 feet, lens of silty sand
22–24	SILTY SAND (SM) dense
24–29	SAND with SILT (SP-SM) dense
29–35	Interbedded SILTY SAND (SM), CLAYEY SILT (ML) and SILTY CLAY (CL) medium dense and stiff to very stiff
35–37	SILTY SAND (SM) dense
37–41	SILT (ML) stiff @ 37.5 and 40 feet, lens of silty clay
41–47	SAND with SILT (SP-SM) very dense
47–53	Interbedded SILTY SAND (SM) and SILTY CLAY (CL) medium dense and hard
53–56	CLAYEY SILT (ML) firm to stiff
56–64	SAND (SP) very dense
64–80	Interbedded SAND (SP), SILTY SAND (SM) and SILTY CLAY (CL) very dense and very stiff to hard
	Terminated @ 80 feet

ELEV. (FEET)
-120 -125 -130 -135 -140 -145 -150 -155 -160 -165 -170 -175 -180 -185 -190 -195

Date performed: 7-20-18

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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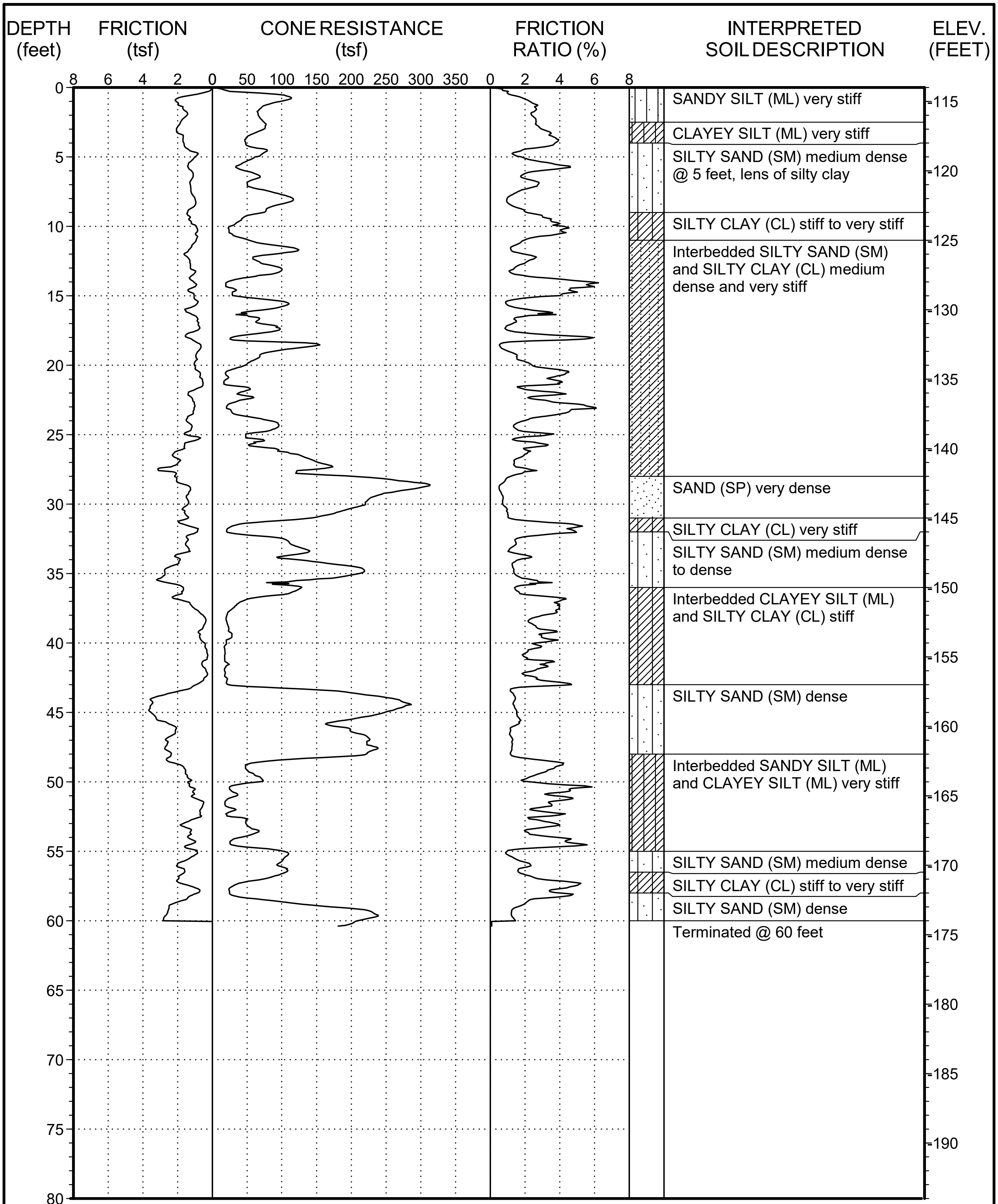
LOG OF CPT NO. C-18

FIGURE A-19



Date performed: 7-19-18

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

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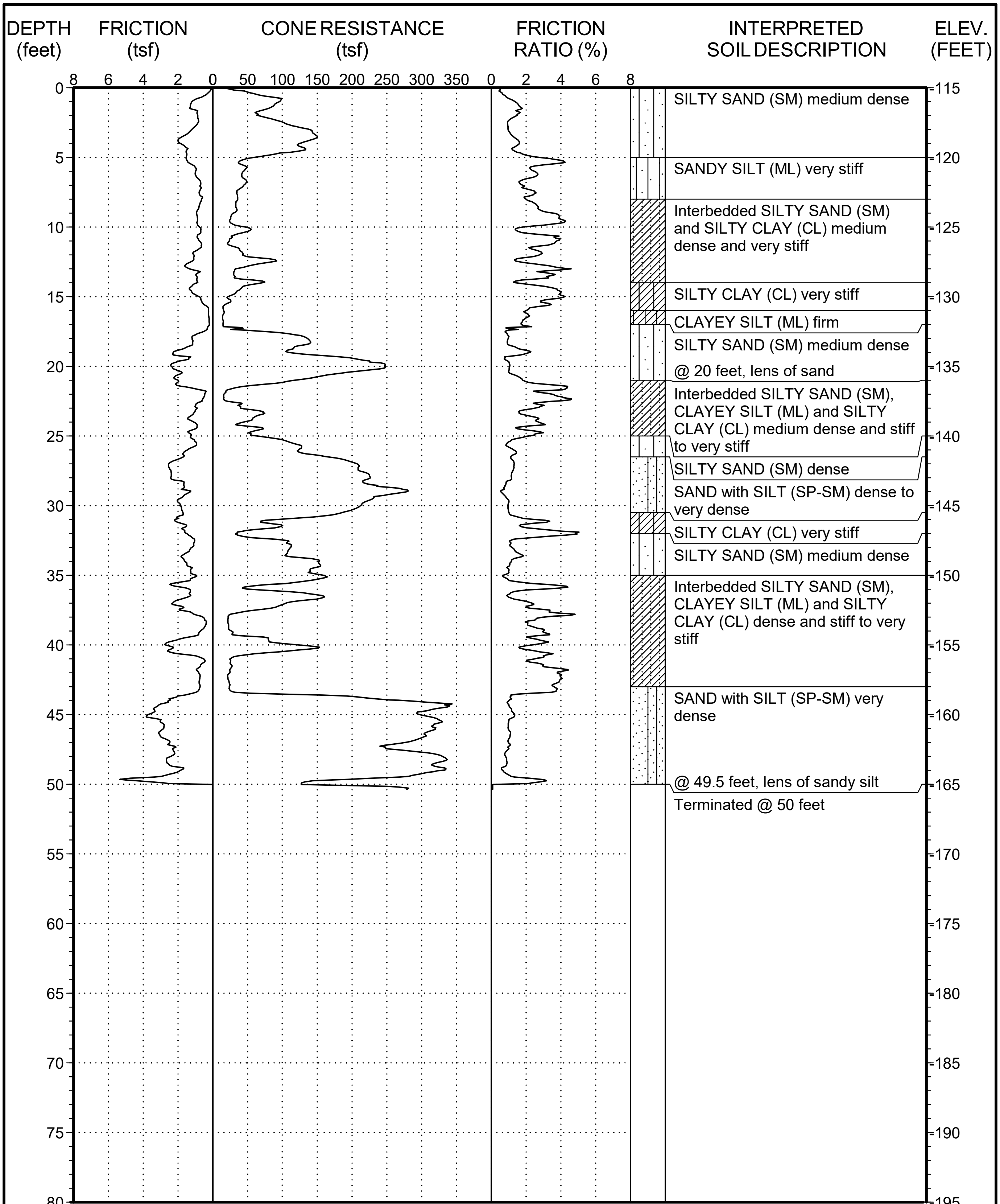
FIGURE A-20



