Appendix B
Geotechnical Investigation

GEOTECHNICAL
PROFESSIONALS INC.

GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE APARTMENT DEVELOPMENT
16922 MAGNOLIA STREET
FOUNTAIN VALLEY, CALIFORNIA

Prepared for:
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Long Beach, CA 90815

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Project No. 3033.1I

November 3, 2023

GEOTECHNICAL
PROFESSIONALS INC.

November 3, 2023

Holland Acquisition CO, LLC
5000 E. Spring St., Suite 500
Long Beach, CA 90815

Attention: Mr. Jacob Stone

Subject: Report of Geotechnical Investigation
Mixed-Use Apartment Development
16922 Magnolia Street
Fountain Valley, California
GPI Project Number: 3033.1I

Dear Mr. Stone:

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

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

Very truly yours,
Geotechnical Professionals Inc.

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

Donald A. Cords, G.E.
Principal

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1.0 INTRODUCTION

1.1 GENERAL

This report presents the results of a geotechnical investigation performed by Geotechnical Professionals Inc. (GPI) for the proposed mixed-use apartment development to be located at 16922 Magnolia Street in Fountain Valley, California. The general site location is shown on the Site Location Map, Figure 1.

1.2 PROJECT DESCRIPTION

The proposed project will consist of a 2-parcel mixed-use apartment development, including 5-stories of wood framed apartments over 2 levels of concrete and 8 stories of concrete parking structures. Both parcels will include the structures as described above. The site location is shown on the Site Location Map, Figure 1.

Structural loads and specific building design details are not available at this time. Based on experience with similar projects, maximum column loads are expected to be approximately 1200 to 1800 kips for the parking structures and maximum wall loads are expected to be approximately 4 to 6 kips per lineal foot for the apartments.

We understand that there will be no subterranean structures associated with this project.

1.3 PURPOSE OF INVESTIGATION

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

2.0 SCOPE OF WORK

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

Our field exploration consisted of six CPT's and five exploratory borings. The locations of the subsurface explorations are shown on the Site Plan, Figure 2.

Our CPT's were advanced to depths ranging from 86 to 100 feet below existing site grades. Detailed logs of the CPT's and a summary of the equipment used are presented in Appendix A. Our exploratory borings were drilled using truck-mounted hollow-stem auger drilling equipment to depths of 11 to 60 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, fines content, shear strength, consolidation, expansion potential, maximum density/optimum moisture, and soil corrosivity. Laboratory testing procedures and results are summarized in Appendix C.

Soil corrosivity testing was performed by Project X Corrosion under subcontract to GPI.

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

3.0 SITE CONDITIONS

3.1 SURFACE CONDITIONS

The project site previously consisted of a family entertainment center. The previous development has since been demolished. The site is currently vacant with remaining asphalt concrete and concrete pavements along with landscape areas.

Historic aerial photographs (historicaerials.com) indicate that the family entertainment center was developed sometime between 1972 and 1987. Prior to the entertainment center, the site was undeveloped and appears to have been used for agriculture.

The site is bounded on the north by the San Diego Freeway, on the east by the San Diego Freeway and a roller-skating center, on the south by Recreation Circle, and on the west by Magnolia Street.

The ground surface at the site is relatively flat. Based on Google Earth, ground surface elevation at the north end of the site is approximately +31 feet with a gentle slope to the south to an elevation of approximately +28 feet.

3.2 SUBSURFACE SOILS

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

In general, we encountered undocumented fills in our explorations ranging from 2 to 5 feet of existing site grades. Deeper fills may be encountered in the footprints of the previous buildings. The fills consist of silty sands and sandy silts. The fill soils were generally moist to very moist, medium dense and very stiff based on field explorations and laboratory testing. Documentation regarding the placement and compaction of the fill soils was not provided.

The natural soils consist predominantly of moist to very moist, interbedded layers of sands, clays, silts, and their mixtures to a depth of approximately 60 feet. These soils are generally moist to wet. The sandy soils were medium dense to very dense while the fine-grained soils were firm to very stiff. From 60 feet to the 100-foot depth explored the soils consisted of dense to very dense silty sands and sands.

3.3 GROUNDWATER AND CAVING

Groundwater was measured at depths of approximately 9 to 10 feet in our borings. Historical high groundwater level in the site vicinity has been reported at about 5 feet below the existing grades (CDMG, 1997). Due to the methods of drilling, the potential for caving was very difficult to determine but sandy layers below groundwater can be anticipated to exhibit caving.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 GENERAL

Based on the results of our investigation, it is our opinion that from a geotechnical viewpoint it is feasible to develop the site as proposed, provided the following geotechnical issues are incorporated into the design and construction of the proposed structure:

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

- The site is located in a Seismic Hazard Zone for liquefaction (CDMG, 1997). There is a potential for liquefaction induced seismic settlement to adversely impact the buildings. Layers of silty and sandy soil between depths of approximately 5 to 55 feet below existing grade exhibit a significant potential for liquefaction from a design earthquake. Should liquefaction of these layers occur, our estimate is that the magnitude of induced settlement at the ground surface is expected to be on the order of 1 to 2 inches.

- The natural clays in the upper 30 feet of the soil profile are highly compressible. The placement of structural loads from the parking structure and apartments will cause significant static settlement. Based on our analysis, the settlement will be greater than typically tolerable by shallow foundations without ground improvement.

- The apartment buildings may be supported on mat foundations or shallow spread footings with ground improvement to control static and liquefaction induced settlement. A slab supported by ground improvement with 12 inches of non-expansive fill or structural slab should also be utilized.

- Parking structure foundations may be supported ground improvement to control static and liquefaction induced settlement. If the potential of excessive settlement of the parking structure slab is not tolerable, the slab-on-grade should be a slab supported by ground improvement or structural slab.

- A portion of near surface silty soils have the potential to be expansive, which shrink and swell with changes in moisture content. Flatwork and slabs-on-grade should be placed non-expansive compacted fill. Granular soils are present at the site and should soils are separated during grading. Further testing should be done during grading to determine the expansion potential of localized soils.

- Excavations extending deeper than approximately 5 feet are expected to encounter moist to very moist and may encounter locally wet soil conditions. Stabilization of excavation bottoms and drying of soils may be required prior to placement of fill. Heavy rubber-tire equipment could cause “pumping” and disturbance of the subgrade soils. The contractor should evaluate the in-place moisture conditions when planning the work to allow for moisture conditioning and reducing subgrade disturbance.

- Storm water infiltration is not to be feasible at the site. Guidelines for storm water infiltration by the County of Orange do not allow infiltration into potentially liquefiable soils.

- The on-site soils are corrosive for buried metals.

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

4.2 SEISMIC CONSIDERATIONS

4.2.1 General

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

We assume the seismic design of the proposed development will be in accordance with the California Building Code (CBC) 2022 edition. For the 2022 CBC, a Site Class D may be used based on the results our shear wave measurements measured during out investigation The weighted average velocity was calculated to be approximately 720 ft/sec in the upper 100 feet. Using the Site Class, which is dependent on geotechnical issues, and the appropriate seismic design maps, the corresponding seismic design parameters from the CBC are as follows:

S_S = 1.38g	$S_{MS} = F_a * S_S$ = 1.38 g	S_{DS} = 2/3 * S_{MS} = 0.92g
S_1 = 0.50g	$S_{M1} = F_V * S_1$ = 0.90 g	S_{D1} = 2/3 * S_{M1} = 0.60g

These above values for the seismic design parameters should be confirmed by the Project Structural Engineer prior to their use.

In accordance with the 2022 CBC, site-specific response spectra are required for structures located in a Site Class D (with S_1 greater than or equal to 0.2) unless, per the exceptions detailed in Section 11.4 8 of ASCE 7-16, the structure is designed using seismic response coefficient (C_S) determined by either:

- Equation 12.8-2 for values of $T \leq 1.5\ T_S$,
- 1.5 times the value computed by Equation 12.8-3 for values of $T_L \geq T > 1.5\ T_S$, or
- 1.5 times the value computed by Equation 12.8-4 for values of $T > T_L$.

If this exception is not taken and the structure will still be designed in accordance with the 2022 CBC, GPI should be notified that site-specific response spectra is requested.

The Project Structural Engineer should determine the seismic design method.

4.2.2 Strong Ground Motion Potential

Based on published information (USGS, 2008), the most significant faults in the proximity of the site are the San Joaquin Hills and Newport Inglewood Faults, which are located about 3 miles from the site, respectively.

During the life of the project, the site will likely be subject to strong ground motions due to earthquakes on nearby faults. Based on the ASCE 7 website (asce7hazardtool.online), we computed that the site could be subjected to a peak ground acceleration (PGA_M) of 0.65 for a modal magnitude 6.8 earthquake. This acceleration has been computed using the mapped Maximum Considered Geometric Mean peak ground acceleration from the ASCE 7-16 (for 2022 CBC) and a site coefficient (F_{PGA}) based on Site Class. The predominant earthquake magnitude was determined using a 2-percent probability of exceedance in a 50-year period, or an average return period of 2,475 years. The structural design will need to incorporate measures to mitigate the effects of strong ground motion.

4.2.3 Potential for Ground Rupture

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

4.2.4 Liquefaction

Soil 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 soils. Thus, three conditions are required for liquefaction to occur: (1) a cohesionless soil of loose to medium density; (2) a saturated condition; and (3) rapid large strain, cyclic loading, normally provided by earthquake motions.

The site is located within a zone identified as having a potential for liquefaction by the State (CDMG, 1997). We evaluated the potential for liquefaction using methods presented in Robertson, 2009 (CDMG, 1997) and modifications provided in Special Publication 117A (CDMG, 1997).

To evaluate the potential for liquefaction at the site, we considered recent and historic groundwater levels. Ground water levels were encountered at depths as shallow as 9 feet in our recent exploration by others. We used a groundwater depth of 5 feet, corresponding to historical high level for the State (CDMG, 1997) for our evaluation.

Based on our evaluation of the CPT data using computer software CLIQ (GeoLogismiki) should a design earthquake occur, the majority of the soils that exhibit a potential for liquefaction induced seismic settlement are present between the depths of approximately 5 to 25 feet. In general, the potentially liquefiable layers consist of medium dense silty sands and firm sandy silts. Laboratory testing was performed on representative samples of the cohesive soil deposits to evaluate their susceptibility to liquefy. Total liquefaction-induced settlement is estimated to be on the order of 1 to 2 inches at the ground surface, with differential settlement of approximately ½ to 1-inch across a span of 40 feet.

Seismic ground subsidence, not related to liquefaction, occurs when loose, granular soils above the groundwater are densified during strong earthquake shaking. Seismic ground subsidence (not related to liquefaction induced settlements) occurs when strong earthquake shaking results in densification of loose to medium dense sandy soils above groundwater. Due to the shallow depths to groundwater used in our liquefaction analysis and the limited amount of sandy soils above this groundwater level that will not be densified during remedial earthwork, the potential for dry seismic to adversely affect the site is considered to be low. As such, we do not anticipate measurable seismic settlement of the soil above the groundwater.

4.3 MITIGATION OF SETTLEMENT

Supporting the proposed parking structures and apartments on a reasonable layer of engineered fill over the natural soils will result in static foundation settlements well beyond the typical performance of a shallow foundation system. Static settlements along with the additional anticipated seismic settlement on the order of 1 to 2 inches, spread footings are not feasible for the buildings. Typically, total settlement (static and seismic) for spread footings is limited to 1½ inches or less. Based on the subsurface conditions and space constraints at the site, the option of mitigating these settlements by performing deeper remedial grading is not feasible.

Alternatives to limit the total settlement (static and seismic) to acceptable limits include ground improvement, proprietary foundation/ground improvement method such as rammed aggregate piers (such as Geopier), rammed aggregate piers supported mat foundations (for the apartments only), or deep foundations (piles).

Rammed aggregate piers consist of drilled holes that are filled with aggregate base or gravel that is mechanically compacted as it is placed. Rammed aggregate piers should be installed under foundations for the proposed parking structures. They should also be installed under the parking structure slabs-on-grade unless settlements up to 2 inches under the floor slab in the event of a design level earthquake is tolerable. Rammed aggregate piers should also be installed under mat foundations for the proposed apartments. If shallow foundations are constructed for the apartments, rammed aggregate piers should be installed under the slab-on-grade unless a structural floor slab is utilized. Because such systems are proprietary, the work is performed on a design-build basis by the specialty contractor. Our design review is typically limited to confirming that the soil parameters used are consistent with the data provided in this report.

In order to support the parking structures or apartments on spread footings and slab-on-grade floors, the ground improvement design will require reducing the static settlement under the footings to less than 1 inch. Based on our analysis we anticipate that the depth of rammed aggregate piers will be required to extend to at least 25 feet.

In order to support the apartments on mat foundations, the ground improvement design will require reducing the static settlement under the footings to less than 2 inches. Based on our analysis we anticipate that the depth of rammed aggregate piers will be required to extend to at least 17 feet.

The design and construction of the rammed aggregate piers are the sole responsibility of the ground improvement designer. In addition, foundation design parameters, including estimated settlements of footings bearing on rammed aggregate piers, must be provided by the ground improvement designer.

A representative of GPI should observe the installation of the rammed aggregate piers, including confirmation of the diameter of the drilled hole, specified lift thickness and duration of compaction effort. The locations of the rammed aggregate piers should be confirmed by the project surveyor.

The placement of new fill over the existing grade with a thickness of greater than about 2 feet will cause long-term settlement of the underlying compressible soils. While grading at the site is not anticipated to raise grades by more than a couple feet, the earth-filled ramps within the lower level of the parking structures will be placed up to approximately 10 feet above pad grade. We anticipate the placement of new fill for the ramp will cause up to ¼-inch of settlement within the silts and clays for every 2 feet of fill placed over pad elevation for at-grade structures. Due to the interbedded sandy soils within the silts and clays, the settlement will likely occur over a period ranging from approximately 30 to 45 days. We recommend that the concrete slab at the ramp not be constructed for at least 45 days after completion of fill. As an alternative, the ramp fill could be monitored with survey points to determine when the underlying settlement has been substantially completed. The ramp fill should be surveyed on a weekly basis and data provided to GPI for review. When GPI determines that the settlement of the ramp has been substantially completed, the concrete slab may be poured. Other alternatives are to backfill the ramp with a lightweight fill, such as Geofoam, or to install rammed aggregate piers under the ramp footprint to reduce the anticipated settlement to less than ½-inch and the settlement time. The design of the depth and spacing of rammed aggregate piers under the ramp to achieve an acceptable settlement and waiting time should be performed by the ground improvement designer.

4.4 EARTHWORK

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

4.4.1 Clearing and Grubbing

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

If pile foundations are encountered from the previous development, GPI should observe the demolition of the existing foundation elements. We recommend the concrete piles be removed to a depth of at least 5 feet below the proposed foundations for the parking structures and buildings and at least 2 feet below floor slabs. We recommend the location and depth of the piles be surveyed during demolition. The resulting excavation should be backfilled as recommended in the "Subgrade Preparation" and "Placement and Compaction of Fill" sections of this report. Excavation and complete removal of the piles should be avoided.

Although none were encountered, cesspools or septic systems encountered during grading should be removed in their entirety. The resulting excavation should be backfilled as recommended in the "Subgrade Preparation" and "Placement and Compaction of Fill" sections of this report. As an alternative, any cesspools can be backfilled with a lean sand-cement slurry. At the conclusion of the clearing operations, a GPI representative should observe and accept the site prior to any further grading.

4.4.2 Excavations

Prior to construction of foundation supported improvements and slabs-on-grade, undocumented fill soils or soils disturbed by demolition beneath the proposed building should be removed and replaced as properly compacted fill. For planning purposes, we recommend that the soils be removed to a minimum depth of 4 feet below the existing grade within the footprint of the structures to remove undocumented fill at the site and to provide a uniform working surface for ground improvement installation. Deeper removals may be required in the footprint of the existing buildings or other areas based on observations of undocumented fills during grading by GPI's field technician. The base of removals should extend laterally a minimum distance of 5 feet beyond the building lines.

Alternatively, if a structural slab is utilized for the parking structures or apartment buildings, the need for the removal of the undocumented fills may be waived for that structure.

In addition to the undocumented fills or disturbed soils in the building footprints, a portion of the upper natural soils should be removed and replaced as properly compacted fill prior to placing fills outside of the building. For planning purposes, if silts or clays are encountered where new hardscape/flatwork surrounding the proposed buildings, removals for should extend at least 12 inches below the proposed subgrade and replaced with imported, non-expansive soils (E.I.$\leq$20).

The above recommendations assume that the footings for the proposed parking structures and will be supported on ground improvement to reduce static and seismic settlement and that the apartment buildings will be supported on mat foundation or shallow spread footings with a structural slab or ground improvement supported slab.

