Appendix C

Geotechnical Investigation

GEOTECHNICAL
PROFESSIONALS INC.

GEOTECHNICAL INVESTIGATION
PROPOSED MIXED-USE APARTMENT DEVELOPMENT
MAGNOLIA FLATS
NEC MAGNOLIA AVENUE AND BANBURY DRIVE
RIVERSIDE, CALIFORNIA

Prepared for:
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Project No. 2924.1I

January 3, 2020

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Magnolia Partnership LLC
1201 Dove Street, Suite 520
Newport Beach, California 92660

Attention: Mr. Todd Cadwell

Subject: Report of Geotechnical Investigation
Proposed Mixed-Use Apartment Development
Magnolia Flats
NEC Magnolia Avenue and Banbury Drive
Riverside, California
GPI Project No. 2924.1I

Dear Mr. Cadwell:

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. When requested, we will provide wet signed originals for submittal to regulatory agencies.

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

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

1.2 PROJECT DESCRIPTION

The proposed project will consist of a mixed-use apartment and retail development. The apartment portion of the development consists of 4-story wood framed structures surrounded by at-grade parking with carports. The apartment building will be in the northern portion of the site away from Magnolia Avenue. The apartment development will include 2 interior courtyards as well as leasing office, amenity buildings, and pool area. A retail shops building will be located near the front of the development along Magnolia Avenue. At-grade parking, outdoor eating areas and a landscape area will surround the shops building. A park is planned at the north portion of the site beyond at-grade apartment parking and carports. Additional improvements will include new paved driveways and landscaping. The proposed site configuration is shown on the Site Plan, Figure 2.

Structural load information is not available at this time. We have assumed column loads for the 4-story apartments are on the order of 150 kips and maximum wall loads of on the order of 5 kips per lineal foot. For the single-story retail, we have assumed column loads on the order of 50 kips and wall loads on the order of 2 to 3 kips. We understand that the type of foundation for the apartments will likely be post-tensioned mat slabs. We assume that the type of foundation for the shops building will be shallow footings with slab-on-grade floors.

Based on preliminary grading plans, finished grades will generally be within 1 to 3 feet of the existing site grades. The preliminary grading plans indicate that approximately 14,500 cubic yards of import material will be required for the project.

Our recommendations are based upon the above structural and grading information. We should be notified if the actual loads and/or grades change during the project design to either confirm or modify our recommendations. When the project structural and grading plans become available, we should be provided with copies for review and comment.

1.3 PURPOSE OF INVESTIGATION

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

We prepared a geotechnical feasibility-level evaluation report earlier in 2019 (Reference 1) for the project. As part of our feasibility-level evaluation, we performed five Cone Penetration Test's (CPT's) which have been incorporated into this report.

Our field exploration consisted of five CPT's and ten exploratory borings. The field locations and designations of the subsurface explorations are shown on the Site Plan, Figure 2. The CPT's were advanced to depths ranging from 22 to 60 feet below existing site grades. Detailed logs of the CPT's and a summary of the equipment used are presented in Appendix A. The exploratory borings were drilled using truck-mounted, hollow-stem auger equipment to depths of 21 to 61 feet below existing site grades. Details of the drilling and boring logs are presented in Appendix B.

Field percolation testing was performed to evaluate the feasibility of dry wells for storm water infiltration. The results of our testing were presented in a separate report (Reference 2).

Laboratory tests were performed on selected representative soil samples as an aid in soil classification and to evaluate the engineering properties of the soils. The geotechnical laboratory testing program included determinations of moisture content and dry density, fines content, shear strength, collapse, compressibility (consolidation), maximum density/optimum moisture, subgrade strength (R-value) and soil corrosivity. 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 to provide earthwork criteria, foundation, retaining wall and slab design parameters, preliminary pavement sections, and assessments of seismic hazards. The results of our evaluations are presented in the remainder of the report.