Date performed:7-23-18

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

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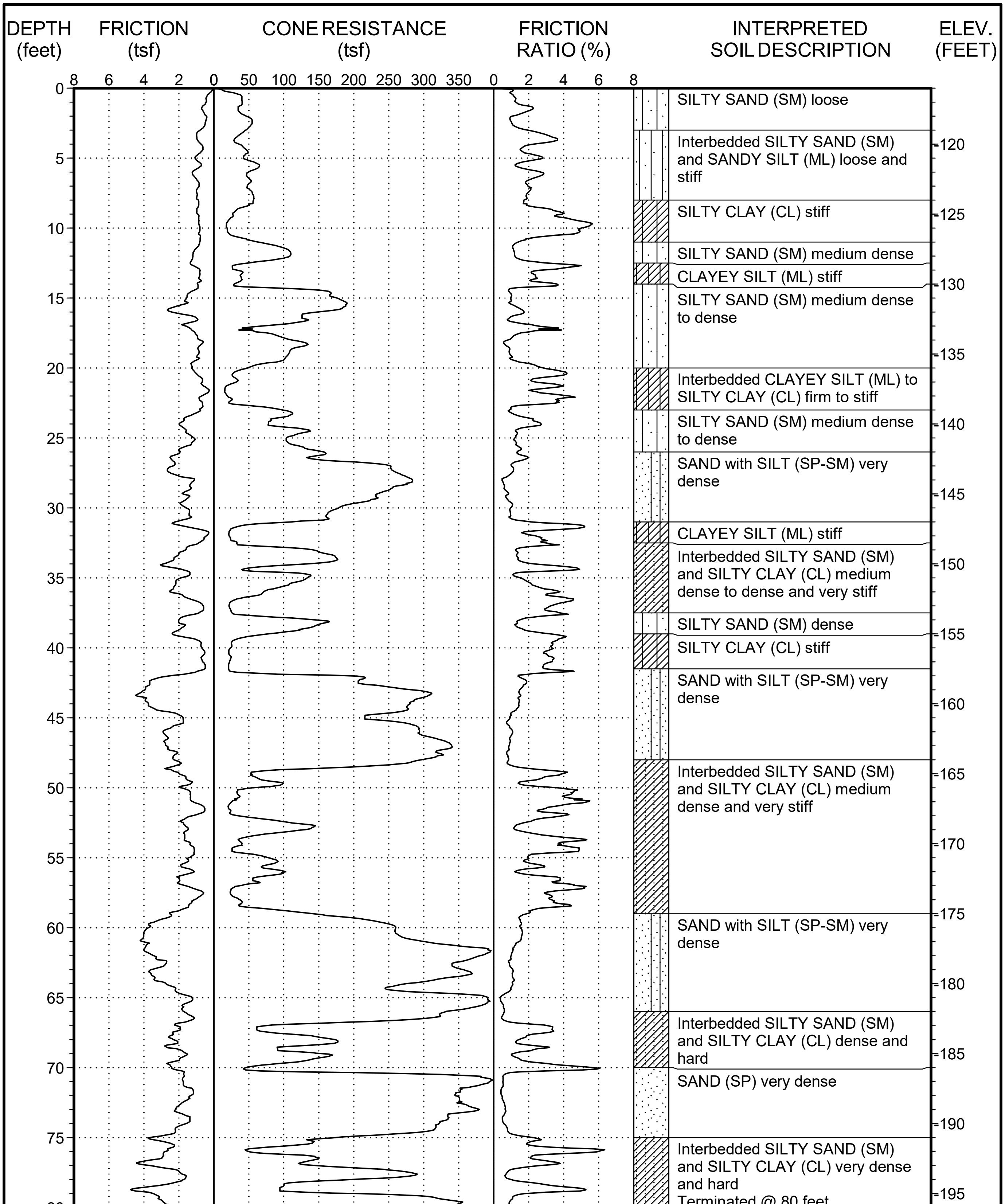
FIGURE A-21



Date performed:7-20-18

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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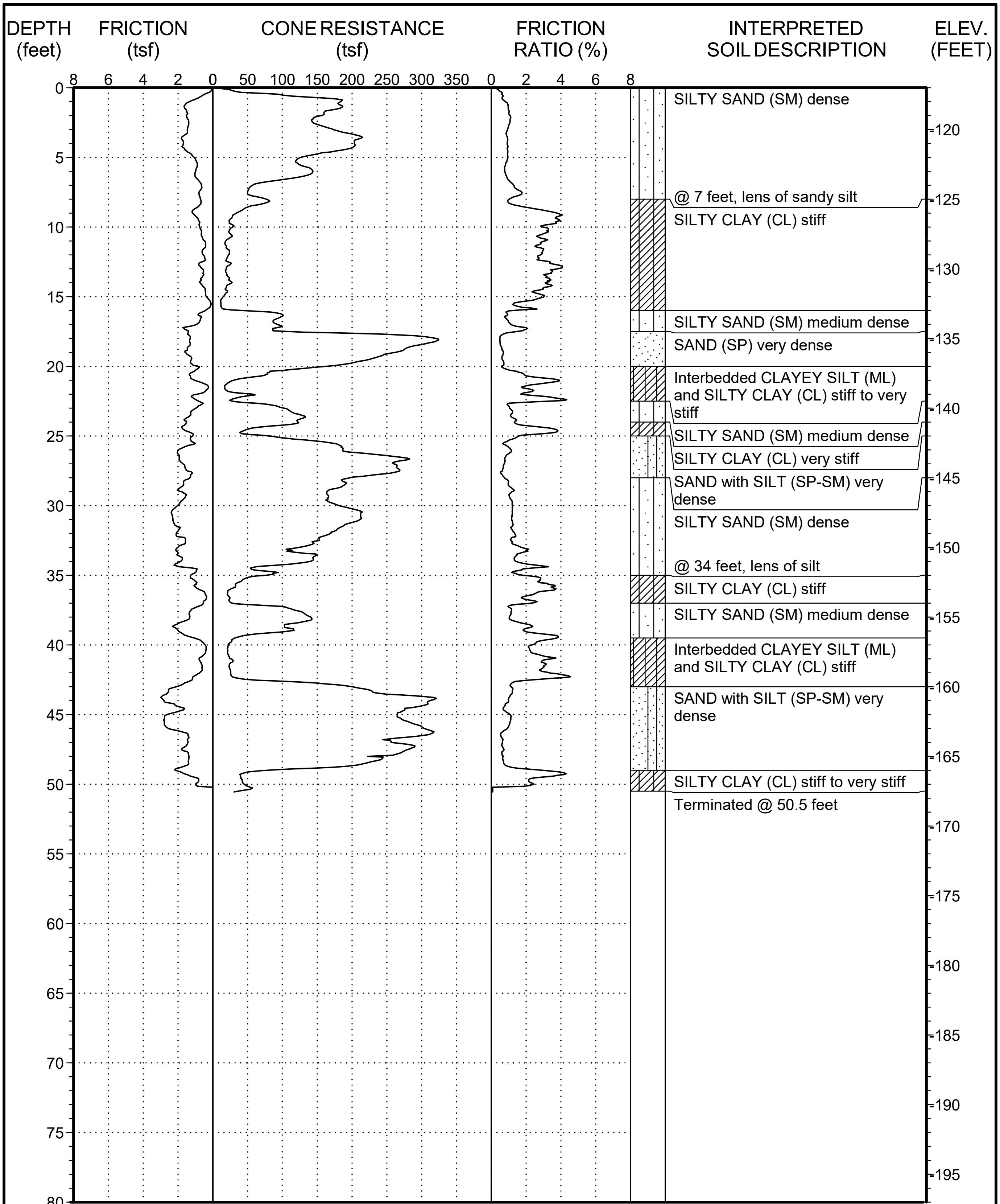
LOG OF CPT NO. C-21

FIGURE A-22



Date performed:7-24-18

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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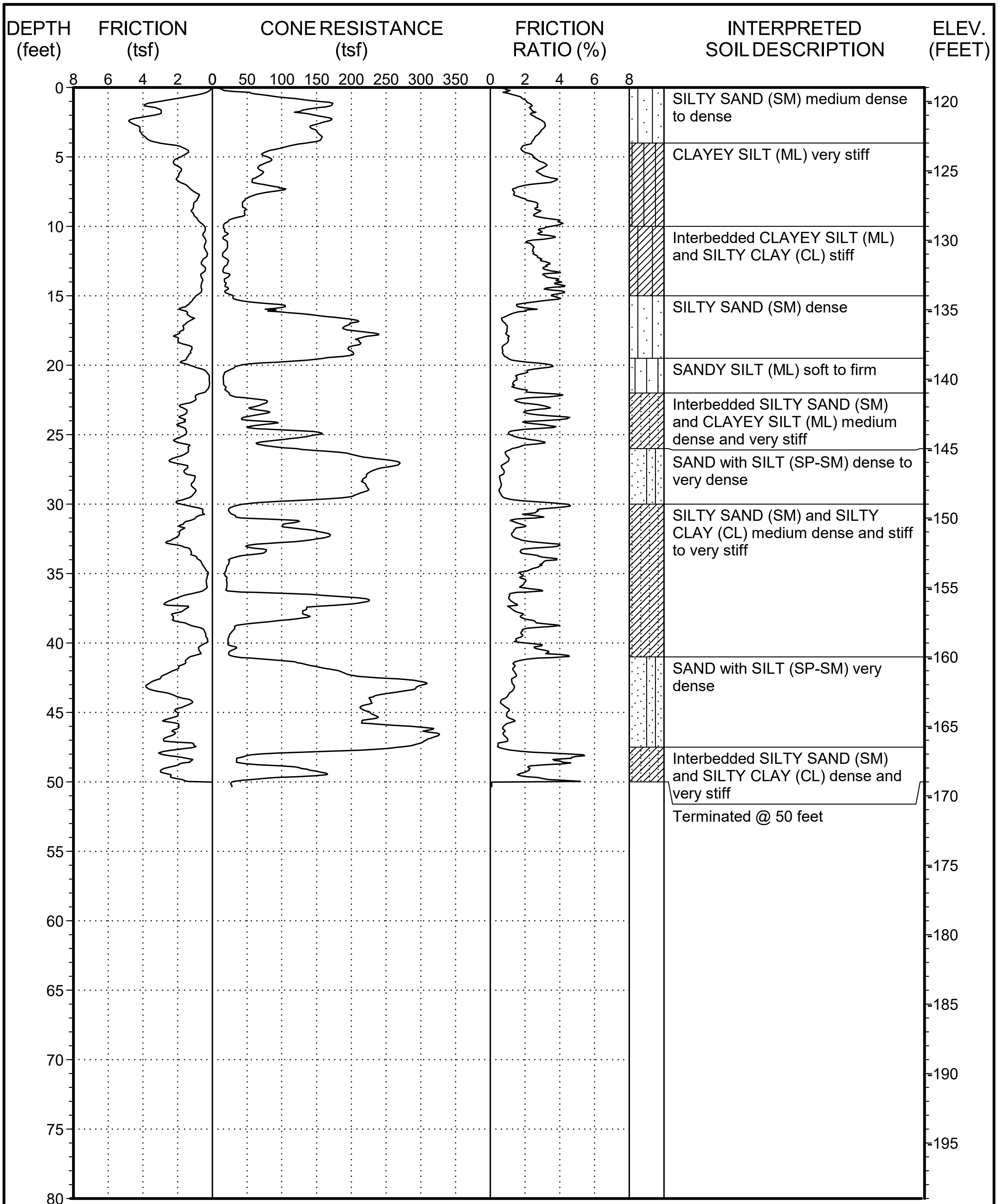
LOG OF CPT NO. C-22