The footings for lightly loaded structures, trash enclosures, screen walls, and canopies should be supported on at least 2 feet of properly compacted fill.

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

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 associated with 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 need to be confirmed in the field. We recommend that all known utilities be shown on the grading plan.

Temporary construction excavations may be made vertically without shoring to a depth of 5 feet below adjacent grade. For deeper cuts up to 10 feet, the slopes should be properly shored or sloped back at least 1:1 or flatter. For cuts deeper than 10 feet, the slopes should be properly shored or sloped back at least 1½:1 (h:v) or flatter. Groundwater should be anticipated in excavations approaching approximately 9 feet below existing grade. The Contractor should be responsible in properly controlling groundwater in deeper excavations. The exposed slope face should be kept moist (but not saturated) during construction to reduce local sloughing. No surcharge loads should be permitted within a horizontal distance equal to the height of cut from the top of the excavation or 5 feet from the top of the slopes, whichever is greater, unless the cut is properly shored. Excavations that extend below an imaginary plane, inclined at 45 degrees below the edge of adjacent existing site facilities, should be properly shored to maintain support of adjacent elements. Excavations and shoring systems should meet the minimum requirements given in the most current State of California Occupational Safety and Health Standards.

4.4.3 Subgrade Preparation

Prior to placing fills or construction of the proposed structures, the subgrade soils should be scarified to a depth of 8 inches, moisture-conditioned, and compacted to at least 90 percent of the maximum dry density in accordance with ASTM D 1557, and to a firm and unyielding condition. In areas to receive pavements (outside of the structures), the top 12 inches below the pavement base should be scarified, moisture-conditioned, and compacted to a minimum of 95 percent (90 percent for cohesive soils) of the maximum dry density. To reduce the potential for subgrade disturbance, subgrade processing requirements may be waived if wet subgrade conditions are encountered, as determined by GPI in the field during grading.

Where exposed, care should be taken to prevent the clayey subgrade soils from drying out during construction. Moisture conditioning should be performed on subgrade soils allowed to dry prior to placing overlying fill or improvements.

The subgrade soils at depths greater than about 5 feet may exhibit over-optimum conditions. Subjecting these materials to heavy rubber-tired equipment may induce pumping/rutting, possibly requiring stabilization with geogrid and aggregate base. It is our opinion that steel track/wheel equipment will minimize disturbance of these materials. The contractor should determine the appropriate type of equipment to minimize disturbance of the over optimum soils.

Stabilization of subgrade soils may be required to facilitate the support of heavy equipment and the compaction of fills. For cost estimating purposes, the stabilization can consist of 12 inches of aggregate base placed over a geogrid, such as Tensar BX1100, Tensar TX140, or equivalent. The recycled on-site concrete and asphalt can be used for stabilization if processed as discussed in the following report section. Based on the laboratory testing of near-surface soils, lime or cement treatment for stabilization is likely to be a feasible alternative for stabilization.

4.4.4 Material for Fill

The on-site soils are, in general, suitable for use as compacted fill under the structures. Silts and clays soils should not be used directly underneath slab-on-grade floors (including the parking structures) or exterior flatwork as described in the "Excavations" section of this report. Fine-grained soils are not suitable for use as ramp or retaining wall backfill unless the walls are specifically designed for increased pressure and the risk is acceptable to the Owner. Granular, non-expansive soils are limited at the site and may need to be imported.

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). 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.

When aggregate base is recommended in this report, the material may be crushed aggregate base or crushed miscellaneous base.

On-site inert demolition debris, such as concrete and asphalt, may be reused in the compacted fills provided approval is obtained from the reviewing regulatory agency and the Owner. The material should be crushed to the consistency of aggregate base and blended with the on-site or imported soils. The recycled material may be used for non-expansive fill or aggregate base under slab-on-grade floors and exterior flatwork. The recycled material may also be used for stabilization of soft and wet areas if encountered in the planned overexcavations.

4.4.5 Placement and Compaction of Fills

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

Plate compactors	4-6 inches
Small vibratory or static rollers (5-ton±)	6-8 inches

Scrapers and heavy loaders	8-12 inches

The maximum lift thickness should not be greater than 12 inches and each lift should be thoroughly compacted and accepted prior to placement of subsequent lifts.

The moisture content of the on-site clayey soils should be between 1 to 3 percent over the optimum moisture content to readily achieve the required degree of compaction. Fills consisting of the imported or on-site sandy soils, if encountered, should be placed at a moisture content of 0 to 2 percent over the optimum moisture content in order to achieve the required compaction and reduce the potential for future swelling.

The on-site soils are generally near or above the optimum moisture content such that a drying of the fill during grading may be required. The on-site soils are susceptible to easily becoming wet when exposed to rain and can be difficult to dry back to near optimum. In the event that the on-site soils are subjected to rain, a significant amount of drying, including discing, during grading will be required.

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 related to shallow overexcavations for slab support and minor structures only, an average shrinkage value of 10 to 15 percent may be assumed for the surficial soils. Subsidence is expected to be less than 0.1 feet. These values are estimates only and exclude losses due to removal of vegetation or debris. Actual shrinkage and subsidence will depend on the types of earthmoving equipment used and should be determined during grading.

4.4.7 Trench/Wall Backfill

Utility trench backfill consisting of the on-site materials or imported soil, or wall backfill consisting of granular material should be mechanically compacted in lifts. The on-site clays should not be placed as retaining wall backfill unless the walls are specifically designed for increased pressure and the risk is acceptable to the Owner. Lift thickness should not exceed those values given in the "Placement and Compaction of Fills" section of this report. Moisture conditioning (drying) of the on-site soils will be required prior to re-use as backfill. Jetting or flooding of backfill materials should not be permitted. A representative of GPI should observe and test trench and wall backfill 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 two sacks of cement per cubic yard and have a maximum slump of 5 inches. When placed against retaining structures, the Project Structural Engineer should be consulted to determine the maximum lift height of the wet slurry.

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

4.4.8 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 to evaluate the overall compaction of the soils. Soils that do not meet minimum compaction requirements should be reworked and tested prior to placement of additional fill.

4.5 FOUNDATIONS

4.5.1 Foundation Type

The proposed parking structures may be supported on conventional isolated and/or continuous shallow spread footings with slab-on-grade floors subsequent to the installation of ground improvement under the footings to control static and seismic settlement.

The apartment buildings may be supported on spread footings or mat foundations subsequent to the installation of ground improvement to control seismic and static settlement. If shallow foundations are utilized, the design should include either a structural slab or ground improvement elements under the slab-on-grade. Isolated footings, not part of the mat foundations, associated with the apartment buildings, such as for entry features, should also be supported on ground improvement to control settlements.

Foundation design parameters should be provided by the design-build contractor constructing the ground improvement elements.

4.5.2 Shallow Foundations

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 2,000 pounds per square foot (psf) may be used for both continuous footings and isolated column footings for minor structures.

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

<u>Minimum Footing Width and Embedment</u>

For minor structures supported on properly compacted fill at-grade or competent natural soils, a static allowable net bearing pressure of 2,000 pounds per square foot may be used. The footings should have a minimum width of 18 inches and be embedded at least 24 inches below lowest adjacent grade for compacted fill and at least 36 inches below grade for competent natural soils.

Estimated Settlements

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

The above settlements should be included with the anticipated seismic settlement caused by liquefaction when evaluating the total settlement of the single-story buildings 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 footings and underlying soils and by passive soil pressures acting against the embedded sides of the footings. For frictional resistance, a coefficient of friction of 0.30 may be used for design for non-rammed aggregate pier supported footings (the ground improvement designer should provide the friction coefficient for the rammed aggregate pier supported footings). In addition, an allowable lateral bearing pressure equal to an equivalent fluid weight of 275 pounds per cubic foot may be used for footings. The allowable lateral bearing pressure values provided are based on the footings being poured tight against compacted fill soils or competent natural soils. The friction and lateral bearing values may be used in combination without reduction.

4.5.4 Foundation Concrete

Laboratory testing by HDR (Appendix C) indicates that the near surface soils exhibit a soluble sulfate content ranging of 111 mg/kg (2.5 percent by weight). For the 2022 CBC, foundation concrete should conform to the requirements outlined in ACI 318, Section 4.3, for very severe levels of soluble sulfate exposure from the on-site soil (Category S3). Chloride levels in the sample of the upper soils tested were 86 mg/kg, which is considered to be low (Category C1).

4.5.5 Footing Excavation Observation

Prior to placement of steel and concrete, a representative of GPI should observe and approve footing excavations including footings overlying ground improvement and mat foundations bottoms.

4.6 SLABS-ON-GRADE

The slab-on-grade floors (including parking structures) should be supported on at least 12 inches of granular (sandy), non-expansive soils (Expansion Index less than 20). As previously discussed in Section 4.4.2, if a structural slab or mat foundation is utilized for the building the need for non-expansive soils in the upper 12 inches may be waived.

The non-expansive granular fill should be compacted as discussed in the "Placement and Compaction of Fill" section. Based on our explorations, granular, non-expansive soils are limited on site and may need to be imported to the project site.

A vapor/moisture retarder should be placed under slabs that are to be covered with moisture-sensitive floor coverings (parquet, vinyl tile, etc.) or will be storing moisture sensitive supplies. Currently, common practice is to use a 15 mil polyolefin product such as Stego Wrap for this purpose. Whether to place the concrete slab directly on the vapor barrier or place a clean sand layer between the slab and vapor barrier is a decision for the Project Architect, 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 soils at the site are not suitable for this purpose. This layer should be nominally compacted using light equipment. The sand placed over the vapor retarder should only be slightly moist. If the sand gets wet (for example as a result of rainfall or excessive moistening) it must be allowed to dry prior to placing concrete. Care should be taken to avoid infiltration of water into the sand layer after placement of the concrete slab, such as at slab cut-outs and other exposures. 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 to cement ratio for the concrete used for the floor slab, effective sealing of joints and edges (particularly at pipe penetrations), as well as excess moisture in the concrete. The manufacturer of the floor coverings should be consulted for establishing acceptable criteria for the condition of floor surface prior to placing moisture-sensitive floor coverings.

4.7 LATERAL EARTH PRESSURES

We assume that retaining walls for the parking structure ramps are planned for the project. The following recommendations are provided for walls up to 12 feet in height.

We recommend that the walls be backfilled with imported non-expansive, granular (sandy) soils. The limits of select fill should extend 2 feet beyond bottom of wall and upwards to at a ¾:1 projection (horizontal:vertical). However, we understand that on-site soils are being considered for backfill of the ramp walls in the parking structure. If on-site soils are used, active earth pressures for these soils are provided below. The Owner should understand that besides greater earth pressures required in the design, the clayey soils have poor drainage characteristics. The ramp walls should be damp-proofed to help mitigate moisture and sulfates impacting the wall face unless the risk is acceptable to the Owner.

Active earth pressures can be used for designing walls that can yield at least 1-inch laterally in 10 feet of wall height under the imposed loads. For level backfill comprised of drained, imported granular soils (no more than 40 percent passing No. 200 U.S. standard sieve), the magnitude of active pressures are equivalent to the pressures imposed by a fluid weighing 35 pounds per cubic foot (pcf). For level backfill comprised of drained, on-site clay soils, the magnitude of active pressures are equivalent to the pressures imposed by a fluid weighing 45 pcf.

At-rest pressures should be used for restrained walls that remain rigid enough to be essentially non-yielding. At-rest pressures for drained, imported granular soils are equivalent to the pressures imposed by a fluid weighing 55 pounds per cubic foot. At-rest pressures for drained, on-site clay soils are equivalent to the pressures imposed by a fluid weighing 65 pounds per cubic foot.

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

The recommended pressures are based on the assumption that the supported earth will be fully drained, preventing the build-up of hydrostatic pressures. For traditional backfilled retaining walls, a drain consisting of perforated pipe and gravel wrapped in filter fabric should be used. One cubic foot of rock should be used for each lineal foot of pipe. The fabric (non-woven filter fabric, Mirafi 140N or equivalent) should be lapped at the top. We prefer pipe and gravel drains to weep holes to avoid potential for constant flow of surface water in front of the wall. If acceptable to the Structural Engineer, drainage can be omitted for ramp walls within parking structure where the ramp walls are not exposed to the exterior of the parking structure.

Walls subject to surcharge loads should be designed for an additional uniform lateral pressure equal to one-third and one-half the anticipated surcharge pressure for unrestrained and restrained walls, respectively. In addition to the recommended earth pressure, the upper 10 feet of the walls adjacent to the traffic should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pound per square foot surcharge due to normal traffic. If traffic is kept at least 10 feet from the walls, the traffic surcharge may be neglected.

The Structural Engineer should indicate on the plans the type of wall backfill recommended based on the earth pressures used for the design. Light wall footings should be designed as discussed in the ″Shallow Foundations″ section.

4.8 CORROSIVITY

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

4.9 DRAINAGE

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

4.10 STORMWATER 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 upon our subsurface investigation and laboratory testing, the subsurface soils underlying the site consist predominantly of clays and silts which have poor infiltration characteristics and are not suitable to accept infiltration. In addition, groundwater was encountered in the upper 9 feet of the soil profile, historical high groundwater is a depth of 5 feet, and the site is located in a liquefaction zone. For the above reasons, we do not recommend stormwater infiltration for the subject site.

4.11 EXTERIOR CONCRETE AND MASONRY FLATWORK

Exterior concrete and masonry flatwork should be supported on non-expansive, compacted granular fill (E.I. ≤ 20). We recommend 12 inches of non-expansive fill under the exterior concrete surrounding the building.

The use of the on-site fine-grained soils directly under the slab subgrade or lesser thicknesses of non-expansive material as discussed should not be permitted unless differential heave is tolerable. This includes areas such as exterior sidewalks, stamped concrete, non-traffic pavement, and pavers. There are ample amounts of non-expansive material available on-site. Testing during grading should be performed to ensure the expansion potential of the material is less than an EI of 20.

Prior to placement of the non-expansive material under the concrete, the subgrade should be prepared as recommended in the "Subgrade Preparation" section of this report. It is imperative that the on-site silts are not allowed to dry-out during construction.

4.12 PAVED AREAS

Preliminary pavement design has been based on an R-value of 20 based upon laboratory testing of the near-surface soils at the site. The California Division of Highways Design Method was used for design of the recommended preliminary pavement sections. Final pavement design should be based on R-value testing performed near the conclusion of rough grading. The following pavement sections are recommended for planning purposes only.

PAVEMENT SECTIONS

PAVEMENT AREA	ASSUMED TRAFFIC INDEX	SECTION THICKNESS (inches)	
		Asphalt Concrete	**Aggregate Base Course**
Auto Parking	4	3	5
Circulation Drives (no trucks)	5	3	6
Truck Driveways	6	3	8
		Portland Cement Concrete	**Aggregate Base Course**
Auto Parking	4	6.0	----
Circulation Drives (no trucks)	5	6.0	----
Truck Driveways	6	6.5	----

If vehicular pavers are to be used for the project, the paver and leveling sand may be supported on the thickness of aggregate base shown for the appropriate traffic index. Pavers for vehicular traffic should be a minimum of 3⅛ inches (80 mm) thick.

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

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

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

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

4.13 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 all fills and subgrade preparation, as well as foundation construction including Geopier installation. 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 Holland Acquisition Co., LLC and their consultants in designing the proposed development. The report is not intended to be suitable for reuse on extensions or modifications of the project or for use on any project other than the currently proposed development as it may not contain sufficient or appropriate information for such uses. If this report or portions of this report are provided to contractors or included in specifications, it should be understood that they are provided for information only. This report cannot be utilized by another entity without the express written permission of GPI. This report is an instrument of our services and remains the property of GPI.

Soil deposits may vary in type, strength, and many other important properties between points of exploration due to non-uniformity of the geologic formations or to man-made cut and fill operations. While we cannot evaluate the consistency of the properties of materials in areas not explored, the conclusions drawn in this report are based on the assumption that the data obtained in the field and laboratory are reasonably representative of field conditions and are conducive to interpolation and extrapolation.

Furthermore, our recommendations were developed with the assumption that a proper level of field observation and construction review will be provided during grading, excavation, and foundation construction by GPI. If field conditions during construction appear to be different than is indicated in this report, we should be notified immediately so that we may assess the impact of such conditions on our recommendations. If construction phase services are performed by others they must accept full responsibility (as Project Geotechnical Engineer) for all geotechnical aspects of the project including this report.

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

Respectfully submitted,
Geotechnical Professionals Inc.