3.0 SITE CONDITIONS

3.1 SURFACE CONDITIONS

The project site covers 16.3 acres and is currently occupied by an abandoned surface parking lot, undeveloped former building pads, and undeveloped vegetated areas. The former building pads are located along Magnolia Avenue and in the northern portion of the site. The slabs and foundations at the former buildings pads have been removed and the exposed soils are lightly vegetated. The abandoned parking is asphalt concrete in very poor condition with alligator cracking in many areas.

Historic aerials indicate that a large retail box store and a smaller building were demolished between 2005 and 2009 (Reference 3). The large box store covered a footprint of approximately 115,000 sf. The rear portion of the site along the northern property line did not formerly contain buildings. In the 1960's prior to the buildings being developed, historic aerials indicate that the land was likely used for agricultural purposes.

The site is bounded by single family homes to the north, a medical supply building and two large tilt-up retail buildings to the east, Magnolia Avenue to the south, and a retail building and a trailer park to the west. Bradbury Drive extends along the retail building at the southwest portion of the site.

The existing site is relatively flat with very gentle slope downward to the west. In general, the eastern side of the site is approximately 3 to 4 feet higher than the western side over a distance of approximately 350 to 450 feet.

3.2 SUBSURFACE SOILS

Our field investigation disclosed a subsurface profile consisting of undocumented fills underlain by natural soils. Detailed descriptions of the subsurface conditions encountered are shown on the Logs of CPT's and Borings in Appendices A and B, respectively. A brief summary is provided below.

The existing pavement sections encountered at our boring locations consisted of 2- to 3-inch thick asphalt concrete without underlying aggregate base.

We encountered undocumented fills to a depth of approximately 2 to 6 feet below existing grades at our borings. The fills consisted of dry to slightly moist, silty sands. The fills were difficult to distinguish from the underlying natural soils. Documentation regarding the placement and compaction of the fill was not provided.

Underlying the undocumented fills, the soils in the upper 10 to 15 feet generally consisted of loose to medium dense silty sands and firm to stiff sandy silts. In general, the upper 10 feet of these soils are dry to slightly moist. The soils below a depth of 10 to 15 feet consist of interbedded clays, silty clays, clayey silts, sandy silts, silty sands, and sands. The clays and silts are very stiff to hard and the sands are medium dense to very dense.

In general, the sands become denser with depth. At the northeast corner of the site, the soils appeared to be a weathered bedrock (decomposed granite) at depths of 10 to 20 feet below the existing grade.

The near-surface silty sands and sandy silts have a low to moderate strength and moderate compressibility characteristics when wetted under load. The near surface soils are anticipated to have very low expansion potential. In general, the soils below a depth of approximately 10 to 15 feet exhibit low to moderate strength and low to moderate compressibility characteristics.

3.3 GROUNDWATER AND CAVING

Groundwater was encountered in our exploratory borings at a depth of 57 feet below the existing ground surface.

The historical high groundwater has not been determined in the area by the State of California.

Caving was not observed in our borings. However, excavations within the loose to medium dense and dry to slight moist silty sands and sands in the upper 10 to 15 feet should be expected to encounter caving conditions.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 GENERAL

Based on the results of our investigation, it is our opinion that from a geotechnical viewpoint it is feasible to develop the site as proposed. The proposed buildings can be supported on shallow foundations or post-tensioned slabs after remedial grading to mitigate the geotechnical constraints discussed below.

The most significant geotechnical issues that will affect the design and construction of the proposed structures are as follows:

- The existing undocumented fills and a portion of the natural soils are not considered suitable for direct support of foundations or floor slabs. Earthwork recommendations are provided in Section 4.3.2 of the report.
- In proposed pavement areas, removals should extend to a depth of 1 foot below existing or finished grades, whichever is deeper, and replaced as properly compacted fill.
- The natural soils in the upper 10 to 15 feet from grade exhibit a potential for seismic ground subsidence in the event of a design level earthquake or hydroconsolidation, if wetted under load. Removal/recompaction, soaking, and heavy vibratory compaction will be required in the building and pavement areas in order to help mitigate these settlements. Details are provided in Section 4.3.2 of the report
- Additionally, methods such as avoiding landscaping within 10 feet of buildings or providing impermeable liners under the landscaping can be implemented in order to help mitigate the potential for hydroconsolidation within the building footprints after grading.
- Resistivity testing of a representative sample of the on-site soils indicates that they are moderately corrosive to 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 understand that the seismic design of the proposed building at the subject site will be in accordance with the 2019 California Building Code (CBC) criteria. For the 2019 CBC, we recommend a Site Class D. The remaining seismic code values can be obtained directly from the tables in the building code using the above values and appropriate SEAOC/OSHPD web site (Reference 4 for S_S, S_1, S_D, and S_M values).

The actual method of seismic design should be determined by the Project Structural Engineer.

4.2.2 Strong Ground Motion Potential

Based on published information (Reference 5), the most significant faults in the proximity of the site is the Elsinore Fault, which is located about 13 kilometers 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 SEAOC/OSHPD website (Reference 4), we computed that the site could be subject to a peak ground acceleration (PGAm) of 0.58g for a magnitude 7.1 earthquake. This acceleration has been computed using the mapped Maximum Considered Geometric Mean peak ground acceleration from ASCE 7-16 and a site coefficient (FPGA) based on Site Class. The structural design of the facility will need to incorporate measures to mitigate the effects of strong ground motion.

If the project will be designed in accordance with the 2019 CBC, site-specific response spectra is required unless the structure is designed for an alternate seismic response coefficient (Cs) in accordance with Section 11.4.8 of ASCE 7-16. If this exception is not taken and the structure will still be designed in accordance with the 2019 CBC, GPI should be notified to provide the required site-specific response spectra.

4.2.3 Ground Rupture

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

4.2.4 Liquefaction

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

The site is not located in a Seismic Hazard Zone for liquefaction by the State (Seismic Hazards Mapping Act, State of California), as the site vicinity has not yet been mapped.

The County of Riverside has mapped the site as an area with a very high susceptibility for liquefaction (Reference 6). The County assigned a groundwater depth of 30 feet for this zone.

The California Building Code and Special Publication 117A (Reference 7) requires that the ground motion used for liquefaction evaluation be based on the peak ground acceleration described above. The potential for liquefaction was evaluated using the methods presented in NCEER, 1998 (Reference 8) and modifications provided in Special Publication 117A. To evaluate the potential for liquefaction at the site, we considered the high groundwater level of 30 feet below existing grade.

A portion of the soils below a depth of 30 feet consists of very stiff to hard silty clays not typically considered to be potentially liquefiable. These clays are resistant to liquefaction based on interpreted I_C from the CPT data.

Based on our evaluation of the CPT data using computer software CLIQ (Reference 9), layers of sandy/silty soil at depths of approximately 40 to 50 feet exhibit a potential for liquefaction during a design earthquake. In general, the potentially liquefiable layers consist of medium dense silty sands and stiff sandy silts. Should liquefaction of these layers occur, the estimated magnitude of induced settlement would be on the order of ¼-inch or less. Differential settlement due to liquefaction across 40 feet would be less than ¼-inch.

4.2.5 Seismic Ground Subsidence

Seismic ground subsidence (not related to liquefaction induced settlements) occurs when strong earthquake shaking results in the densification of loose to medium dense sandy soils above groundwater. We estimate the total magnitude of subsidence would be on the order of ¼- to ½-inch should a design level earthquake occur. With remedial grading in the building pads as recommended in Section 4.3.2 of this report, the seismic ground subsidence under the buildings will be on the order of ¼-inch or less.

4.3 EARTHWORK

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

4.3.1 Clearing and Grubbing

Prior to grading, performing excavations or constructing the proposed improvements, the areas to be developed should be cleared of structures, vegetation, debris, and pavements. Buried obstructions, such as footings, abandoned utilities, abandoned underground storage tanks, and tree roots should be removed from areas to be developed. Deleterious material generated during the clearing operation should be removed from the site.