FIGURE A-23



Date performed:7-23-18

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-24

FIGURE A-24

APPENDIX B

APPENDIX B

EXPLORATORY BORINGS

The subsurface conditions at the site were investigated by drilling and sampling eleven exploratory borings. The borings were advanced to depths of 6 to 81½ 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.

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

The field explorations for the investigation were performed under the continuous technical supervision of GPI's representative, who visually inspected the site, maintained detailed logs of the borings, classified the soils encountered, and obtained relatively undisturbed samples for examination and laboratory testing. The soils encountered in the borings were classified in the field and through further examination in the laboratory in accordance with the Unified Soils Classification System. Detailed logs of the borings are presented in Figures B-1 to B-11 in this appendix.

The boring locations were laid out in the field by measuring from existing site features. Ground surface elevations at the boring locations were estimated from topographic map dated July 5, 2018 by The Altum Group using a project datum and should be considered approximate. The project datum is 500 feet greater than actual MSL elevations to avoid negative elevations.

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
			B	0	Natural: **SILTY SAND (SM)** light brown, dry, loose	
2.2	94	14	D			-115
8.6	105	15	D	5	**SILT (ML)** brown, slightly moist, stiff	
17.1	94	11	D		@ 7 feet, very moist, firm	-120
15.2	99	11	D	10	**SANDY SILT (ML)** grey, very moist, firm	
					CLAY (CL) grey, moist, firm	
					Total Depth 11 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: 7-25-18

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): Not Encountered

GPI

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COACHELLA

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)
					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	Natural: **SANDY SILT (ML)** light brown, dry to slightly moist, firm	
9.7	89	12	D		**SILT (ML)** light brown, slightly moist, firm	-120
6.7	92	10	D	5	**SILTY SAND (SM)** light brown, slightly moist, loose	
					Total Depth 6 feet	

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

DATE DRILLED:
7-25-18

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

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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 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	Natural: **SANDY SILT (ML)** light brown, very dry	
	5.4	84	14	D		**SANDY SILT (ML)** light brown, dry, stiff	-120
						SILT (ML) light brown / grey, dry to slightly moist, stiff, with gravel	
	32.0	88	15	D	5	**SANDY SILT (ML)** light brown, wet, stiff	-125
	4.0	101	7	D	10	**SILTY CLAY (CL)** light brown, dry, firm	
						CLAY (CL) light brown, dry, firm	
						Total Depth 11 feet	

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

DATE DRILLED:
7-25-18

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

PROJECT NO.: 2884.I

COACHELLA

LOG OF BORING NO. B-3

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					0	Natural: **SANDY SILT (ML)** light brown, dry	-115
	0.4	92	30	D		**SILTY SAND (SM)** light brown, dry, medium dense	
	2.0	95	18	D	5		-120
						Total Depth 6 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: 7-23-18

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): Not Encountered

GPI

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COACHELLA

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)
				B	0	Natural: **SANDY SILT (ML)** light brown, dry	
	2.3	95	16	D		@ 2 feet, stiff	-120
	14.8	88	16	D	5	**SILT (ML)** brown, moist, stiff	
						Total Depth 6 feet	

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

DATE DRILLED:
7-25-18

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

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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	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	Natural: **SANDY SILT (ML)** light brown, dry	-120
	2.0	85	16	D		**SILT (ML)** light brown, dry, stiff	
	2.8	88	15	D	5		-125
	9.5		12	S		@ 7 feet, dry to slightly moist	
	23.5	93	12	D	10	**CLAYEY SILT (ML)** brown, wet, stiff	-130
	33.1	86	6	D	15	**SILTY SAND (SM)** brown, wet, loose	
						CLAYEY SILT (ML) brown, wet, firm, trace sand	-135
	24.7	95	14	D	20	**SANDY SILT (ML)** grey brown, wet, stiff	-140
	21.2		19	S	25	**SILTY SAND (SM)** grey brown, wet, medium dense	-145
					30		-150
	39.2	82	16	D		**SILT (ML)** grey, wet, stiff, trace sand	
	35.9	87	8	D	35	**CLAYEY SILT (ML)** brown grey, wet, firm	-155

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

DATE DRILLED:
7-23-18

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
14

GPI

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LOG OF BORING NO. B-6

FIGURE 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)
	35.1	85	9	D	40	**SILTY CLAY (CL)** grey brown, wet, firm, with shells	-160
	22.8		34	S	45	**SAND (SP)** grey, wet, dense, trace silt	-165
	25.3	96	28	D	50	**SANDY SILT (ML)** grey, wet, very stiff, with clay lenses	-170
	22.2	105	15	D	55	**SILT (ML)** grey and brown, wet, stiff, with porosity, trace sand	-175
	21.5		52	S	60	**SILTY SAND (SM)** grey, wet, very dense	-180
					65		-185
	19.8		60	S	70		-190
					75		-195

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

DATE DRILLED:
7-23-18

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
14

PROJECT NO.: 2884.I

COACHELLA

LOG OF BORING NO. B-6

FIGURE 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)
28.3		65	S	80	**SANDY SILT (ML)** grey, wet, hard	-200
					Total Depth 81.5 feet	

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

DATE DRILLED:
7-23-18

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
14

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COACHELLA

LOG OF BORING NO. B-6

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	Natural: **SANDY SILT (ML)** light brown, dry	
2.1	81	8	D		**SILT (ML)** light brown, dry, firm, trace sand	
						-120
1.4	93	16	D	5	@ 5 feet, stiff	
14.1	92	9	D		**SILTY SAND (SM)** light brown, very moist, loose	
						-125
8.1		10	S	10	@ 10 feet, moist, medium dense	
						-130
21.4	101	25	D	15	**SAND (SP)** grey, wet, medium dense, trace silt	
						-135
20.2		11	S	20	**SAND with SILT (SP-SM)** grey, wet, medium dense	
					SAND (SP) grey, wet, medium dense, trace silt	
						-140
28.2	95	18	D	25	**SILT (ML)** grey, wet, stiff, trace sand and shells	
						-145
17.5		15	S	30	**SAND with SILT (SP-SM)** grey, wet, medium dense	
						-150
26.2	96	19	D	35		
						-155

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

DATE DRILLED:
7-23-18

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
14

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LOG OF BORING NO. B-7

FIGURE B-7

	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)
	33.5	85	13	D	40	**CLAY (CL)** grey with brown, wet, stiff	
							-160
	26.7		17	S	45	**SANDY SILT (ML)** grey, wet, very stiff	
							-165
	23.1 29.7	97 92	17	D	50	**SILT (ML)** grey, wet, stiff	
							-170
	31.7	88	11	D	55	@ 55 feet, firm	
							-175
	31.8		28	S	60	**CLAYEY SILT (ML)** grey, wet, very stiff	
							-180
					65		
							-185
	15.8		51	S	70	**SILTY SAND (SM)** grey, wet, very dense	
							-190
					75		
							-195

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

DATE DRILLED:
7-23-18

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
14

GPI

PROJECT NO.: 2884.I

COACHELLA

LOG OF BORING NO. B-7

FIGURE B-7

	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)
	16.2		26	S	80	@ 80 feet, medium dense	
						Total Depth 81.5 feet	