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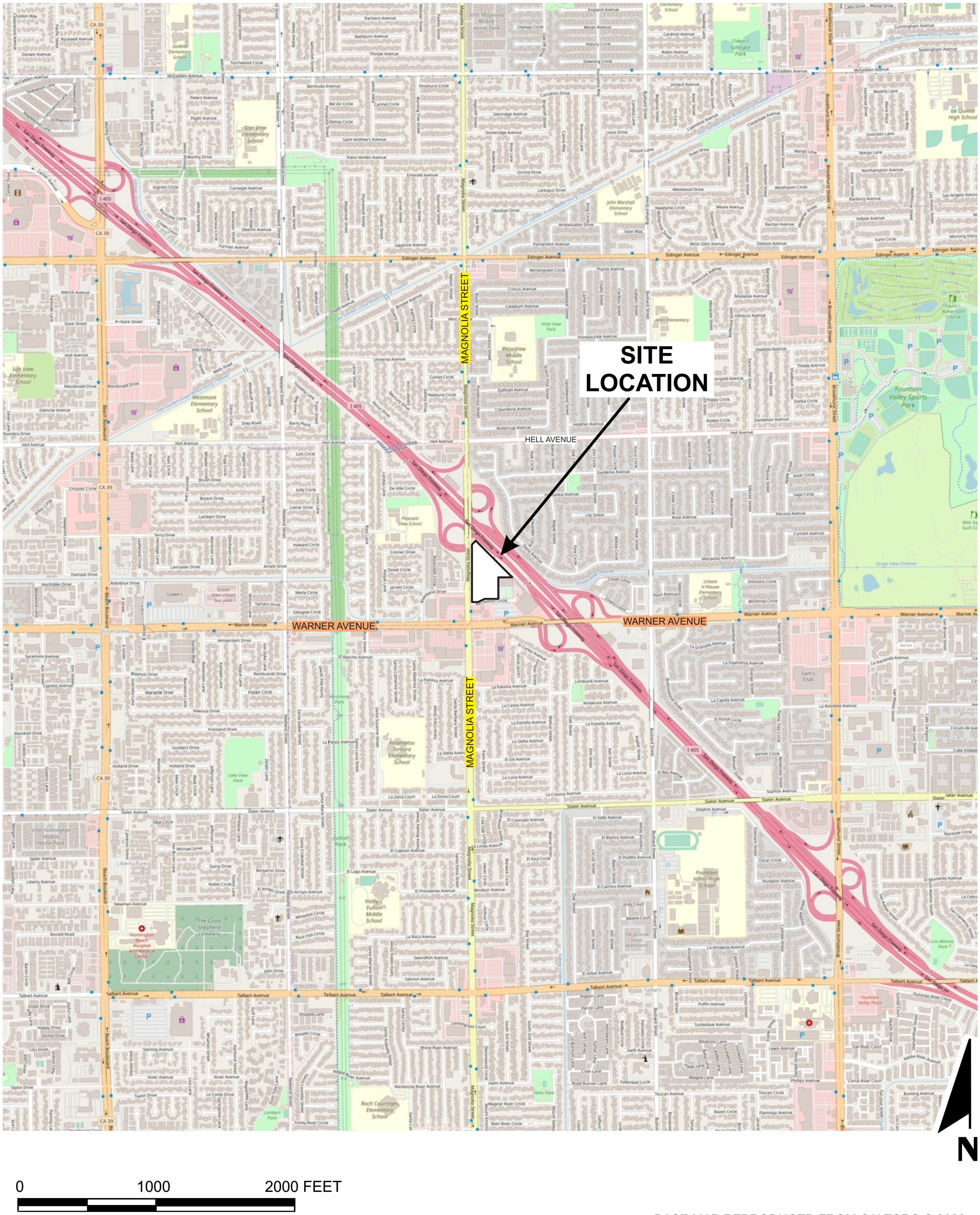

BASE MAP REPRODUCED FROM CALTOPO © 2022

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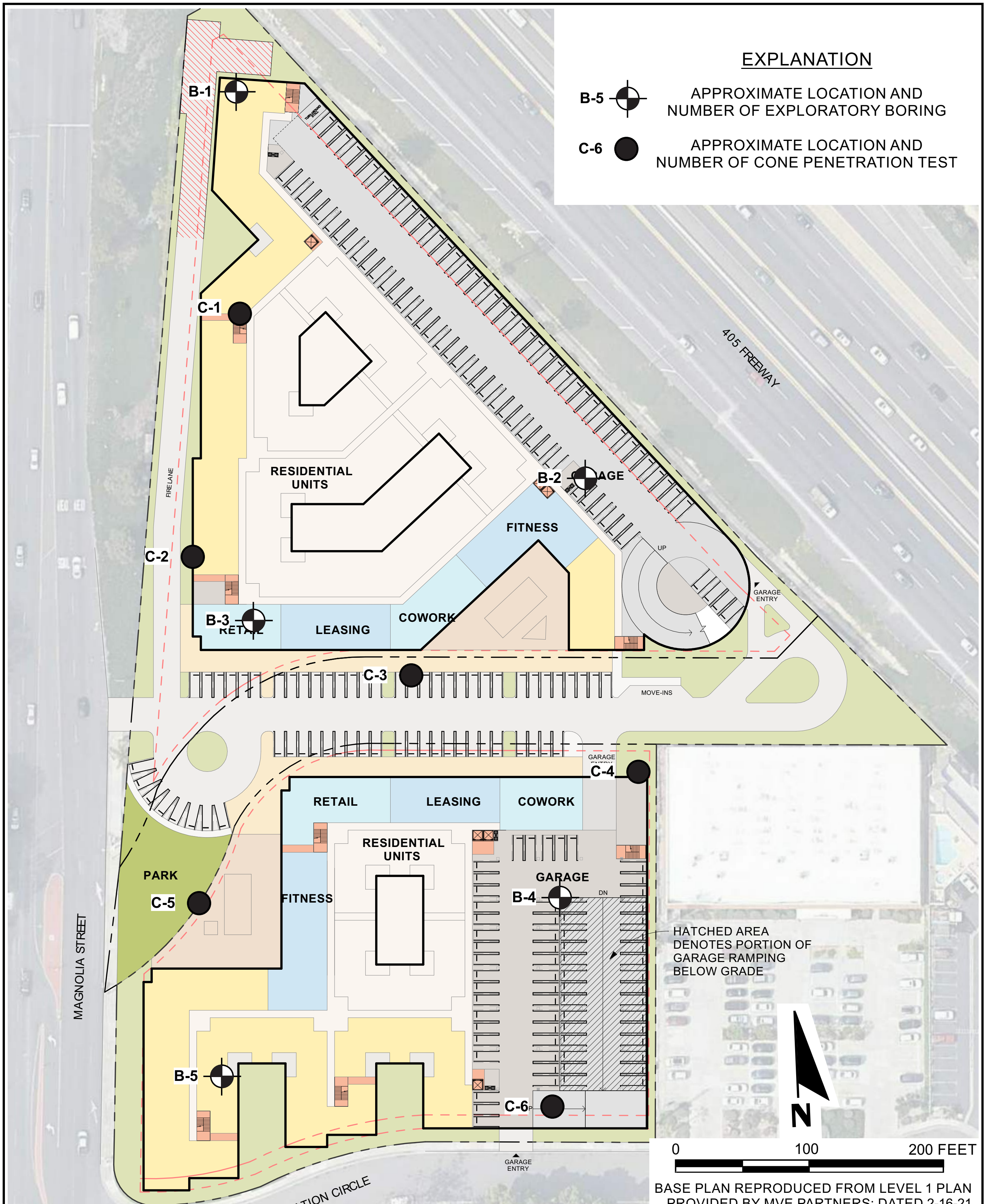
HOLLAND MAGNOLIA

GPI PROJECT NO.: 3033.1I

SCALE: 1" = 1000'

SITE LOCATION MAP

FIGURE 1



GPI GEOTECHNICAL PROFESSIONALS, INC.

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GPI PROJECT NO.: 3033.1I | SCALE: 1" = 100'

SITE PLAN

FIGURE 2

APPENDIX A

APPENDIX A

CONE PENETRATION TESTS

The subsurface conditions were investigated by performing six Cone Penetration Tests (CPTs) at the site. The CPT's were advanced to depths ranging from 86 to 100 feet below existing grades. The locations of the CPTs are shown on the Site Plan, Figure 2.

The Cone Penetration Test consists of pushing a cone-tipped probed into the soil deposit while simultaneously recording the cone tip resistance and side friction resistance of the soil to penetration (refer to Figure A-1). The CPTs described in this report were conducted in general accordance with ASTM specifications (ASTM D5778) using an electric cone penetrometer.

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

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

Computer plots of the reduced CPT data acquired for this investigation is presented in Figures A-2 to A-7 of this appendix. The field testing and computer processing was performed by Kehoe Testing and Engineering under subcontract to Geotechnical Professionals Inc. (GPI). The interpreted soils descriptions were prepared by GPI.

A seismic cone penetration test provided shear wave velocity measurements of the soil profile. A standard cone penetrometer is equipped with two sets of geophones located approximately 2 feet apart on the cone penetrometer. At 10-foot intervals, a shear wave source is activated at the ground surface using an air-actuated hammer. A seismograph measures the travel time of the shear wave detected at each set of geophones. The time difference provides the velocity of the shear wave in the layer between the two geophone sets.

Seismic cone penetration tests was performed at CPT C-2. The cone penetration test at location C-2 was performed to 100 feet below existing grade and the subsequent data was used in order to estimate the average shear wave velocity for the upper 100 feet of soil profile. Table A-1 provides the shear wave velocity from the surface and the interval of soil between the geophones.

The CPT locations were laid out in the field by measuring from existing site features. Upon completion, the un-caved portions of the CPT holes were backfilled with bentonite chips. Ground surface elevations at the exploration locations were estimated from google earth.

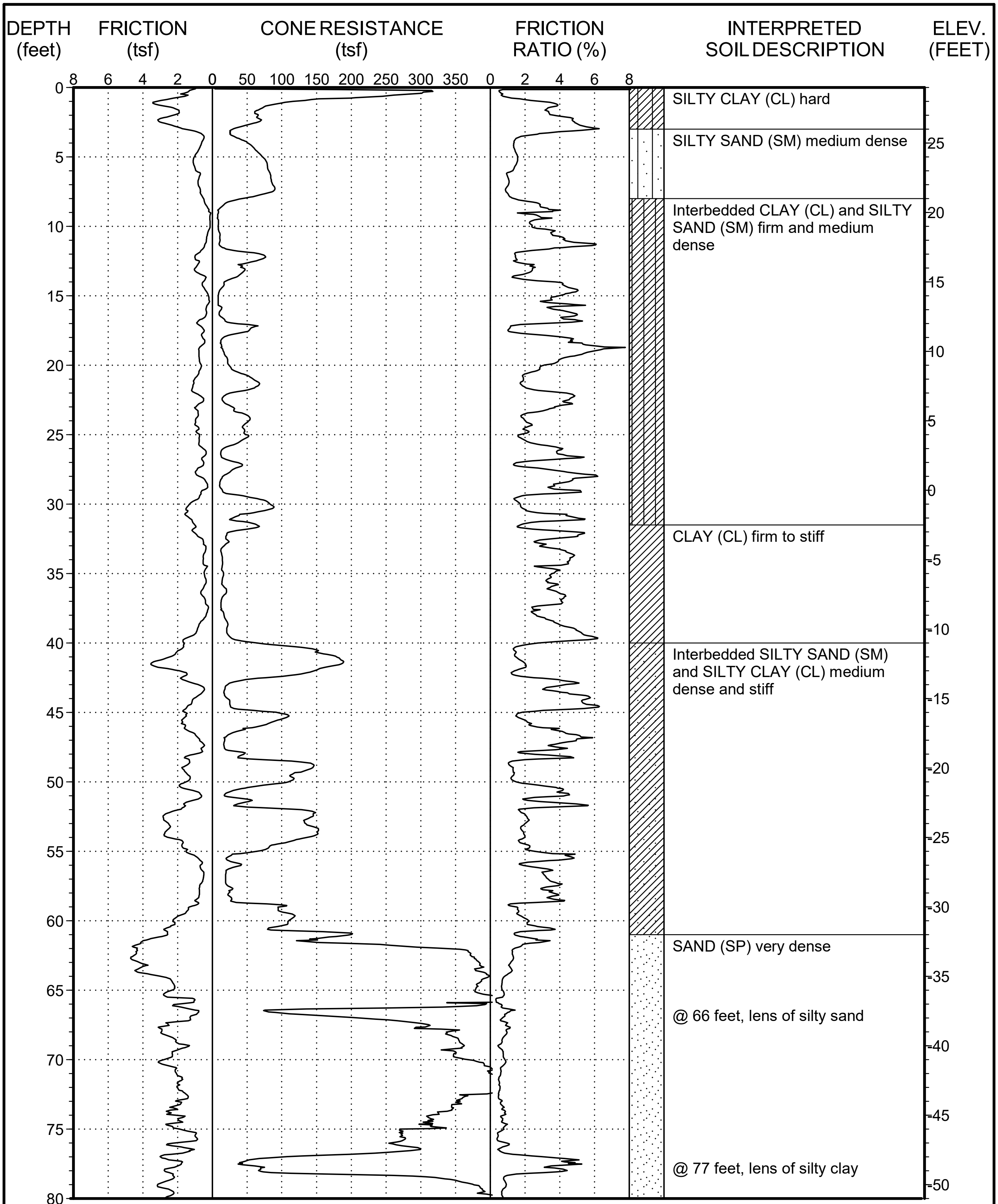
TABLE A-1
SEISMIC SHEAR WAVE VELOCITY MEASUREMENTS

Geotechnical Professionals Inc.
Holland Magnolia - Proposed Mixed Use Apartments
Fountain Valley, CA

CPT Shear Wave Measurements

Location	Tip Depth (ft)	Geophone Depth (ft)	Travel Distance (ft)	S-Wave Arrival (msec)	S-Wave Velocity from Surface (ft/sec)	Interval S-Wave Velocity (ft/sec)
CPT C-2	10.01	9.01	9.01	16.60	543	
	20.08	19.08	19.08	36.06	529	517
	30.05	29.05	29.05	53.54	543	570
	39.99	38.99	38.99	69.60	560	619
	50.03	49.03	49.03	83.60	586	717
	60.01	59.01	59.01	97.54	605	716
	70.05	69.05	69.05	108.32	637	931
	79.99	78.99	78.99	119.68	660	875
	90.03	89.03	89.03	128.80	691	1101
	100.03	99.03	99.03	137.88	718	1101

Average Shear Wave Velocity in Upper 100 feet: 716 ft/sec



Date performed:3-31-21

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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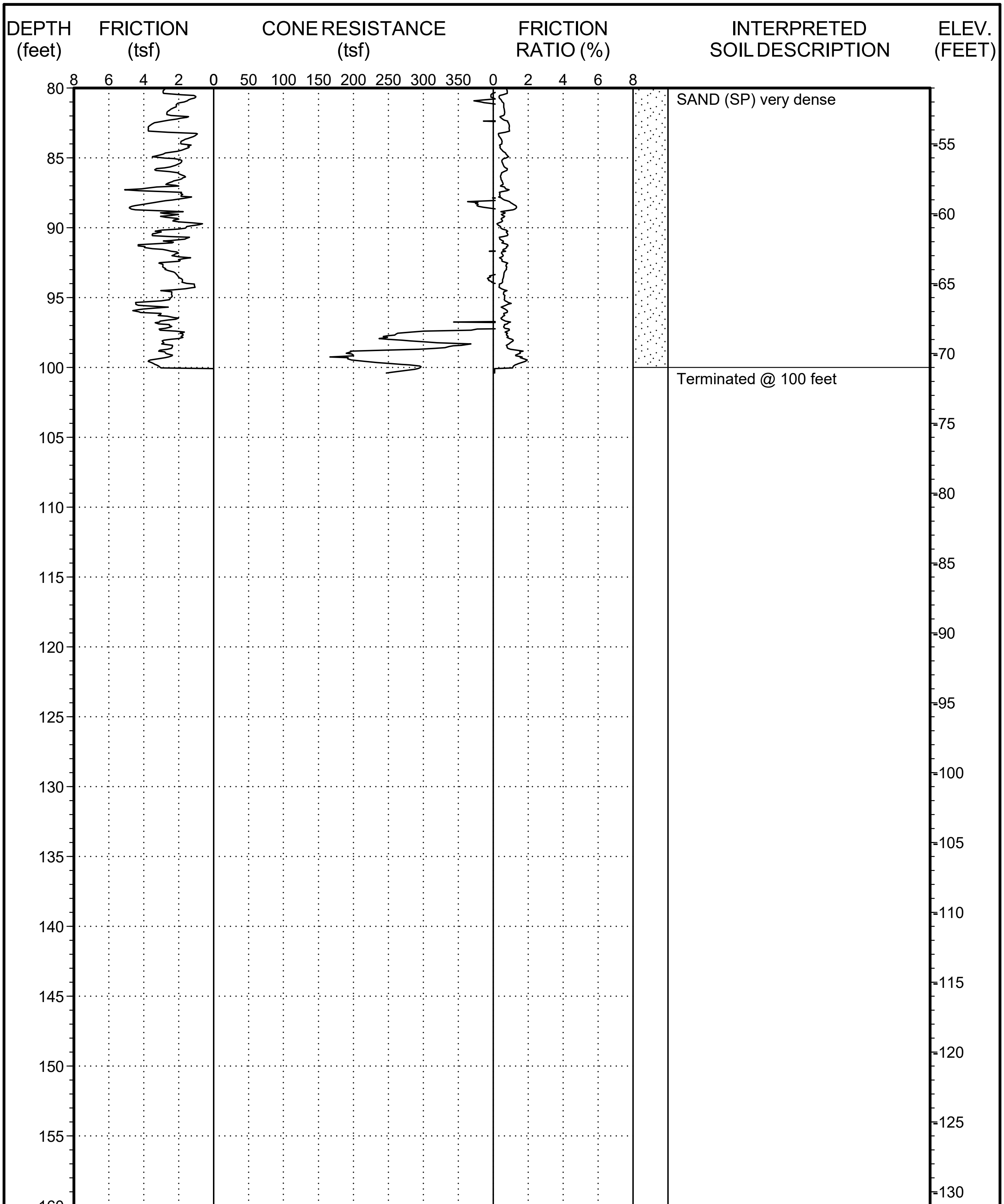
LOG OF CPT NO. C-1