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

DATE DRILLED:
7-23-18

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
14

GPI

PROJECT NO.: 2884.I

COACHELLA

LOG OF BORING NO. B-7

FIGURE B-7

	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	Natural: **SANDY SILT (ML)** light brown, dry	
	3.6	89	14	D		**SILT (ML)** light brown grey, dry, stiff	-115
	3.7	100	15	D	5	**SILTY SAND (SM)** light brown, dry to slightly moist, loose	
	10.0	95	11	D		**SANDY SILT (ML)** light brown, slighty moist to moist, stiff, trace clay	-120
	27.3	91	6	D	10	**CLAYEY SILT (ML)** light brown grey, wet, firm	-125
	24.2		7	D	15	**SANDY SILT (ML)** grey, wet, very stiff	-130
	24.4		9	S	20		-135
			29	S	25		-140
	20.6		20	S	30	**SILTY SAND (SM)** grey, wet, medium dense	-145
					35		-150
	17.8	101	21	D		**SILTY CLAY (CL)** grey, wet, firm	

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

DATE DRILLED:
7-24-18

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
19

GPI

PROJECT NO.: 2884.I

COACHELLA

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)
					40		
	34.9		19	S		@ 42 feet, very stiff	-155
						CLAYEY SILT (ML) grey, wet, very stiff	
					45		
							-160
	29.4	95	17	D	50		
						SANDY SILT (ML) grey, wet, stiff, trace clay	
							-165
					55		
							-170
	28.9		19	S	60		
						SILT (ML) grey, wet, very stiff	
							-175
					65		
							-180
			32	S	70	@ 70 feet, no recovery	
							-185
					75		
							-190

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:
7-24-18

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
19

GPI

PROJECT NO.: 2884.I

COACHELLA

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* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
			26	S	80		@ 80 feet, no recovery	
							Total Depth 81.5 feet	

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

DATE DRILLED:
7-24-18

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
19

GPI

PROJECT NO.: 2884.I

COACHELLA

LOG OF BORING NO. B-8

	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	Natural: **SILT (ML)** light brown, very dry	-115
	4.1	96	14	D		**SILT (ML)** light brown, dry, stiff, trace sand	
	10.6	89	15	D	5	@ 5 feet, brown, moist	-120
	14.9	98	11	D		@ 7 feet, firm, trace clay	
						SANDY SILT (ML) light brown grey, moist, firm	
	29.5	92	6	D	10	**CLAY (CL)** grey, wet, firm	-125
	30.1	89	7	D	15		-130
	19.3		9	S	20	**SILTY SAND (SM)** light brown, grey, wet, loose	-135
	25.2		29	S	25	@ 25 feet, medium dense	-140
	12.7		20	S	30	**SAND (SP)** light brown, wet, medium dense	-145
	28.7	93	21	D	35	**CLAYEY SILT (ML)** grey, wet, stiff	-150

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

DATE DRILLED:
7-24-18
EQUIPMENT USED:
8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft):
19

GPI

PROJECT NO.: 2884.I
COACHELLA

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	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.	
				40	**CLAYEY SILT (ML)** grey, very moist, stiff	-155
				45		-160
28.0	92	23	D		**SILT (ML)** grey, wet, very stiff	
				50		-165
26.9 28.8		12	S	55	@ 55 feet, wet, stiff, trace sand	
					CLAYEY SILT (ML) grey, wet, stiff	-170
18.7		32	S	60	**SILTY SAND (SM)** grey, very moist to wet, dense	-175
					Total Depth 61.5 feet	

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

DATE DRILLED:
7-24-18

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
19

GPI

PROJECT NO.: 2884.I

COACHELLA

LOG OF BORING NO. B-9

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
				0	Natural: **SANDY SILT (ML)** light brown, dry	
3.4	91	21	D		@ 2 feet, stiff	-115
3.3	96	17	D	5		
3.0	97	12	D		**SILTY SAND (SM)** light brown, dry, loose	-120
29.9	91	12	D	10	**CLAY (CL)** grey with brown, wet, stiff	
26.6	92	8	D			-125
25.0	94	7	D	15	**CLAYEY SILT (ML)** grey, wet, firm, with shells	
						-130
28.7		9	S	20	**SANDY SILT (ML)** grey, wet, trace clay	
						-135
28.8		20	S	25	**SILT (ML)** grey, wet, very stiff	
						-140
18.7	108	26	D	30	**SILTY SAND (SM)** grey, wet, medium dense	
						-145
36.5	86	10	D	35	**SILT (ML)** light brown, wet, firm	
						-150

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:
7-24-18

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
19

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PROJECT NO.: 2884.I
COACHELLA

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)
	33.0	88	9	D	40	@ 40 feet, with shells	
						Total Depth 41 feet	

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

DATE DRILLED:
7-24-18

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
19

GPI

PROJECT NO.: 2884.I

COACHELLA

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	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	Natural: **SANDY SILT (ML)** light brown, very dry	
3.0	109	20	D		**SILT (ML)** light brown, dry, stiff	-120
3.3	85	21	D	5	**SILTY SAND (SM)** light brown, dry, medium dense	
1.1	95	22	D			-125
30.5	82	10	D	10	**CLAY (CL)** brown, wet, firm, trace silt	
					@ 12 feet, stiff	
30.7	87	16	D		**SILTY CLAY (CL)** light brown, wet, stiff	-130
22.3		18	S	15	**SAND with SILT (SP-SM)** grey, wet, medium dense	
					SILTY CLAY (CL) grey, wet, very stiff	-135
				20		
					@ 22 feet, stiff	
27.1	96	15	D		**SANDY SILT (ML)** grey, wet, stiff	-140
28.9		19	S	25	**CLAY (CL)** grey brown, wet, very stiff, trace silt	
						-145
19.4		24	S	30	**SILTY SAND (SM)** grey, wet, medium dense	
						-150
				35		
						-155

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

DATE DRILLED:
7-25-18

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
20

PROJECT NO.: 2884.I

COACHELLA

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)
						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.	
	34.3	85	8	D	40	**SANDY CLAY (CL)** grey, wet, stiff	
							-160
	26.4		21	D	45	**SAND (SP)** grey, wet, medium dense, trace silt	
							-165
	33.3						
	41.8		7	S	50		
						CLAY (CL) grey, wet, stiff	
						Total Depth 51.5 feet	

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

DATE DRILLED:
7-25-18
EQUIPMENT USED:
8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft):
20

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PROJECT NO.: 2884.I
COACHELLA

LOG OF BORING NO. B-11

FIGURE B-11

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

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. A summary of the percentages passing the No. 200 sieve is presented below.

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	PERCENT PASSING No. 200 SIEVE
B-1	2	Silty Sand (SM)	40
B-3	0-4	Sandy Silt (ML)	68
B-7	20	Sand w/Silt (SP-SM)	10
B-7	35	Silty Sand (SP-SM)	13
B-7	45	Sandy Silt (ML)	54
B-8	30	Silty Sand (SM)	26
B-8	50	Sandy Silt (ML)	59
B-10	15	Clayey Silt (ML)	91
B-10	30	Silty Sand (SM)	44
B-11	15	Sand w/Silt (SP-SM)	8
B-11	30	Silty Sand (SM)	20
B-11	45	Sand (SP)	5

ATTERBERG LIMITS

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

DIRECT SHEAR

Direct shear tests were performed on relatively undisturbed and remolded bulk samples in accordance with ASTM D 3080. The bulk samples were remolded to approximately 90 percent of the maximum dry density. The test specimens were placed in the shear machine, and a normal load comparable to the in-situ overburden stress was applied. The samples were inundated, allowed to consolidate, and then were sheared to failure at a strain rate of 0.001 to 0.002 inches per minute. 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-2 to C-6.

CONSOLIDATION

One-dimensional consolidation tests were performed on undisturbed samples in accordance with ASTM D 2435. After trimming the ends, the samples were placed in the consolidometer and loaded to up to 0.4 ksf. Thereafter, the samples were incrementally loaded to a maximum load of up to 25.6 ksf. The samples were inundated at 1.6 ksf. Sample deformation was measured to 0.0001 inch. Rebound behavior was investigated by unloading the sample back to 0.4 ksf. Results of the consolidation tests, in the form of percent consolidation versus log pressure are presented in Figures C-7 to C-9.

COLLAPSE

Collapse tests were performed on 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 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-6	15	Sandy Silt (SM)	33.1	5.1	5.2
B-9	7	Sandy Silt (SM)	14.9	1.5	1.5
B-10	7	Sand w/Silt (SP-SM)	3.0	1.4	2.2

COMPACTION TEST

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

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	OPIMUM MOISTURE (%)	MAXIMUM DRY DENSITY (pcf)
B-1	0–4	Silty Sand (SM)	112	13.0
B-7	0–4	Sandy Silt (ML)	111	14.0

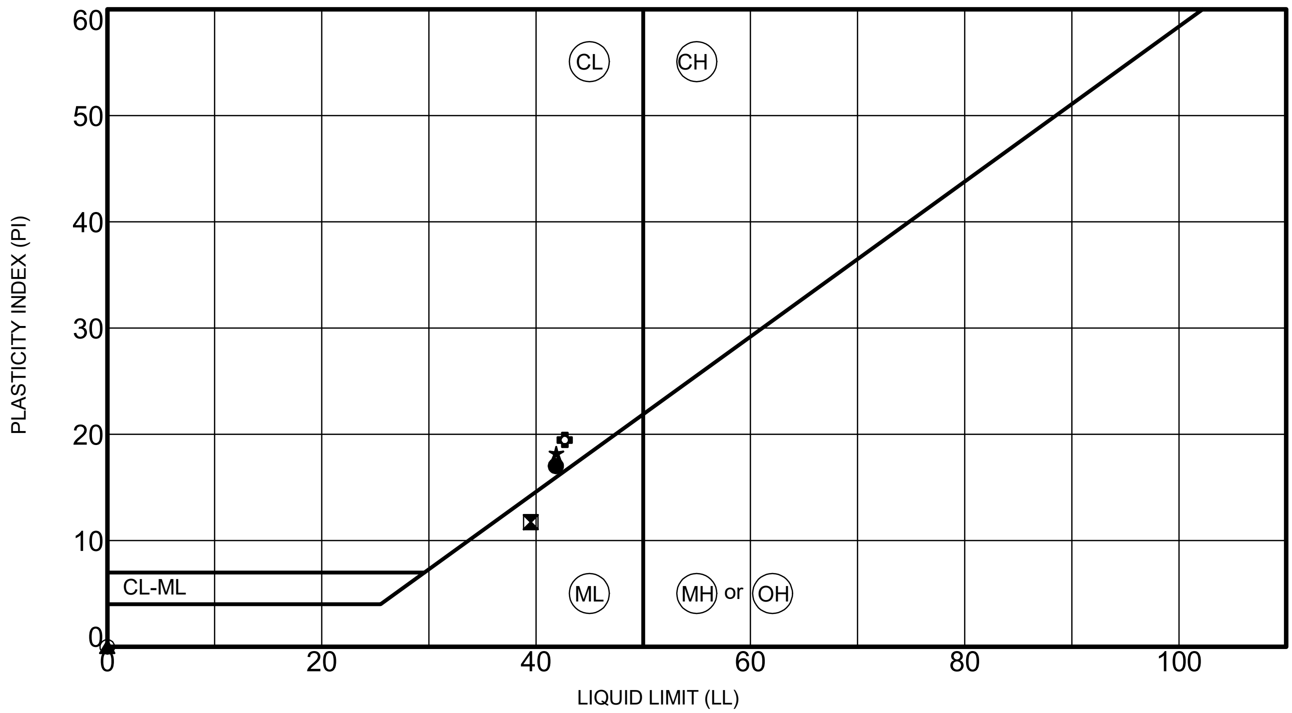
R-VALUE

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
B-3	0 – 4	Silt w/Gravel (ML)	42

CORROSIVITY

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



	SAMPLE LOCATION		LL	PL	PI	Fines, %	Classification
●	B-10	12.0	42	25	17		CLAY (CL)
⧗	B-10	35.0	40	28	12		SILT (ML)
▲	B-7	25.0	NP	NP	NP		SILT (ML)
★	B-7	40.0	42	24	18		CLAY (CL)
⊙	B-7	50.0	NP	NP	NP		SILT (ML)
✿	B-9	15.0	43	23	19		CLAY (CL)

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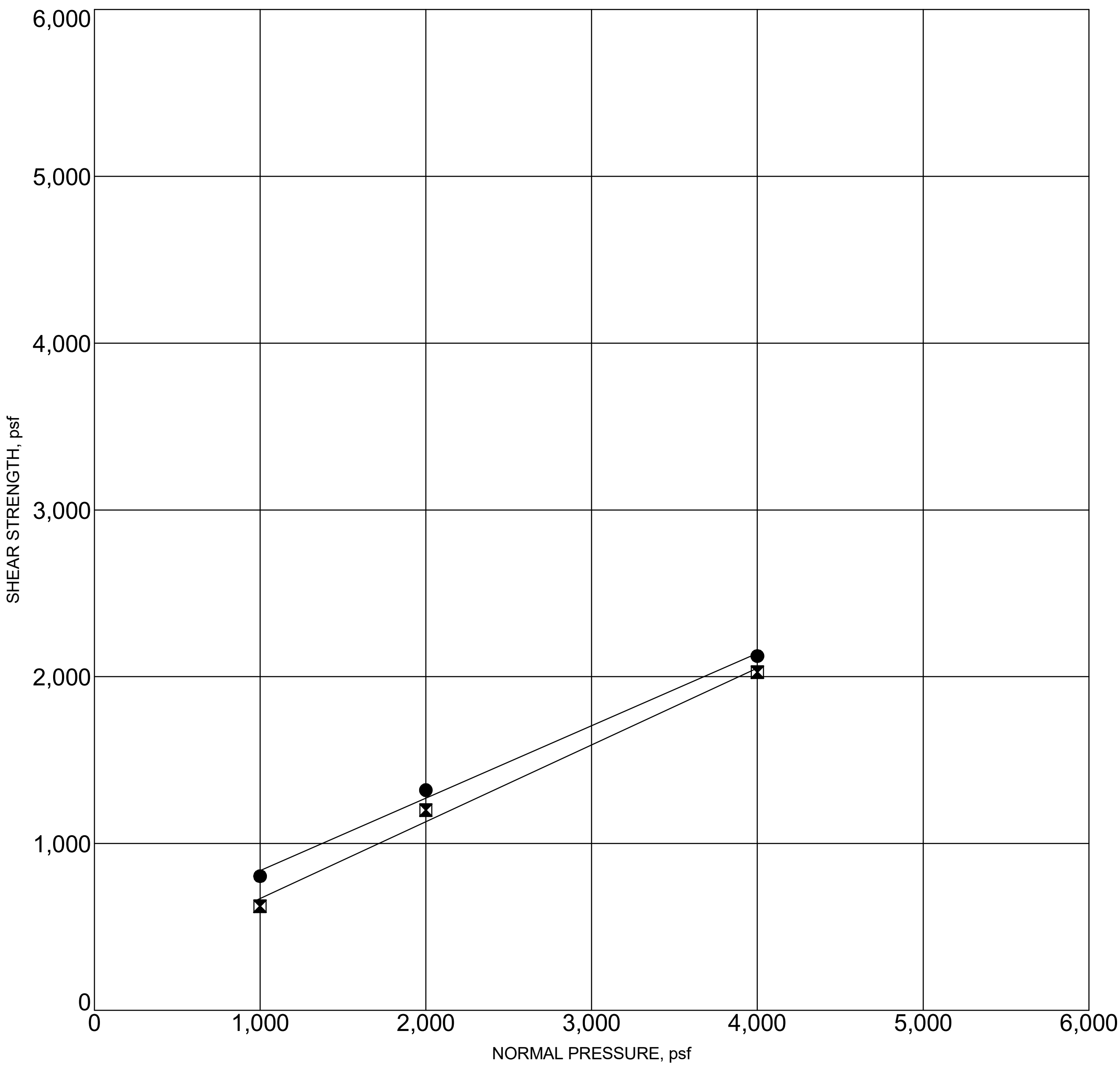
ATTERBERG LIMITS TEST RESULTS

FIGURE C-1



● **PEAK STRENGTH**
Friction Angle= 23 degrees
Cohesion= 402 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 25 degrees
Cohesion= 210 psf

Sample Location		Classification	DD,pcf	MC,%
B-6	10.0	CLAYEY SILT (ML)	93	23.5

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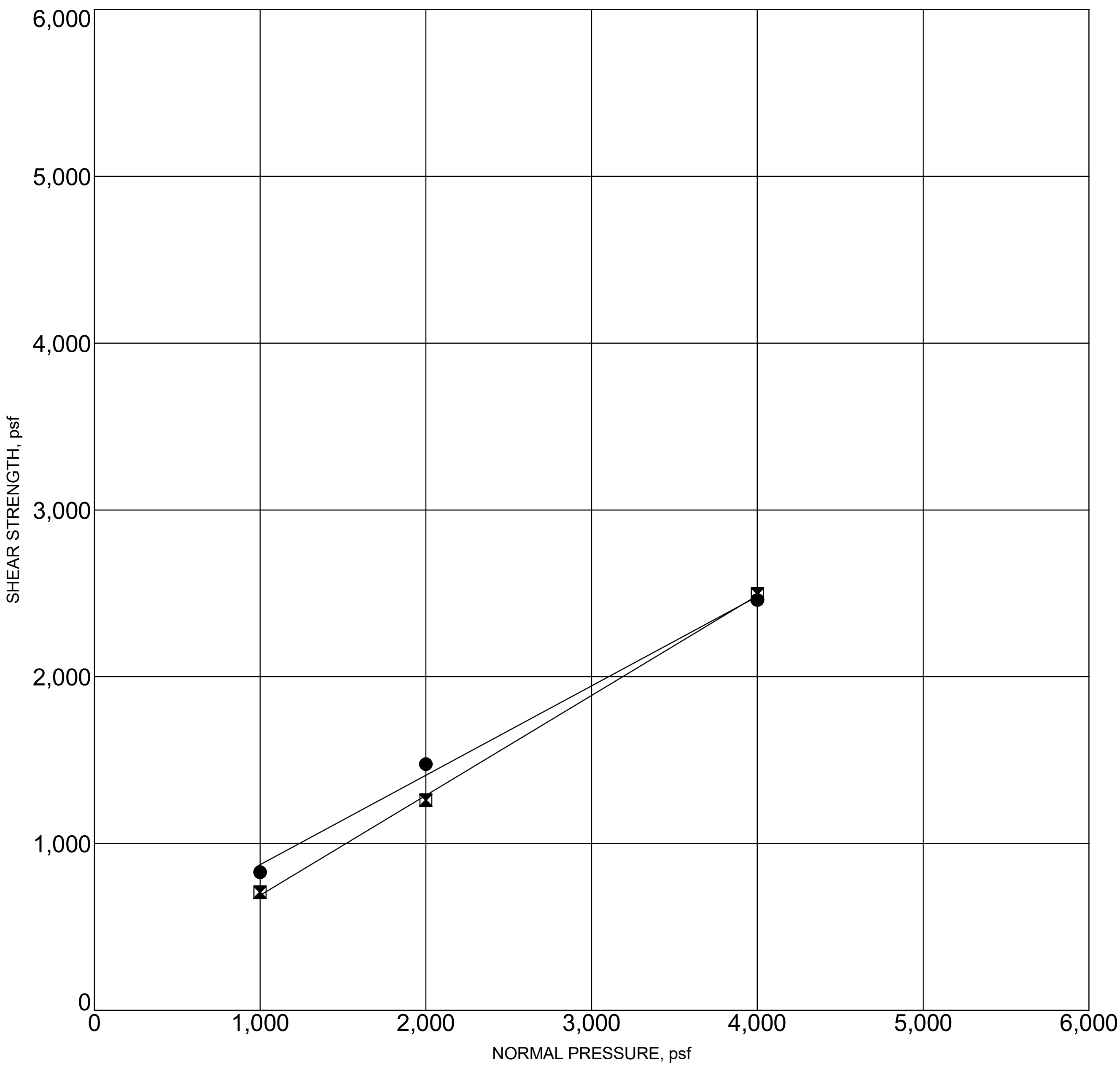
DIRECT SHEAR TEST RESULTS