FIGURE A-2



Date performed:3-31-21

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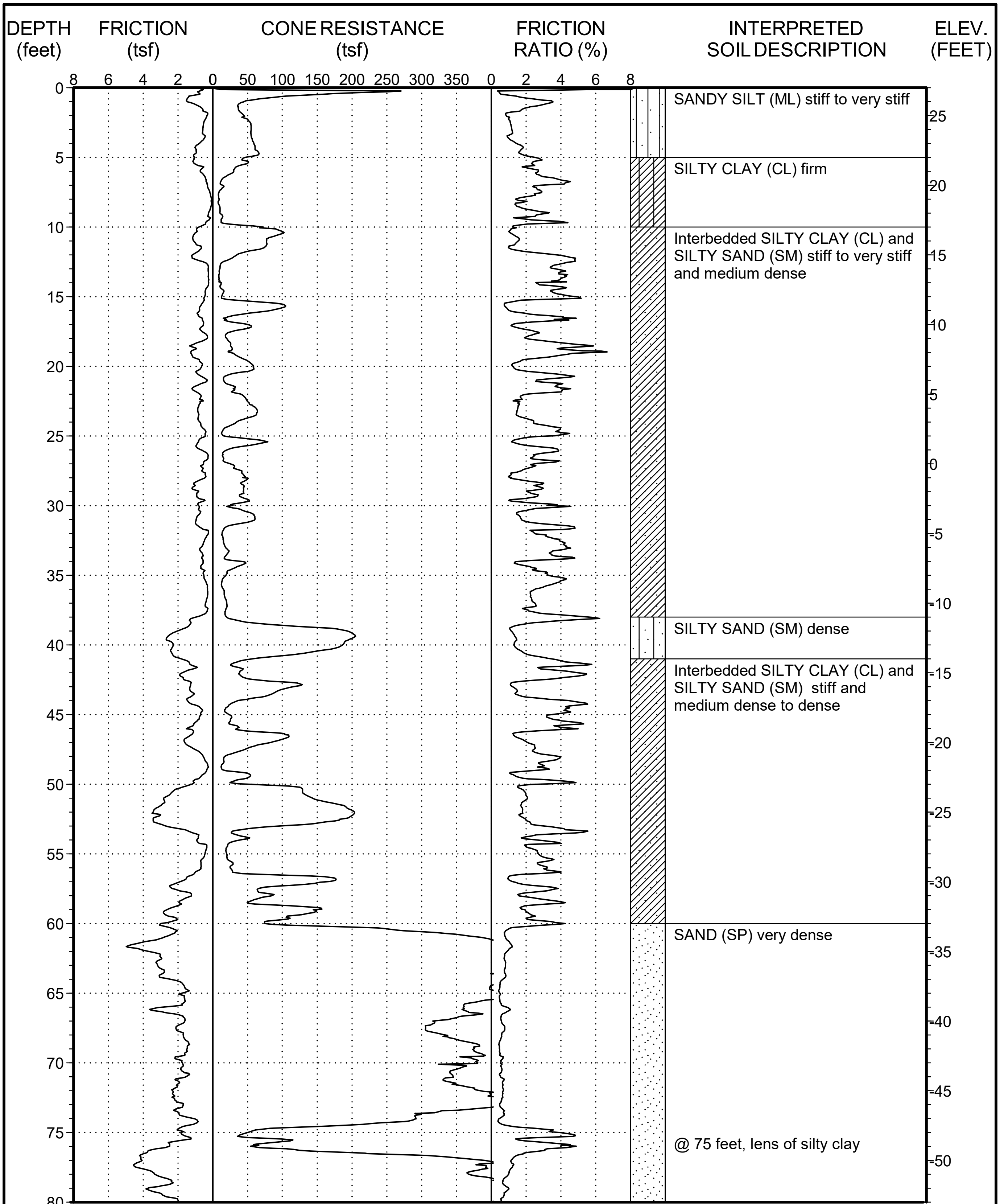
PROJECT NO.: 3033.1I

HOLLAND MAGNOLIA

LOG OF CPT NO. C-1

FIGURE A-2



Date performed:3-31-21

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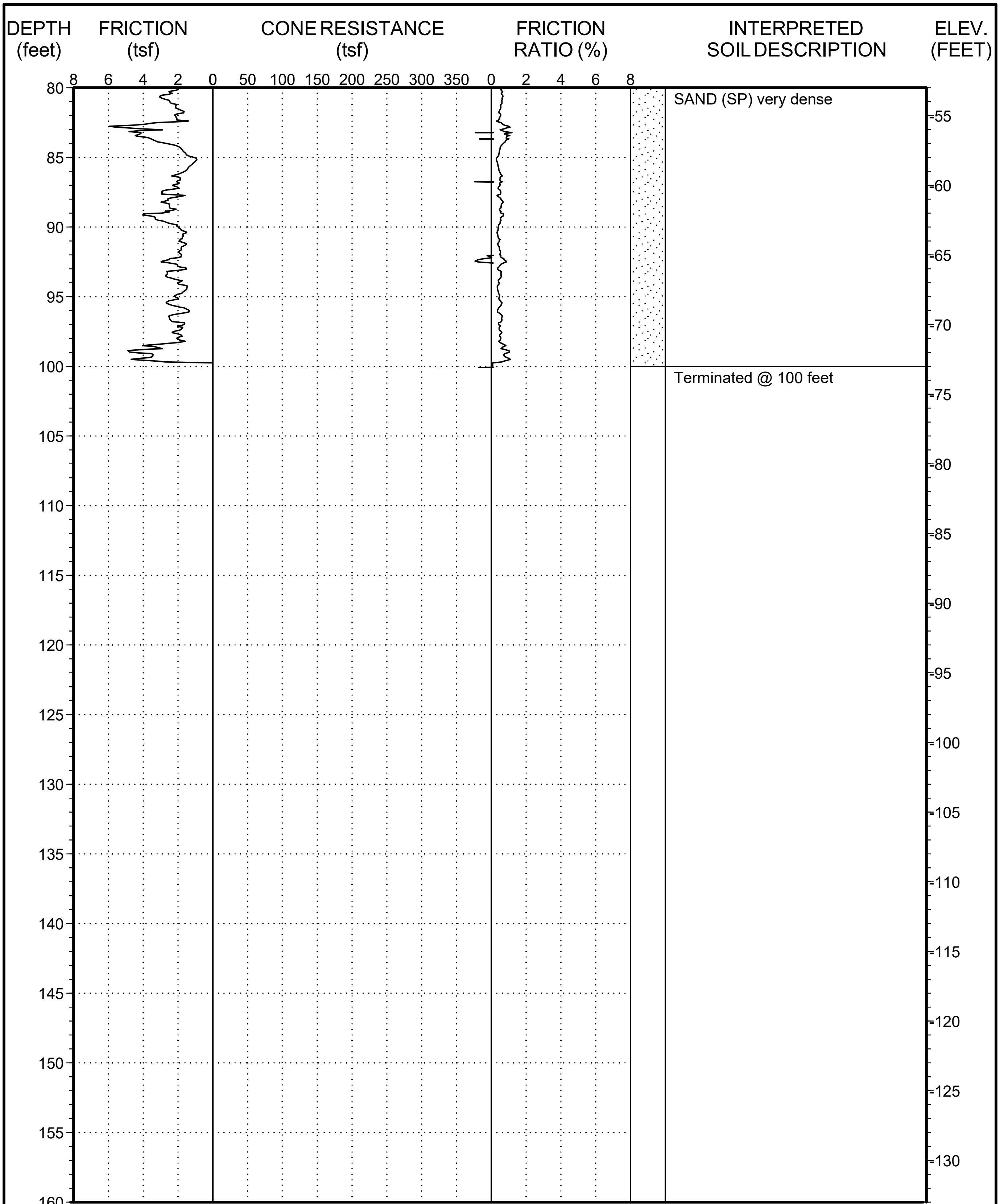
LOG OF CPT NO. C-2

FIGURE A-3



Date performed:3-31-21

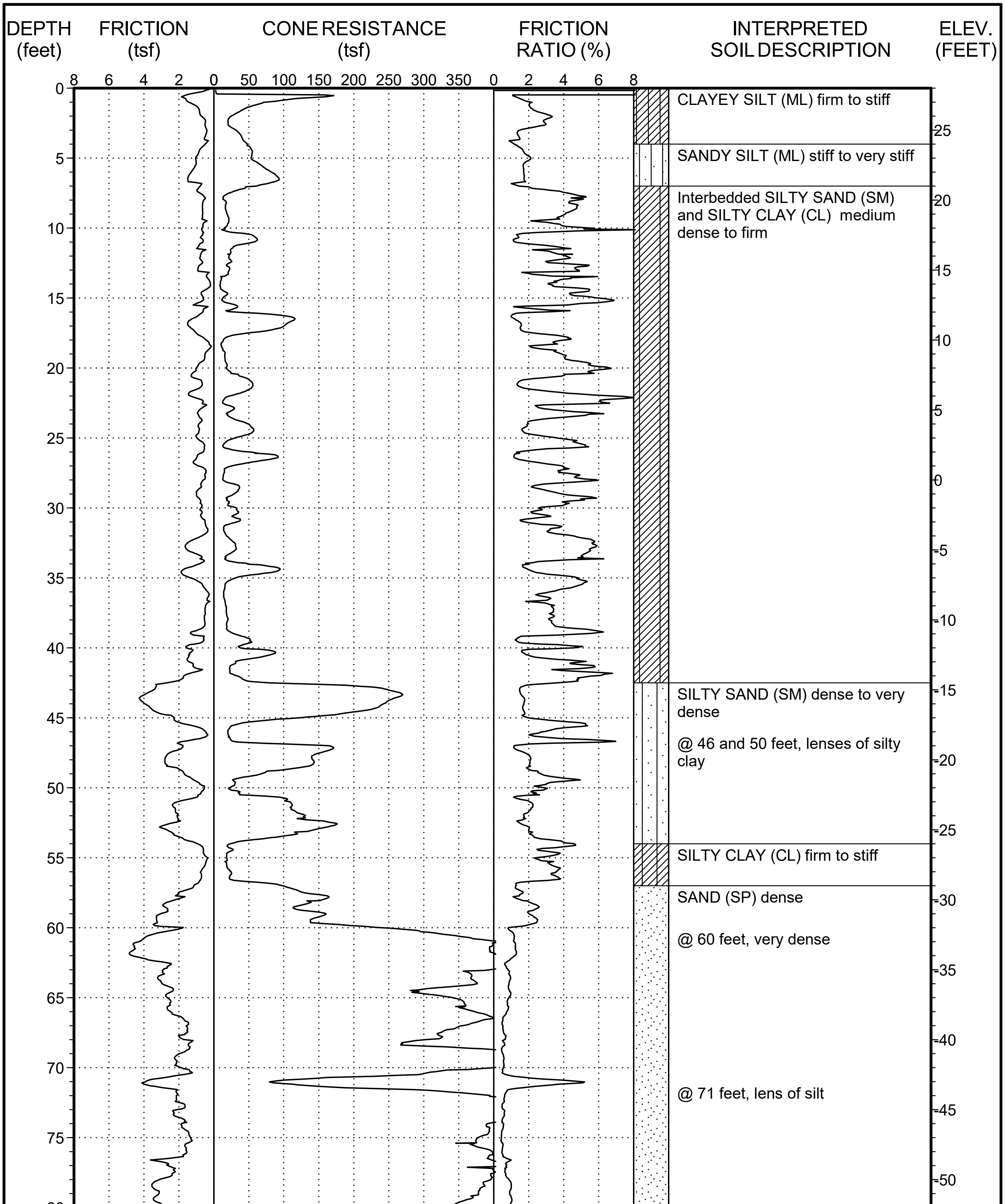
This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance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

PROJECT NO.: 3033.1I

HOLLAND MAGNOLIA

LOG OF CPT NO. C-2

FIGURE A-3



Date performed:3-31-21

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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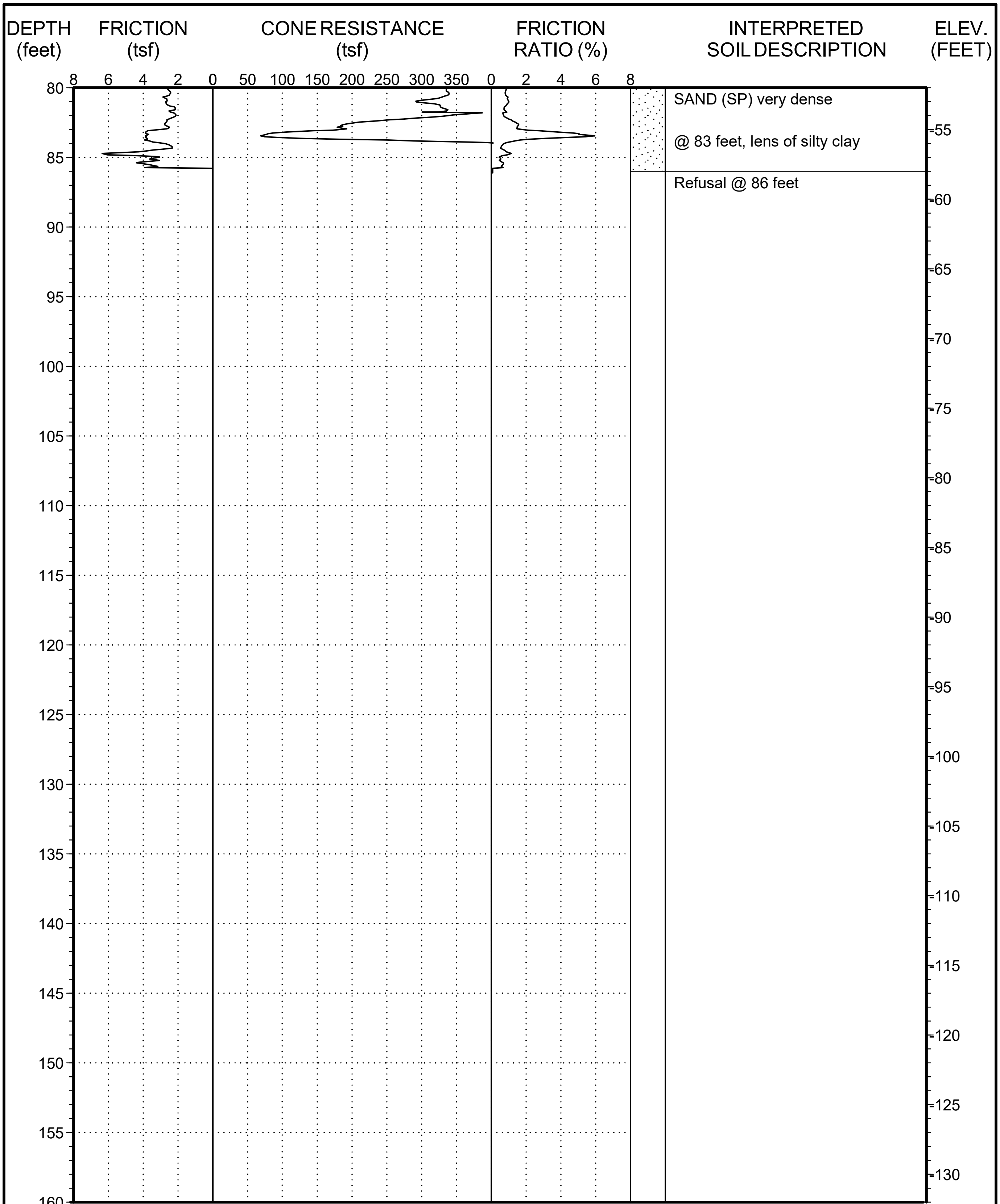
LOG OF CPT NO. C-3