FIGURE C-2



● **PEAK STRENGTH**
Friction Angle= 28 degrees
Cohesion= 336 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 31 degrees
Cohesion= 90 psf

Note: Samples remolded to 90% maximum dry density

Sample Location		Classification	DD,pcf	MC,%
B-7	0-4	SANDY SILT (ML)	100	14.0

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PROJECT NO.: 2884.I

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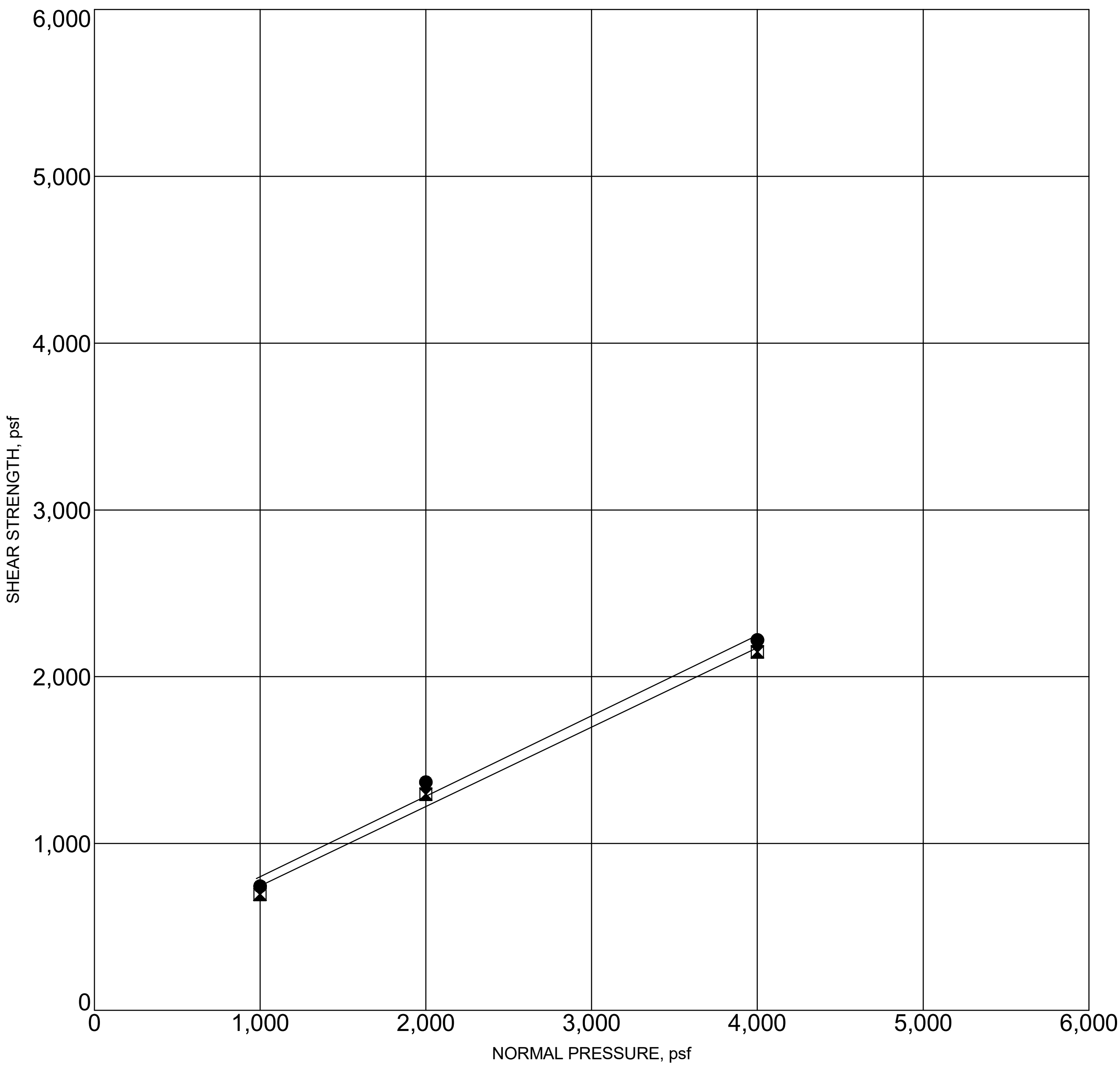
DIRECT SHEAR TEST RESULTS

FIGURE C-3



● **PEAK STRENGTH**
Friction Angle= 26 degrees
Cohesion= 318 psf

ULTIMATE STRENGTH
Friction Angle= 25 degrees
Cohesion= 270 psf

Sample Location		Classification	DD,pcf	MC,%
B-7	7.0	SILTY SAND (SM)	92	14.1

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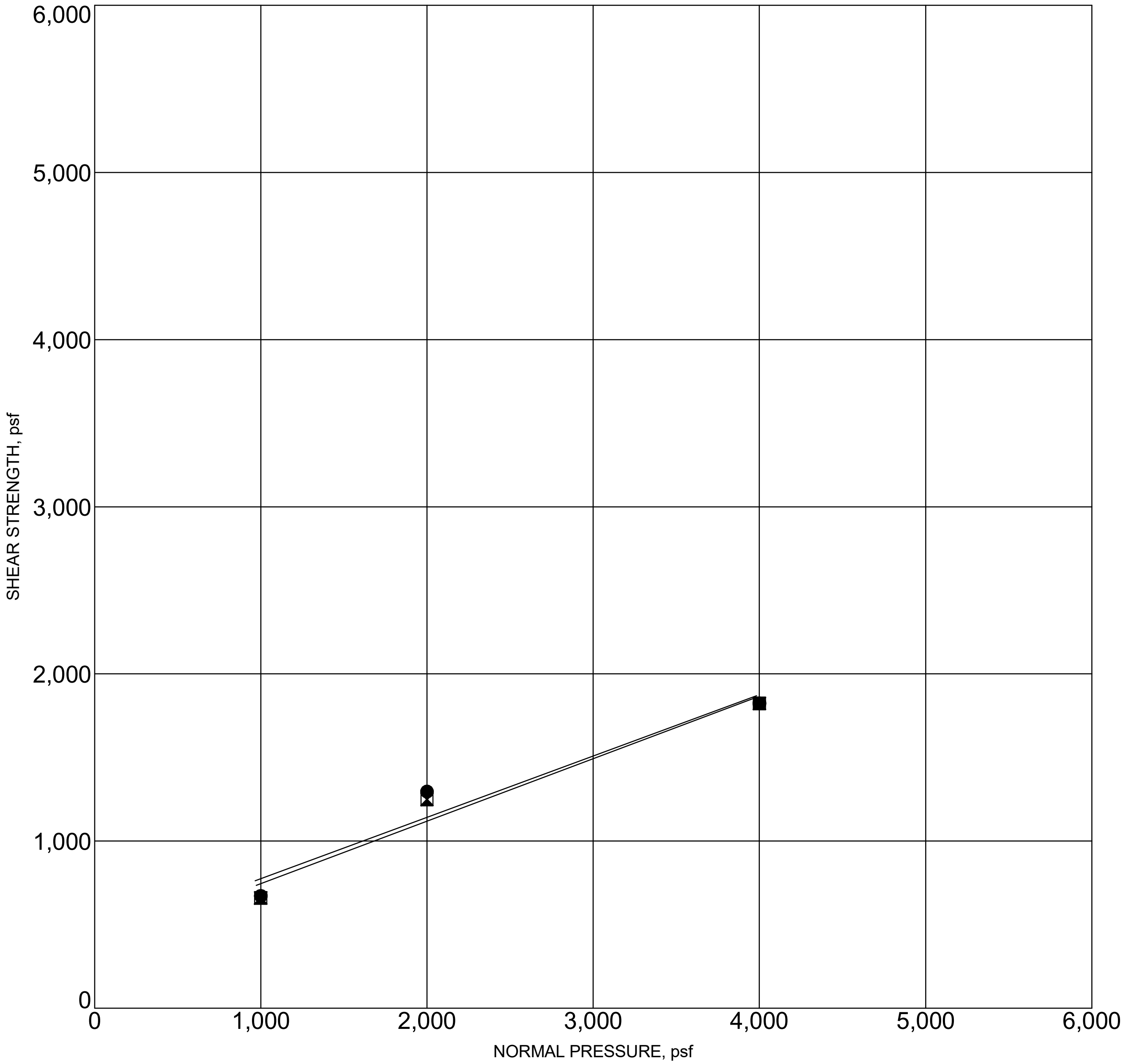
DIRECT SHEAR TEST RESULTS