FIGURE A-4



Date performed:3-31-21

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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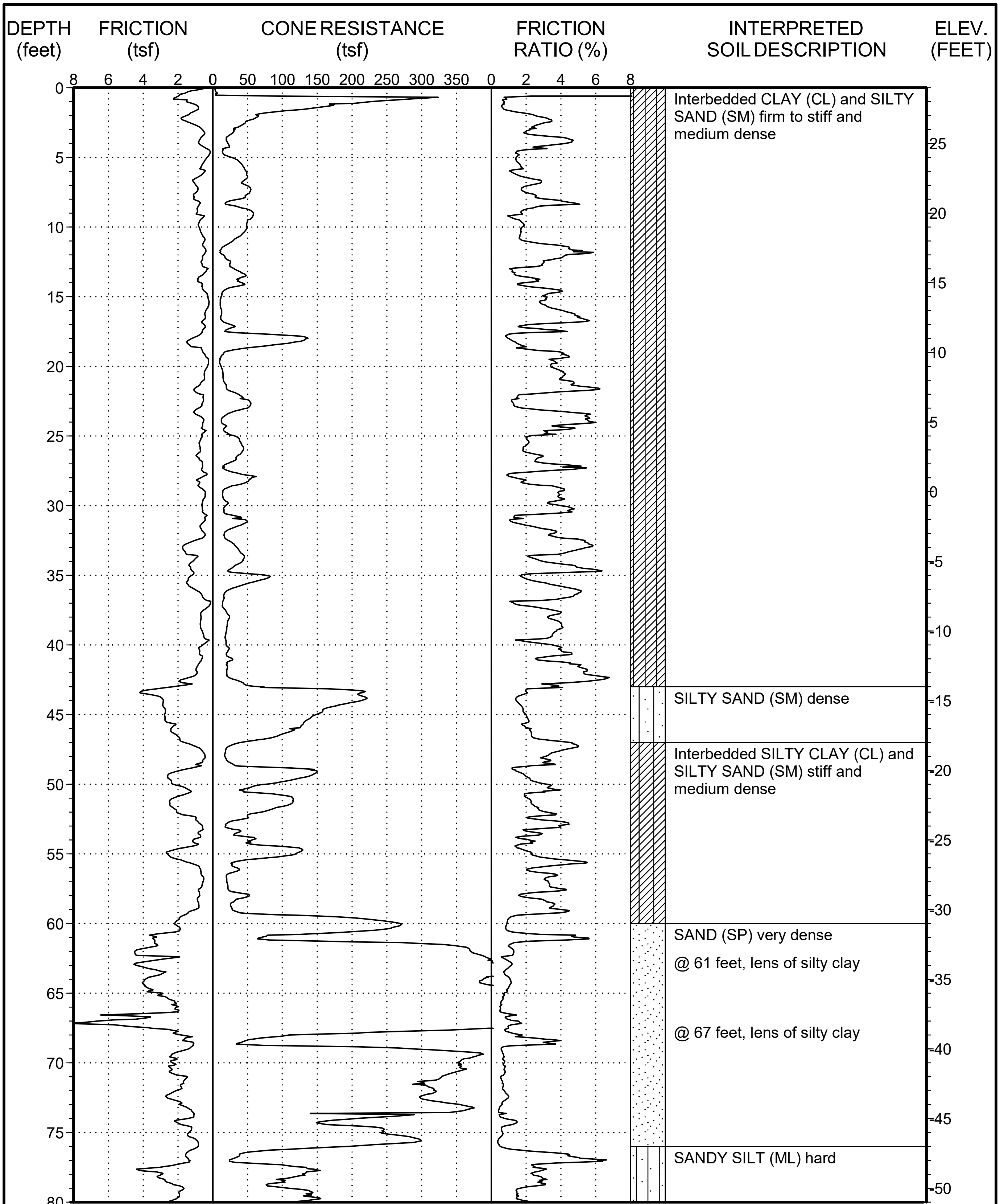
HOLLAND MAGNOLIA

LOG OF CPT NO. C-3

FIGURE A-4



Date performed:3-31-21

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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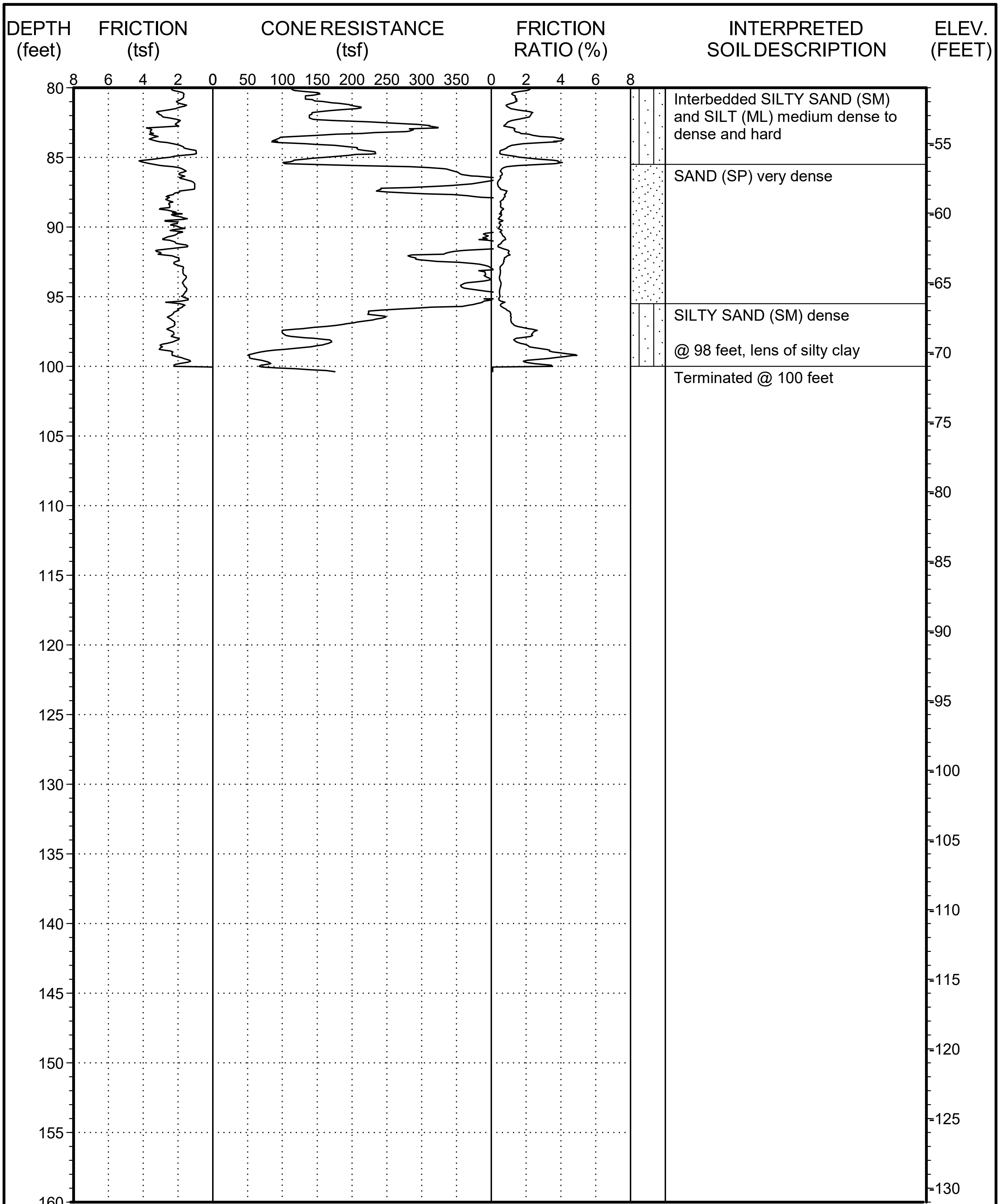
LOG OF CPT NO. C-4

FIGURE A-5



Date performed:3-31-21

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PROJECT NO.: 3033.1I

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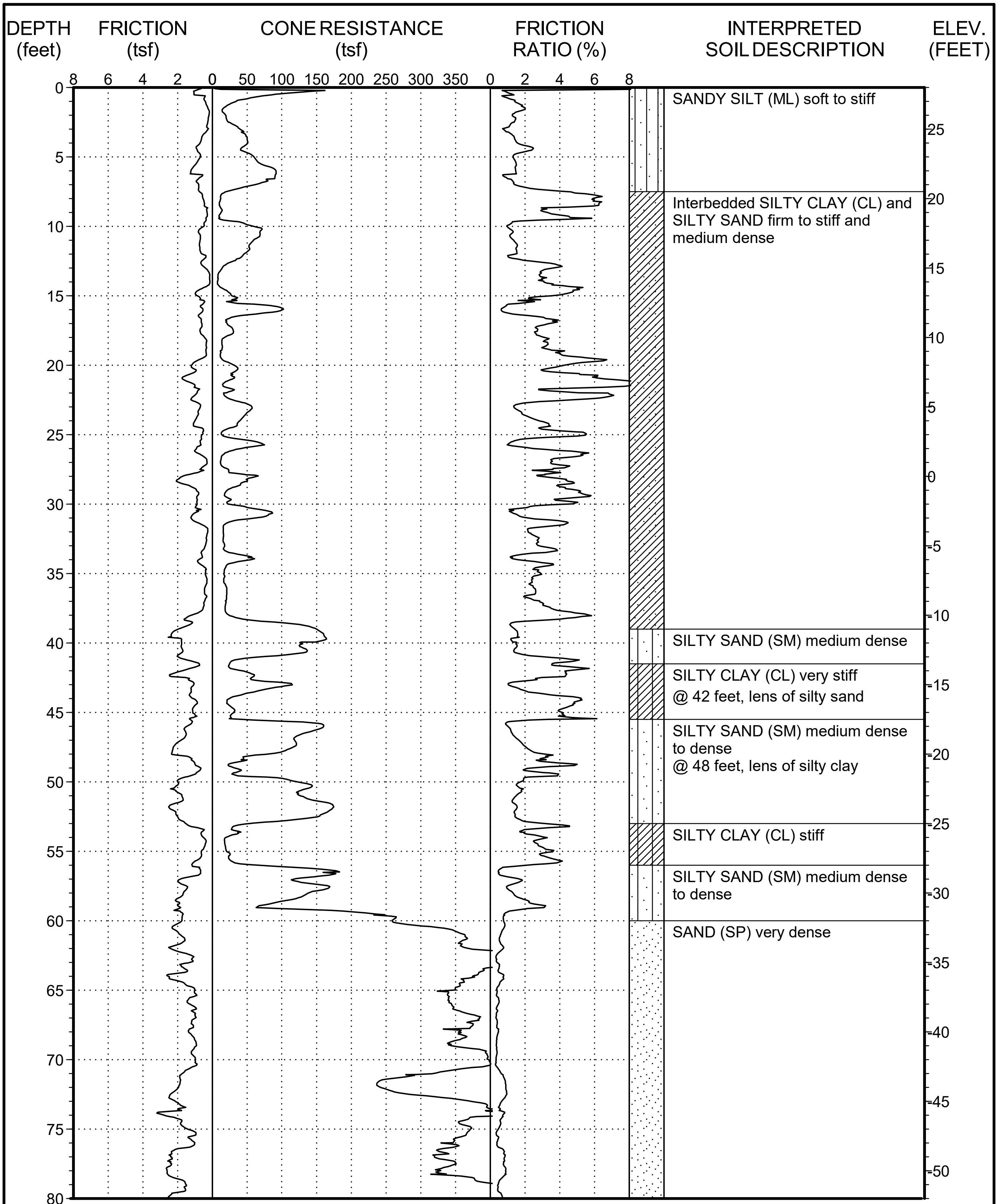
LOG OF CPT NO. C-4

FIGURE A-5



Date performed:3-31-21

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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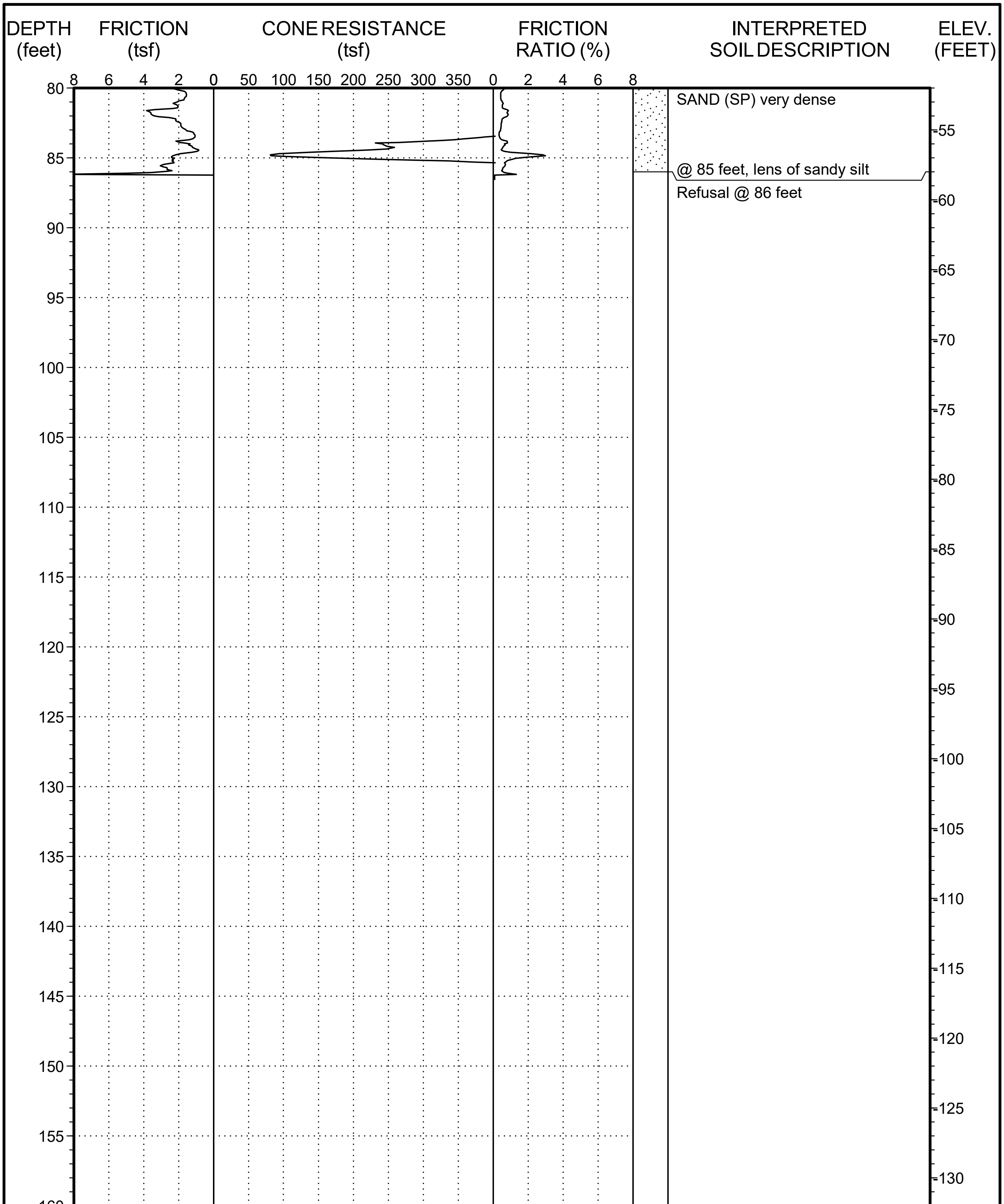
PROJECT NO.: 3033.1I