FIGURE C-4



● **PEAK STRENGTH**
Friction Angle= 20 degrees
Cohesion= 408 psf

ULTIMATE STRENGTH
Friction Angle= 20 degrees
Cohesion= 372 psf

Sample Location		Classification	DD,pcf	MC,%
B-10	5.0	SANDY SILT (ML)	96	3.3

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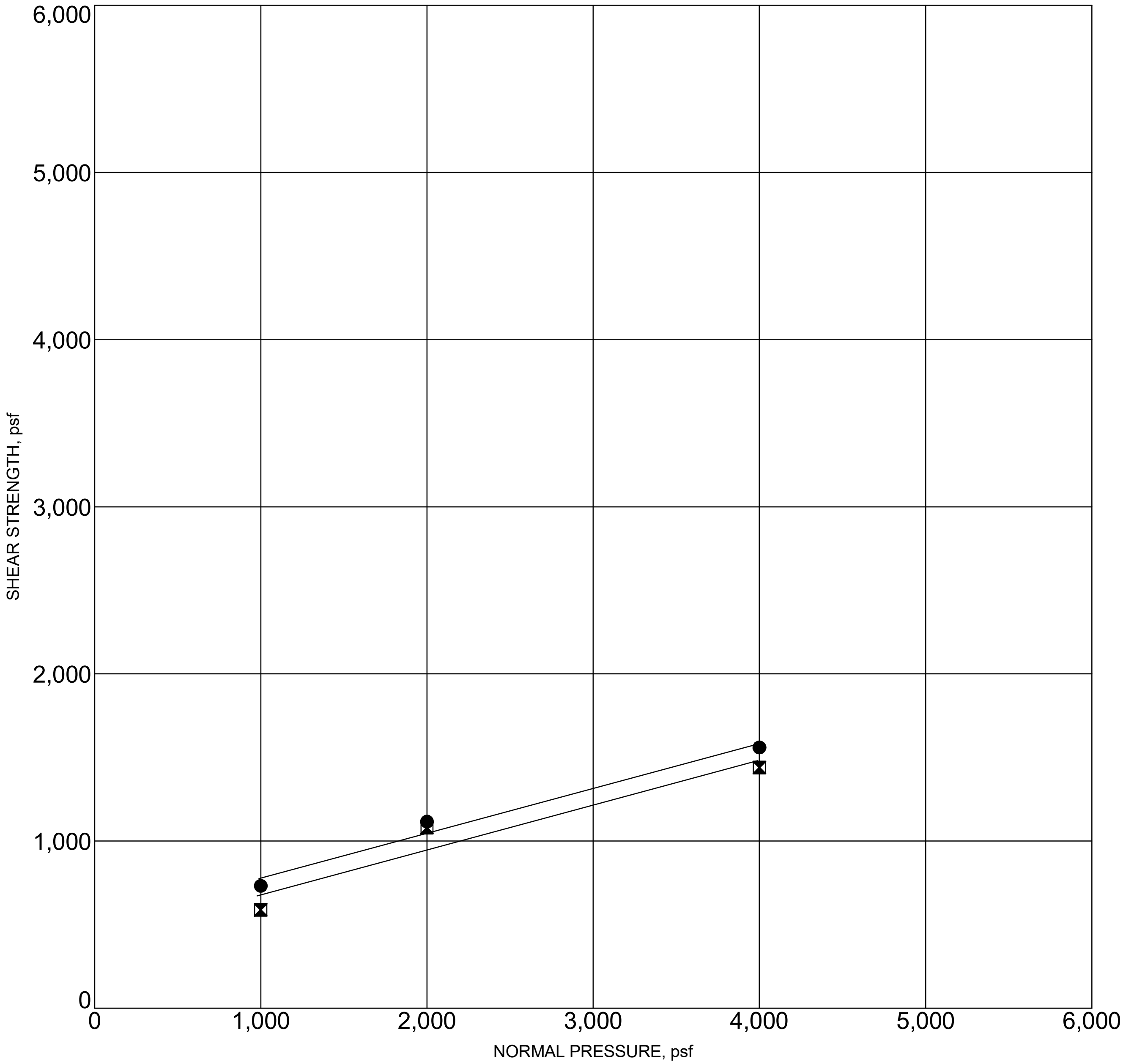
DIRECT SHEAR TEST RESULTS

FIGURE C-5



● **PEAK STRENGTH**
Friction Angle= 15 degrees
Cohesion= 510 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 15 degrees
Cohesion= 408 psf

Sample Location		Classification	DD,pcf	MC,%
B-11	10.0	CLAY (CL)	82	30.5

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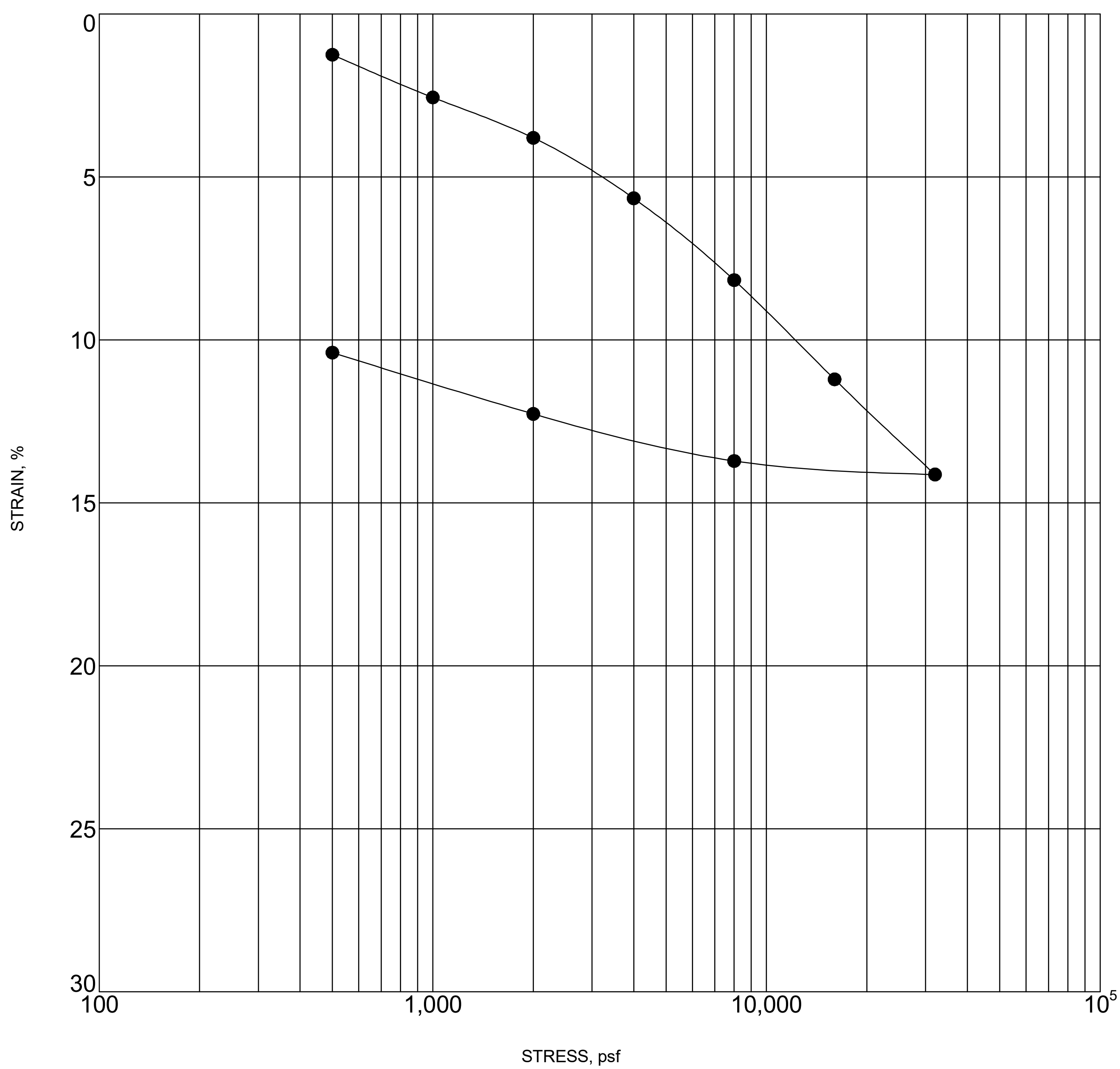
DIRECT SHEAR TEST RESULTS

FIGURE C-6



Sample inundated at 2000 psf

	Sample Location		Classification	DD,pcf	MC,%
●	B-6	32.0	SILT (ML)	82	39.2

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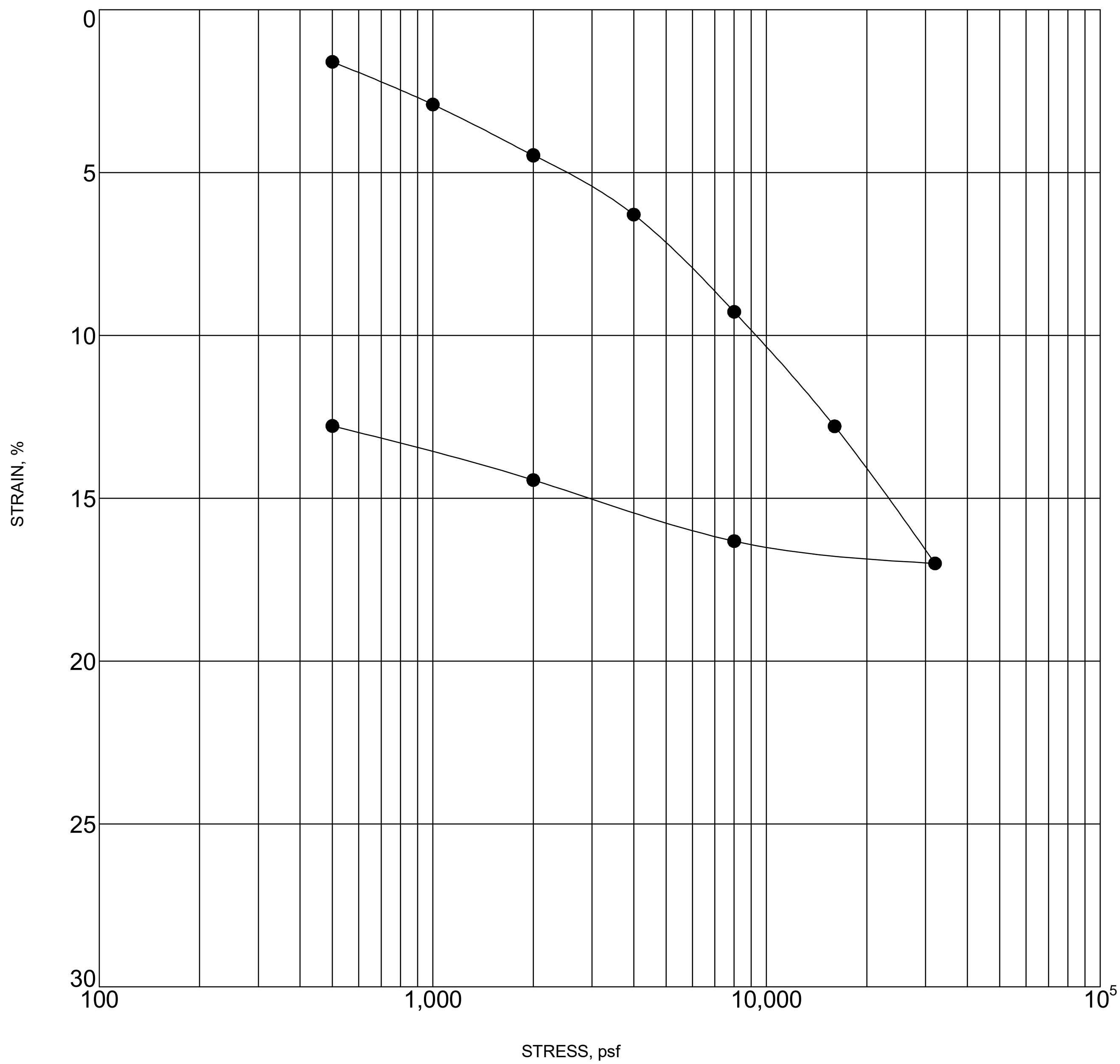
CONSOLIDATION TEST RESULTS

FIGURE C-7



Sample inundated at 2000 psf

	Sample Location		Classification	DD,pcf	MC,%
●	B-9	15.0	CLAY (CL)	89	30.1

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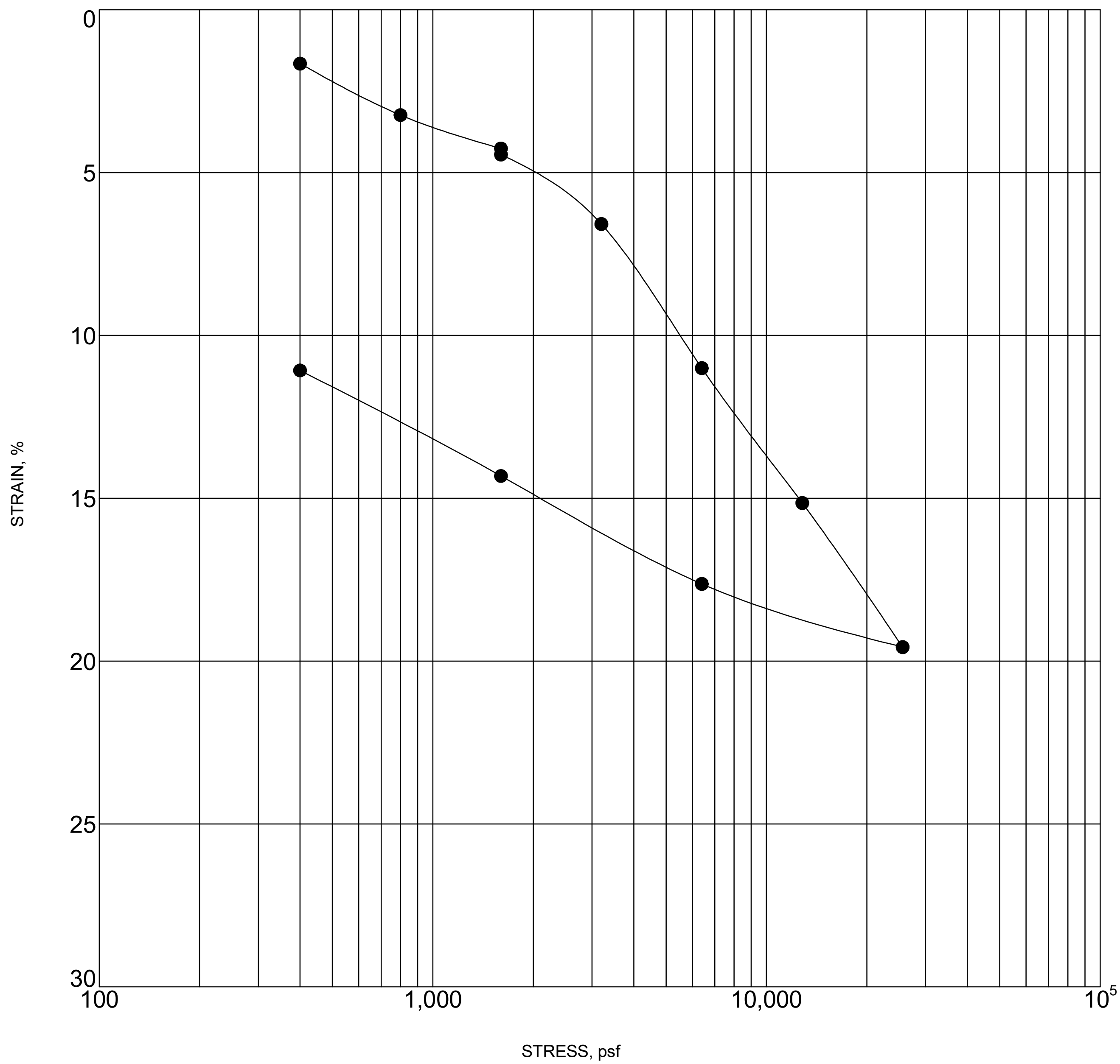
CONSOLIDATION TEST RESULTS

FIGURE C-8



Sample inundated at 1600 psf

	Sample Location		Classification	DD,pcf	MC,%
●	B-11	10.0	CLAY (CL)	82	30.5

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CONSOLIDATION TEST RESULTS

FIGURE C-9

Table 1 - Laboratory Tests on Soil Samples

Geotechnical Professionals, Inc.
Coachella Business Park
Your #2884.I, HDR Lab #18-0502LAB
9-Aug-18

Sample ID

			B-3 @ 0-4'	B-7 @ 0-4'
Resistivity		**Units**		
as-received		ohm-cm	2,480	600,000
saturated		ohm-cm	160	1,040
pH			7.7	7.7
Electrical Conductivity		mS/cm	3.04	0.27
Chemical Analyses				
Cations				
calcium	Ca^{2+}	mg/kg	1,220	100
magnesium	Mg^{2+}	mg/kg	232	16
sodium	Na^{1+}	mg/kg	2,290	128
potassium	K^{1+}	mg/kg	218	40
Anions				
carbonate	CO_3^{2-}	mg/kg	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	95	146
fluoride	F^{1-}	mg/kg	7.8	4.0
chloride	Cl^{1-}	mg/kg	2770	125
sulfate	SO_4^{2-}	mg/kg	4,080	163
phosphate	PO_4^{3-}	mg/kg	ND	ND
Other Tests				
ammonium	NH_4^{1+}	mg/kg	ND	ND
nitrate	NO_3^{1-}	mg/kg	861	174
sulfide	S^{2-}	qual	na	na
Redox		mV	na	na

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