HOLLAND MAGNOLIA

LOG OF CPT NO. C-5

FIGURE A-6



Date performed: 3-31-21

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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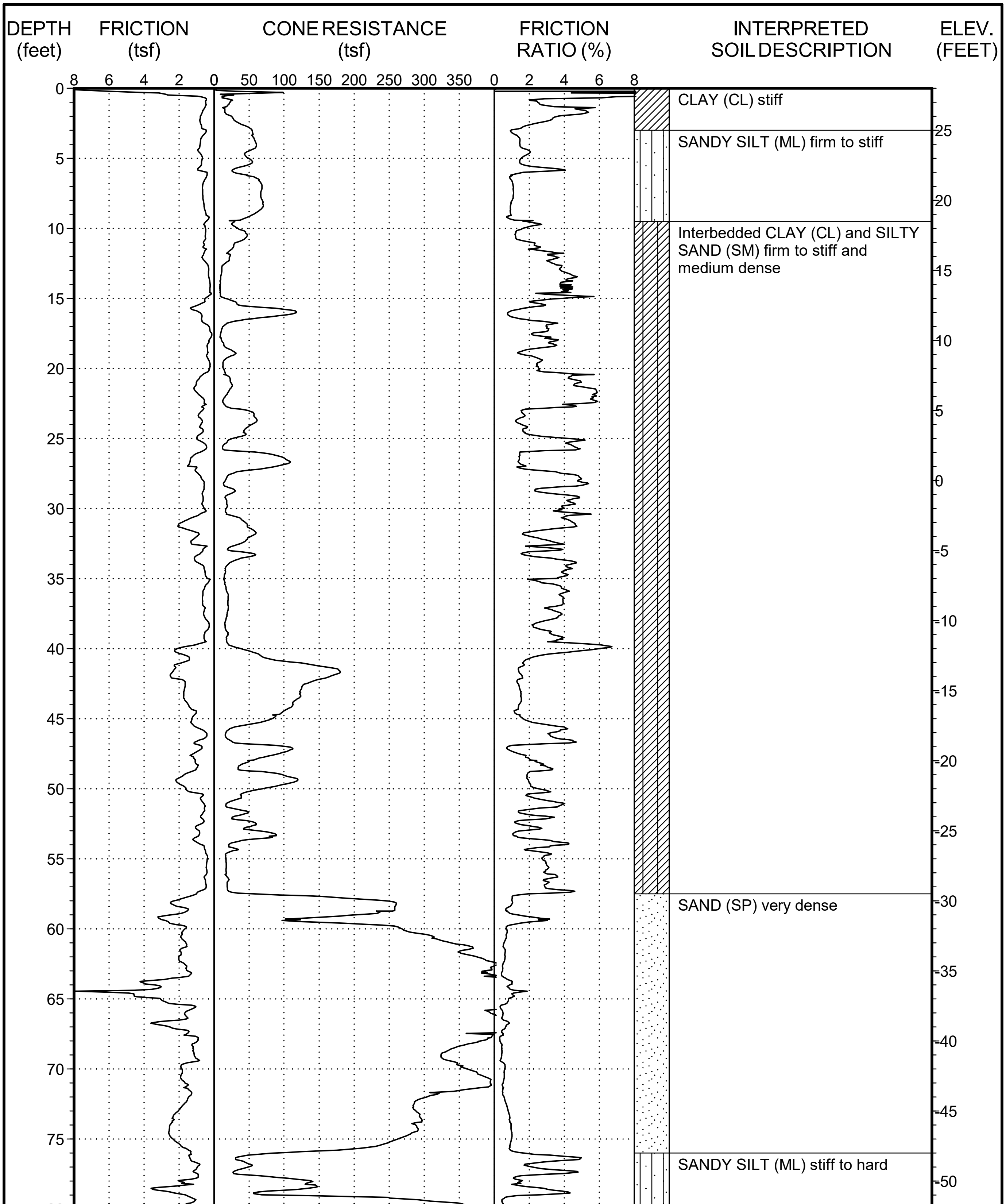
HOLLAND MAGNOLIA

LOG OF CPT NO. C-5

FIGURE A-6



Date performed:3-31-21

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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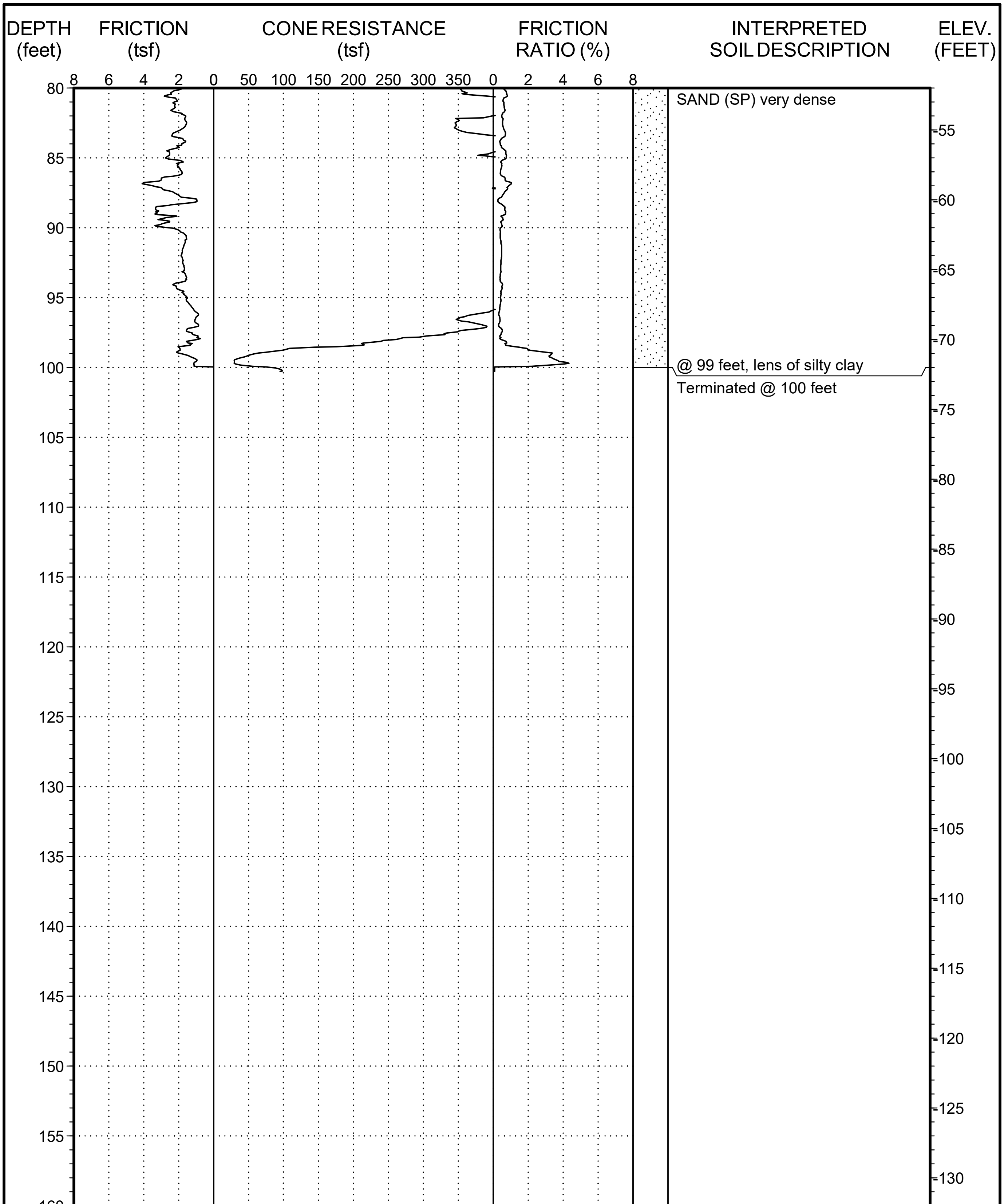
PROJECT NO.: 3033.1I

HOLLAND MAGNOLIA

LOG OF CPT NO. C-6

FIGURE A-7



Date performed:3-31-21

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

APPENDIX B

APPENDIX B

EXPLORATORY BORINGS

The subsurface conditions at the site were investigated by drilling and sampling five exploratory borings. The borings were advanced to depths of 11 to 60 feet below the existing ground surface. The locations of the explorations are shown on the Site Plan, Figure 2.

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

The field explorations for the investigation were performed under the continuous technical supervision of GPI's representative, who visually inspected the site, maintained detailed logs of the borings, classified the soils encountered, and obtained relatively undisturbed samples for examination and laboratory testing. The soils encountered in the borings were classified in the field and through further examination in the laboratory in accordance with the Unified Soils Classification System. Detailed logs of the borings are presented in Figures B-1 and B-5 in this appendix.

The boring locations were laid out in the field by measuring from existing site features. Ground surface elevations at the exploration locations were estimated using Google Earth.

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
			B	0	Fill: **SILTY SAND (SM)** brown, moist	35
11.3	107	33	D		Natural: **SILTY SAND (SM)** brown, moist, medium dense	
13.7	106	13	D	5	**SANDY SILT (ML)** brown, moist, stiff	30
22.5	94	18	D		**SILTY SAND (SM)** brown, wet, stiff	
30.4	87	13	D	10	@ 10 feet, loose	25
28.1	93	16	D	15	@ 15 feet, medium dense	20
26.6 37.7	93 85	7	D	20	@ 20 feet, loose **CLAY (CL)** brown, wet, firm	15
46.2 28.5	78 89	10	D	25	**ELASTIC SILT (MH)** brown, wet, firm	10
37.1	81	7	D	30		5
37.7	80	8	D	35	**CLAY (CL)** brown, wet, firm	0

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

DATE DRILLED:
9-18-23

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
10

GPI

PROJECT NO.: 3033.1I
HOLLAND MAGNOLIA

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.	
	22.7	102	9	D	40	@ 40 feet, with sandy clay	-5
	25.9	97	13	D	45	**SANDY CLAY (CL)** brown, wet, stiff	-10
						SANDY SILT (ML) brown, wet, stiff	
			21	D	50	@ 50 feet, no recovery	-15
	29.7	94	38	D	55	**SILTY SAND (SM)/ SANDY SILT (ML)** brown, wet, medium dense/ very stiff	-20
	36.8	81	30	D		**CLAY (CL)** brown, wet, very stiff	
					60	**SANDY CLAY (CL)**brown, wet, very stiff	
						Total Depth 60 feet	

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

DATE DRILLED:
9-18-23

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
10

GPI

PROJECT NO.: 3033.1I

HOLLAND MAGNOLIA

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.	
				0	Fill: **SILTY SAND (SM)** brown, moist	35
9.8	101	16	D		@ 2 feet, medium dense	
12.2	95	18	D	5	Natural: **SILTY SAND (SM)** brown, moist, medium dense	30
29.8	85	6	D		**SANDY SILT (ML)** brown, very moist, soft to firm	
33.3	86	7	D	10	**CLAYEY SILT (ML)** brown, wet, firm	25
45.6	72	5	D	15	**CLAY (CL)** gray, very moist, soft	20
27.4	93	7	D	20	**SANDY CLAY (CL)** gray, wet, firm	15
34.4	84	10	D	25	**CLAYEY SILT (ML)** gray, wet, firm	10
37.2	81	12	D	30	**CLAY (CL)** gray, wet, firm to stiff	5
32.3	86	14	D	35	**CLAYEY SILT (ML)** gray, wet, stiff	0

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

DATE DRILLED:
9-18-23

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
10

GPI

PROJECT NO.: 3033.1I

HOLLAND MAGNOLIA

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)
	23.1	102	15	D	40	**SANDY SILT (ML)** gray, wet, stiff	-5
	28.2	91	20	D	45	**SILTY SAND (SM)** gray, wet, medium dense	-10
	35.4	83	13	D	50	**SANDY SILT (ML)** gray, wet, stiff	-15
						CLAYEY SILT (ML) gray, wet, stiff	
	32.8	84	43	D	55	**SANDY SILT (ML)** gray, wet, very stiff	-20
						CLAY (CL) gray, wet, very stiff	
	16.9	108	50/5"	D		**SILTY SAND (SM)** gray, wet, very dense	
					60	Total Depth 60 feet	

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

DATE DRILLED:
9-18-23
EQUIPMENT USED:
8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft):
10

GPI

PROJECT NO.: 3033.1I
HOLLAND MAGNOLIA

LOG OF BORING NO. B-2

FIGURE B-2

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
						This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
					0	2-Inch AC	
						Fill: **SILTY SAND (SM)** brown, moist, with gravel	
	14.4	102	28	D		@ 2 feet, medium dense	
						Natural: **SANDY SILT (ML)** brown, moist, very stiff	25
	12.7	87	22	D	5	**SILTY SAND (SM)** brown, moist, medium dense	
	31.9	86	8	D		**CLAY (CL)** gray, wet, firm	
	28.2	88				**SANDY SILT (ML)** gray, wet, firm	20
			13	D	10	@ 10 feet, no recovery	
	42.5		7	D		**SILTY CLAY (CL)** gray, wet, firm, sample disturbed	
							15
					15		
							10
					20		
							5
					25		
							0
					30	Total Depth 30 feet	

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

DATE DRILLED:
9-18-23

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
10

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PROJECT NO.: 3033.1I

HOLLAND MAGNOLIA

LOG OF BORING NO. B-3

FIGURE B-3

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
			B	0	5-Inch Concrete	
					Fill: **SANDY SILT (ML)** brown, very moist	
16.1	96	17	D		Natural: **SANDY SILT (ML)** brown, very moist, stiff	
						25
17.9	89	13	D	5	@ 5 feet, loose	
29.6	91	10	D		@ 7 feet, wet	
					SILTY CLAY (CL) brown, wet, firm	20
30.4		8	D	10	**CLAYEY SILT (ML)** gray, wet, firm, sample disturbed	
					Total Depth 11 feet	

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

DATE DRILLED:
9-18-23

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
10

GPI

PROJECT NO.: 3033.1I

HOLLAND MAGNOLIA

LOG OF BORING NO. B-4

FIGURE B-4

MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					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	6-Inch Concrete	30
					Fill: **SANDY SILT (ML)** brown, very moist	
17.1	109	30	D		@ 2 feet, very stiff	
					SILTY SAND (SM) brown, very moist, medium dense	
12.2	92	16	D	5	Natural: **SILTY SAND (SM)** brown, very moist, medium dense	25
37.7	84	6	D		**SILTY CLAY (CL)** brown, wet, soft to firm	
29.0	90	14	D	10	**CLAYEY SILT (ML)** brown, wet, stiff	20
					SILTY SAND (SM) brown, wet, loose	
33.8	85	11	D	15	**SILTY CLAY (CL)** brown, wet, firm	15
					CLAYEY SILT (ML) brown, wet, firm	
23.0	100	8	D	20	**CLAY (CL)** gray, wet, firm	10
					SANDY CLAY (CL) gray, wet, firm	
34.8	85	12	D	25	**CLAYEY SILT (ML)** gray, wet, firm to stiff	5
31.8	86	12	D	30	**SANDY SILT (ML)** gray, wet, firm to stiff	0
					CLAYEY SILT (ML) gray, wet, firm to stiff	
27.9	117	13	D	35	**SANDY SILT (ML)** gray, wet, stiff	-5
32.1	88				**CLAYEY SILT (ML)** gray, wet, stiff	

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

DATE DRILLED: 9-18-23
EQUIPMENT USED: 8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft): 10

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PROJECT NO.: 3033.1I
HOLLAND MAGNOLIA

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.	
27.3	95	30	D	40	**SILTY SAND (SM)** gray, wet, medium dense	-10
30.4	86	11	D	45	**SANDY SILT (ML)** gray, wet, firm	-15
					SILTY CLAY (CL) gray, wet, firm	
52.7	74	12	D	50	**CLAY (CL)** gray, wet, firm to stiff	-20
43.5	75	13	D	55	@ 55 feet, stiff	-25
					CLAYEY SILT (ML) gray, wet, stiff	
35.5	86	20	D		**SILTY SAND (SM)** gray, wet, medium dense	
				60	Total Depth 60 feet	

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

DATE DRILLED:
9-18-23
EQUIPMENT USED:
8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft):
10

GPI

PROJECT NO.: 3033.1I
HOLLAND MAGNOLIA

LOG OF BORING NO. B-5

FIGURE B-5

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

GRAIN SIZE DISTRIBUTION

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

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	PERCENT PASSING No. 200 SIEVE
B-1	10	Silty Sand (SM)	17
B-2	20	Sandy Clay (CL)	62
B-5	5	Silty Sand (SM)	34

ATTERBERG LIMITS

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

DIRECT SHEAR

Direct shear tests were performed on undisturbed and remolded bulk samples in accordance with ASTM D 3080. The bulk samples were remolded to approximately 90 percent of the maximum dry density. The test specimens were placed in the shear machine, and a normal load comparable to the in-situ overburden stress was applied. The samples were inundated, allowed to consolidate, and then were sheared to failure at a strain rate of 0.001 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 tests. The results of the direct shear tests are presented in Figures C-2 to C-4.

CONSOLIDATION

One-dimensional consolidation testing was performed on undisturbed samples in accordance with ASTM D 2435. After trimming the ends, the samples were placed in the consolidometer and loaded to 0.4 or 0.5 ksf. Thereafter, the samples were incrementally loaded to a maximum load of 25.6 or 32 ksf. The samples were inundated at 1.6 or 2 ksf. Sample deformation was measured to 0.0001 inch. Rebound behavior was investigated by unloading the samples back to 0.4 or 0.5 ksf. Results of the consolidation tests, in the form of percent consolidation versus log pressure, are presented in Figures C-5 to C-8.

EXPANSION INDEX

An expansion index test was performed on a bulk sample. The test was performed in accordance with ASTM 4289 to assess the expansion potential of on-site soils. The results of the test are summarized below:

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	EXPANSION INDEX
B-4	0 - 5	Sandy Silt (ML)	30

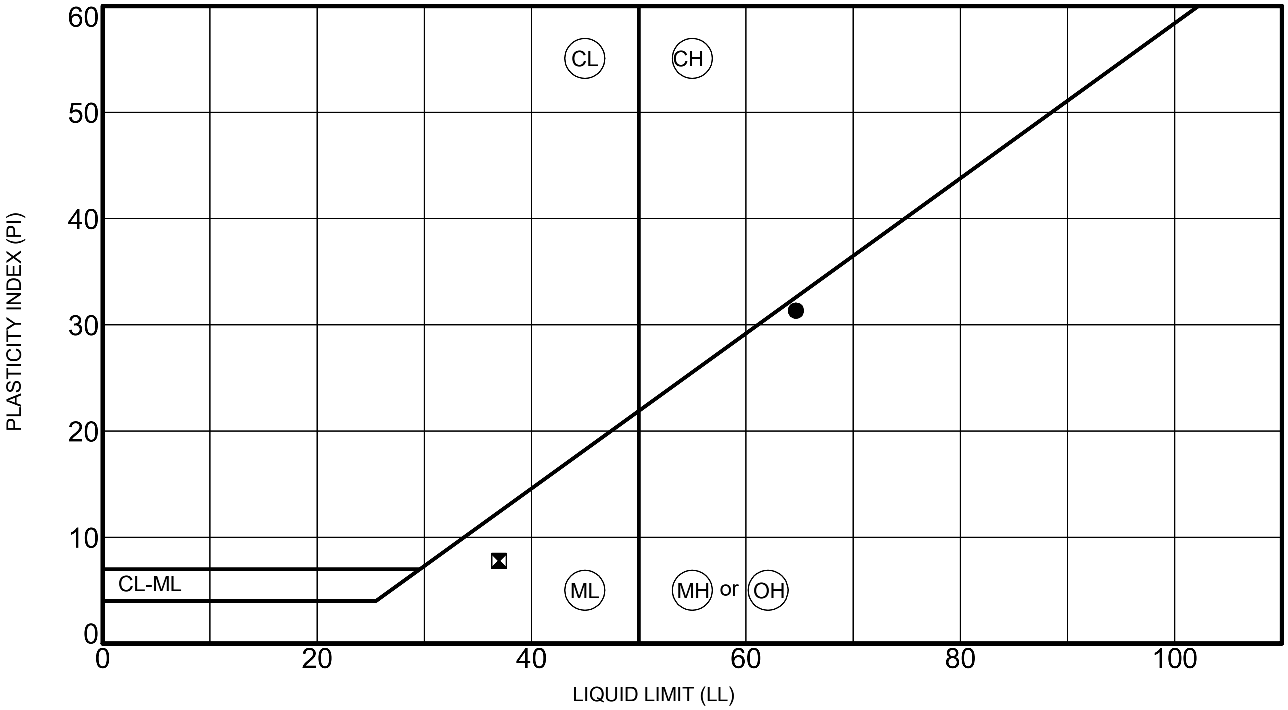
COMPACTION TEST

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

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)
B-4	0-5	Sandy Silt (ML)	118	8

CORROSIVITY

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



	SAMPLE LOCATION		LL	PL	PI	Fines, %	Classification
●	B-1	30.0	65	33	31		ELASTIC SILT (MH)
⧗	B-2	10.0	37	29	8		CLAYEY SILT (ML)

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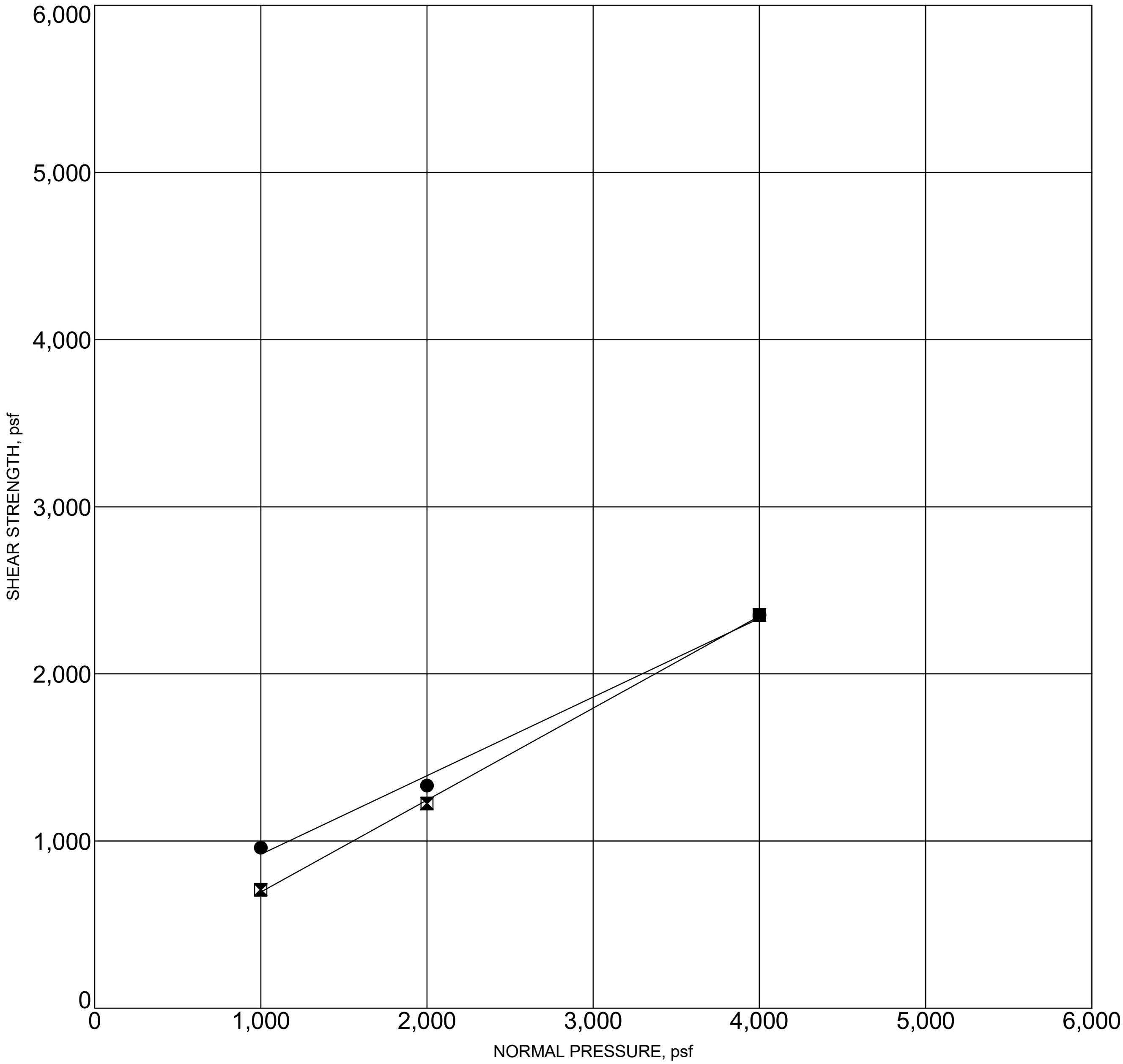
ATTERBERG LIMITS TEST RESULTS

FIGURE C-1



● **PEAK STRENGTH**
Friction Angle= 25 degrees
Cohesion= 450 psf

ULTIMATE STRENGTH
Friction Angle= 29 degrees
Cohesion= 144 psf

Sample Location		Classification	DD,pcf	MC,%
B-1	5.0	SANDY SILT (ML)	106	13.7

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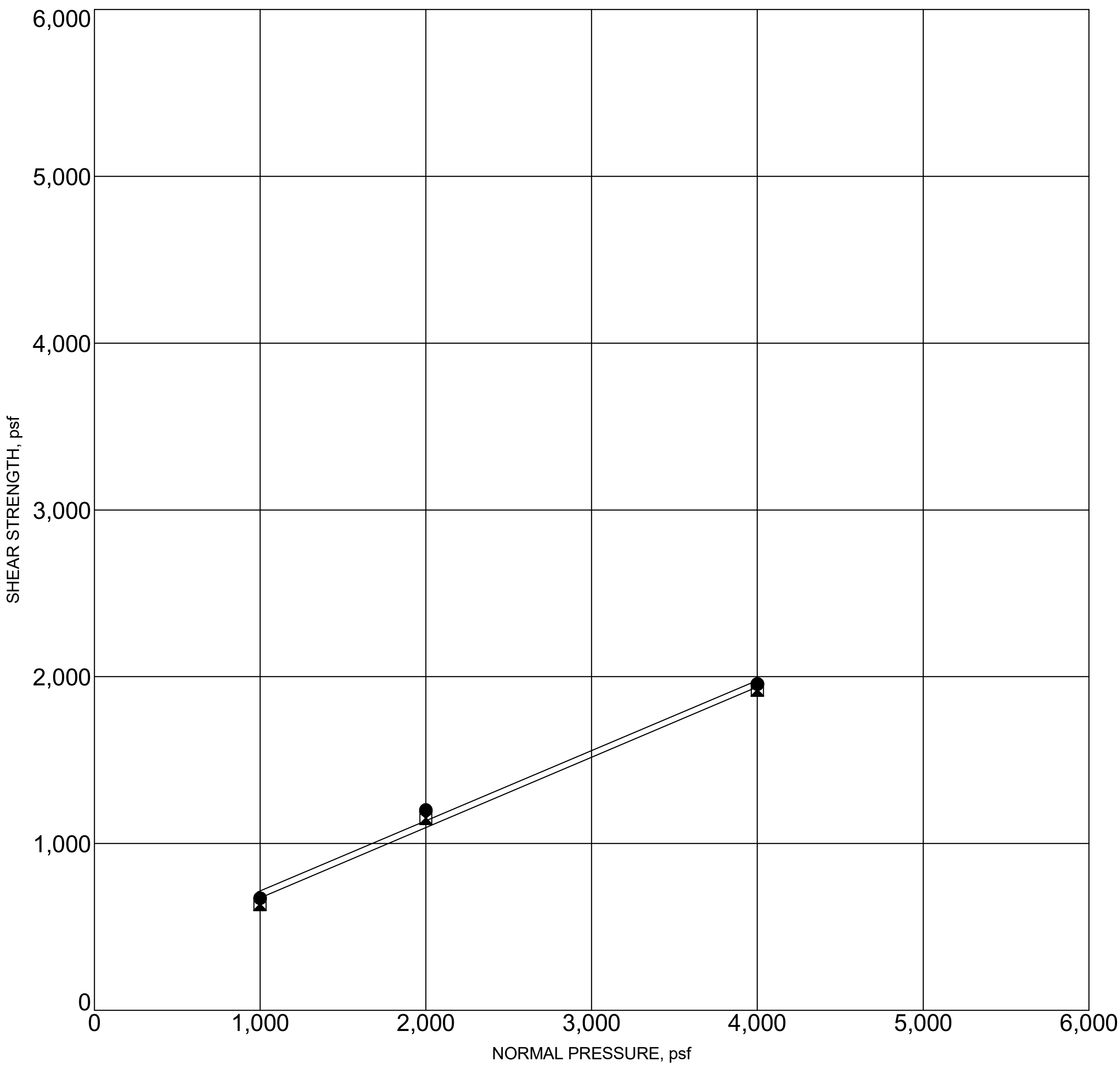
DIRECT SHEAR TEST RESULTS

FIGURE C-2



● **PEAK STRENGTH**
Friction Angle= 23 degrees
Cohesion= 294 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 23 degrees
Cohesion= 252 psf

Sample Location		Classification	DD,pcf	MC,%
B-5	7.0	SILTY CLAY (CL)	84	37.7

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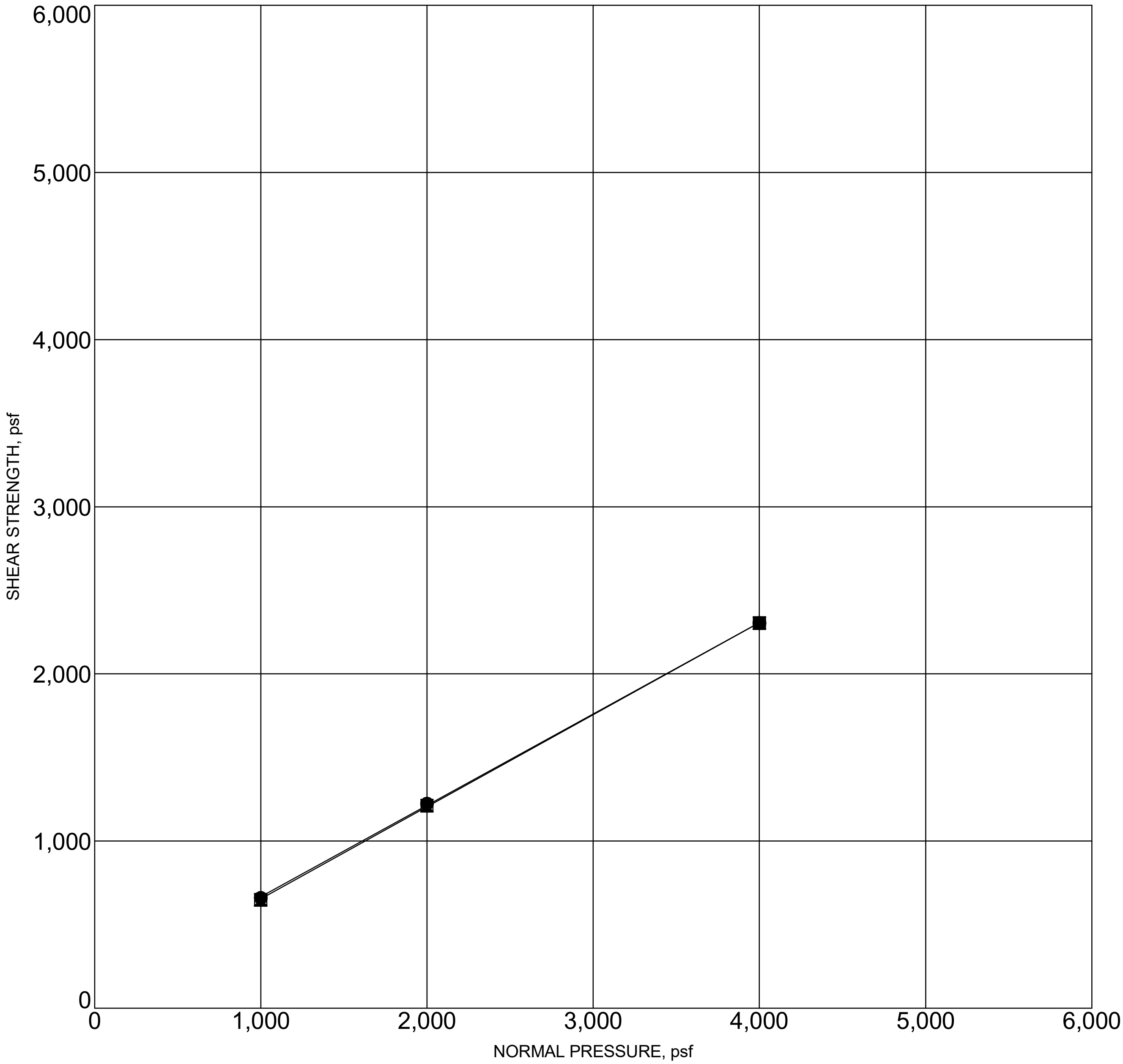
DIRECT SHEAR TEST RESULTS

FIGURE C-3



● **PEAK STRENGTH**
Friction Angle= 29 degrees
Cohesion= 120 psf

ULTIMATE STRENGTH
Friction Angle= 29 degrees
Cohesion= 102 psf

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

Sample Location		Classification	DD,pcf	MC,%
B-4	0-5	SANDY SILT (ML)	106	8.0

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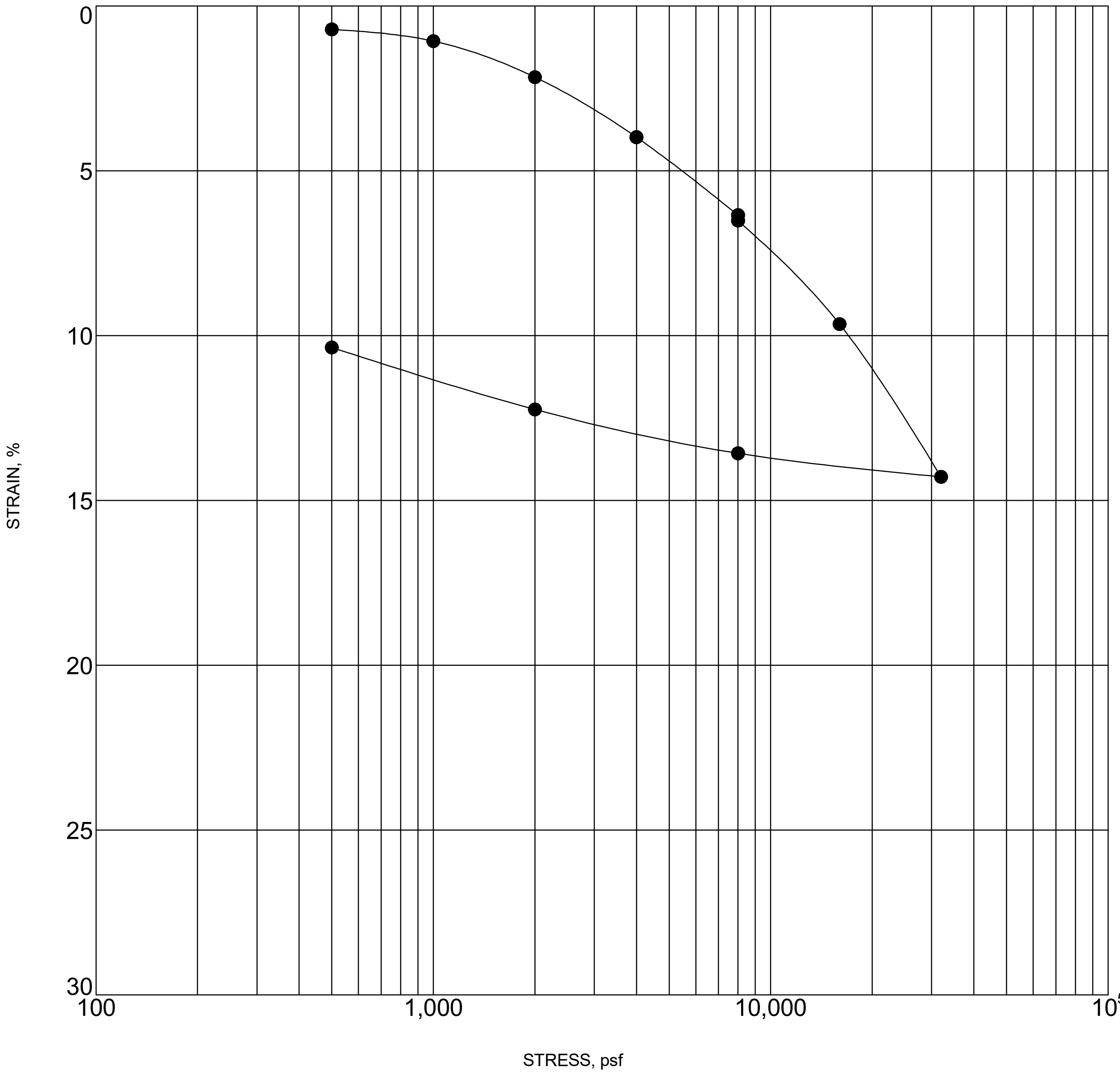
DIRECT SHEAR TEST RESULTS

FIGURE C-4



Sample inundated at 8000 psf

	Sample Location		Classification	DD,pcf	MC,%
●	B-1	35.0	CLAY (CL)	80	37.7

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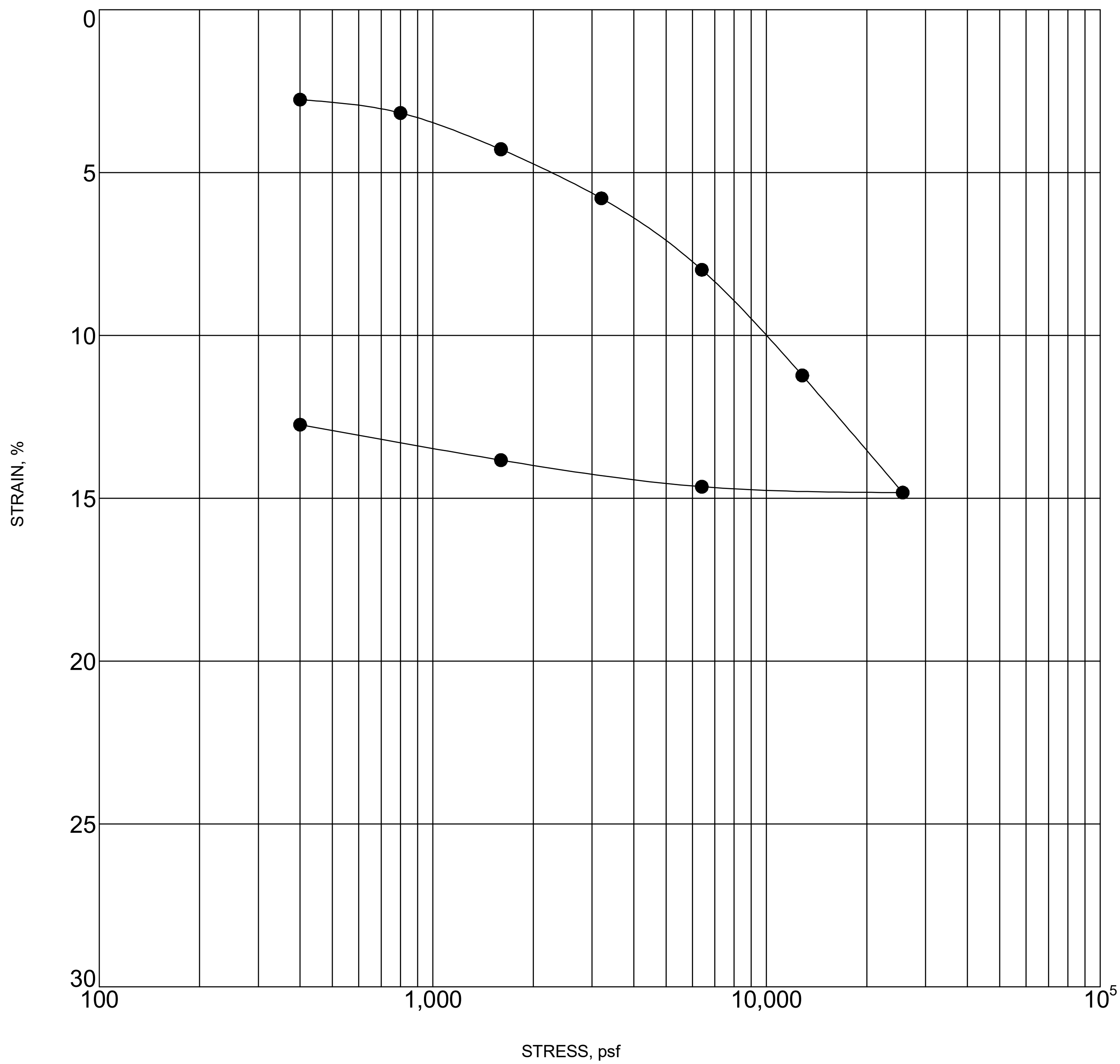
CONSOLIDATION TEST RESULTS

FIGURE C-5



Sample inundated at 1600 psf

	Sample Location		Classification	DD,pcf	MC,%
●	B-2	10.0	CLAYEY SILT (ML)	86	33.3

PROJECT: HOLLAND MAGNOLIA

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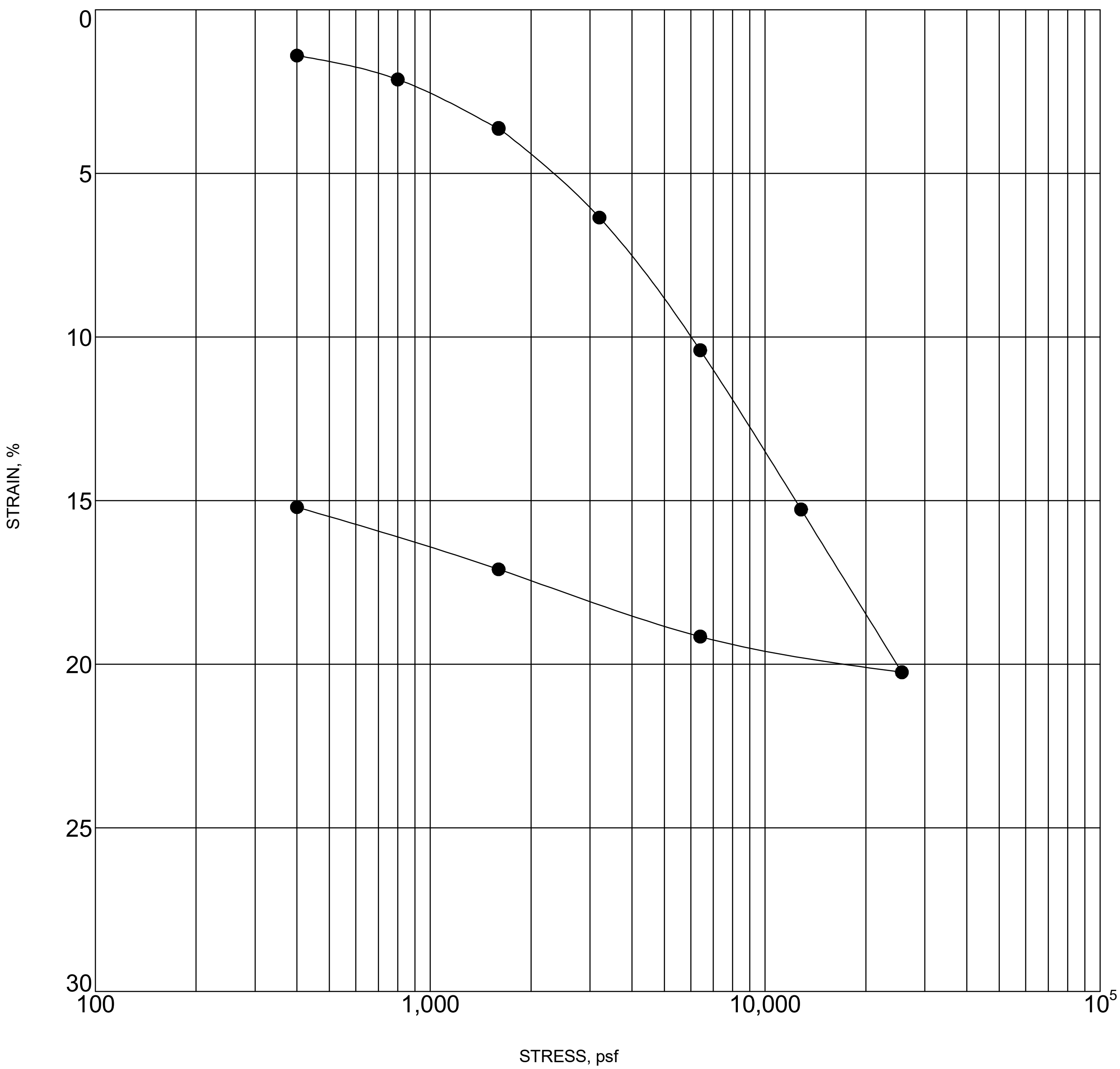
CONSOLIDATION TEST RESULTS

FIGURE C-6



Sample inundated at 1600 psf

	Sample Location		Classification	DD,pcf	MC,%
●	B-2	15.0	CLAY (CL)	72	45.6

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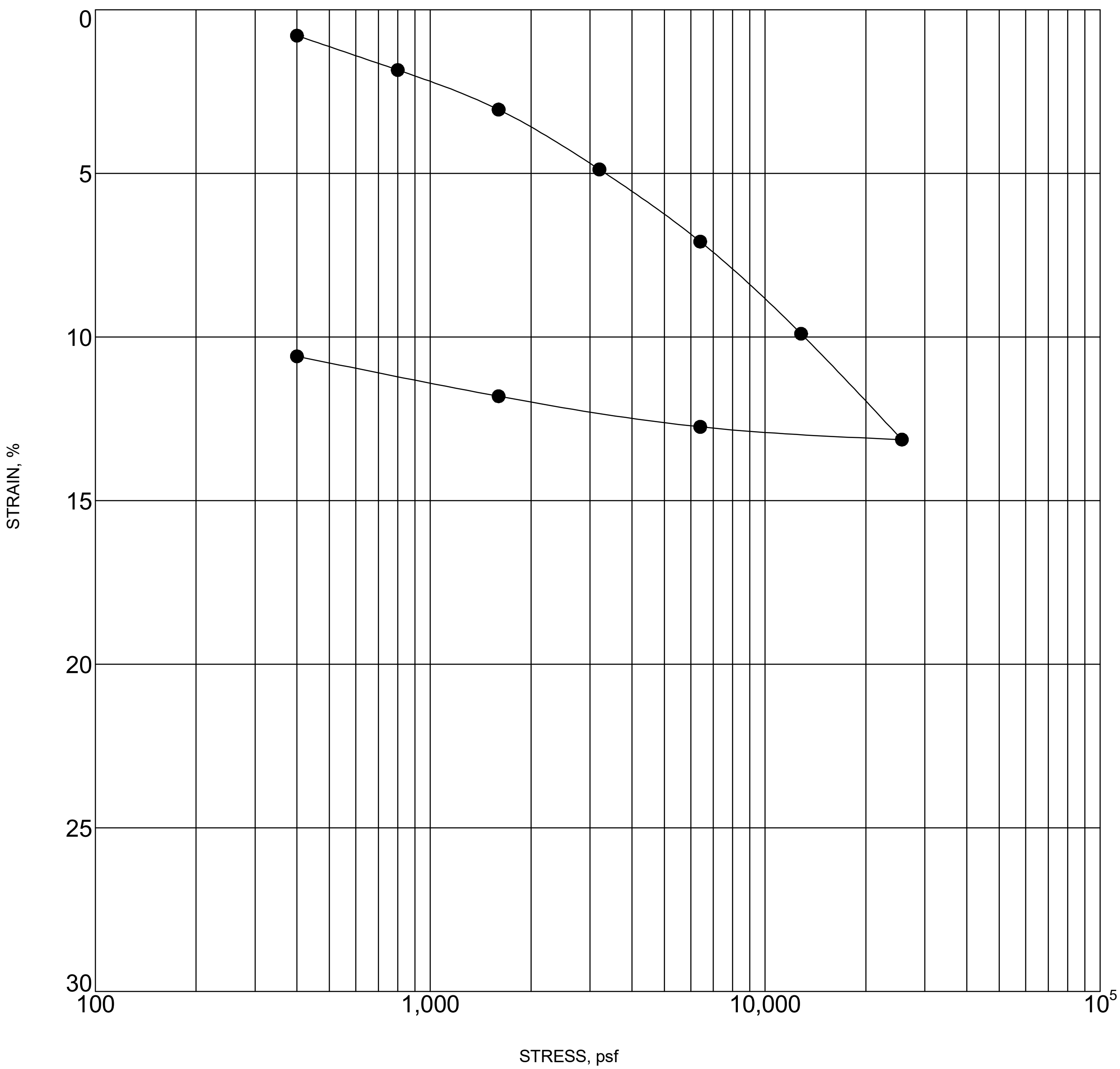
CONSOLIDATION TEST RESULTS

FIGURE C-7



Sample inundated at 1600 psf

	Sample Location		Classification	DD,pcf	MC,%
●	B-5	20.0	CLAY (CL)	100	23.0

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CONSOLIDATION TEST RESULTS

FIGURE C-8

Soil Analysis Lab Results

Client: Geotechnical Professionals Inc.
Job Name: Holland Magnolia
Client Job Number: 3033.1I
Project X Job Number: S230927A
September 28, 2023

	Method	ASTM D4327		ASTM D4327		ASTM G187		ASTM G51	ASTM G200	SM 4500-D	ASTM D4327	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D6919	ASTM D4327	ASTM D4327
Bore# / Description	**Depth**	**Sulfates** SO_4^{2-}		**Chlorides** Cl^-		**Resistivity** As Rec'd | Minimum		**pH**	**Redox**	**Sulfide** S^{2-}	**Nitrate** NO_3^-	**Ammonium** NH_4^+	**Lithium** Li^+	**Sodium** Na^+	**Potassium** K^+	**Magnesium** Mg^{2+}	**Calcium** Ca^{2+}	**Fluoride** F_2^-	**Phosphate** PO_4^{3-}
	(ft)	(mg/kg)	(wt%)	(mg/kg)	(wt%)	(Ohm-cm)	(Ohm-cm)		(mV)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)	(mg/kg)
B-4	0-5	110.6	0.0111	15.4	0.0015	3,618	1,809	7.4	150	0.2	101.0	7.0	ND	63.2	6.7	26.8	104.2	14.7	3.3

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

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