Although not anticipated, if cesspools or septic systems are encountered during grading, they should be removed in their entirety. The resulting excavation should be backfilled with properly compacted fill soils. As an alternative, cesspools can be backfilled with lean sand-cement slurry. At the conclusion of the clearing operations, a representative of GPI should observe and accept the site prior to further grading.

4.3.2 Excavations

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

Prior to placement of fills or construction of apartment and retail buildings, the existing fills and near-surface compressible soils within the proposed building pad should be removed and replaced as properly compacted fill. These materials require densification to provide adequate support of foundations and slab-on-grade floors. To density these materials, we recommend a combination of removal/replacement and in-place densification. For bidding/planning purposes, removals should extend at least 6 feet below existing or finished grades, whichever is deeper, in the footprint of proposed building with conventional shallow footings and slab-on-grade floors. Existing grade refers to elevations at locations of explorations.

If the apartment buildings are to be supported on post-tensioned mat slabs, removals should extend to 4 feet below existing or finished grades, whichever is deeper, in the proposed building pads. Localized deeper removals may be required to remove and replace undocumented fills.

In proposed pavement areas, removals should extend to a depth of 1 foot below existing or finished grades, whichever is deeper, and replaced as properly compacted fill.

The actual depths of removals will need to be determined during grading in the field by the GPI.

After the recommended removals are performed and prior to placing any fills or construction of the proposed building pads, 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 90 percent 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.

In order to help mitigate the collapse potential in the upper 10 to 15 feet of the soil profile, we recommend that the soils be soaked at the bottom of the excavations within the building footprints prior to proofrolling. Moistening of the soils below to a depth of 2 feet below the excavation bottom can usually be accomplished by deep ripping and liberal watering (including “rainbirds” or flooding) prior to compaction.

If the grading contractor does not provide the aforementioned heavy vibratory equipment to proof-roll the bottoms in the building and pavement areas, the recommended removals discussed above should be deepened by 1 foot.

For other lightly-loaded site structures outside the areas of heavy vibratory compaction, such as retaining walls, trash enclosures, equipment pads, or other minor structures, the soils should be overexcavated to a depth of at least 3 feet beneath the base of foundations.

The base of overexcavations should extend laterally at least 5 feet beyond the building lines, or a minimum distance equal to the depth of overexcavation below finish grade (i.e., a 1:1 projection below the top outside edge of footings), whichever is greater. Building lines include the footprint of the building and other foundation supported improvements, elevator pits, stairwells, screen walls, loading docks, ramps, and canopies. The corners of the areas to be overexcavated should be accurately staked in the field by the Project Surveyor.

Where not removed by the aforementioned excavations, existing utility trench backfill within building areas 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 4 feet below adjacent grade. For cuts up to 10 feet deep, the slopes should be properly shored or sloped back to at least 1:1 (horizontal:vertical) or flatter. For cuts up to 15 feet deep, the slopes should be properly shored or sloped back to at least 1.5:1 (horizontal:vertical) or flatter. The exposed slope face should be kept moist (but not saturated) during construction to reduce local sloughing. Some raveling of the loose sandy deposits should be anticipated at the slope inclinations recommended. If raveling cannot be tolerated, flatter slope inclinations should be considered. 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 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 State of California Occupational Safety and Health Standards.

4.3.3 Subgrade Preparation

After removals are complete and prior to placing fills, the subgrade soils should be scarified to a depth of 8 inches, moisture-conditioned, and compacted to dry densities equal to at least 90 percent of the maximum dry density, determined in accordance with ASTM D 1557.

In areas to receive pavements, the top 12 inches below the pavement base should be scarified, moisture-conditioned, and compacted to a minimum of 95 percent (90 percent for clayey soils) of the maximum dry density.

As discussed above, the exposed subgrade soils in building and parking areas should be moisture-conditioned and proofrolled with a heavy vibratory pad-foot-roller (minimum 40,000 pounds dynamic force) until the soils have been compacted to at least 90 percent of maximum dry density to a depth of 2 feet.

4.3.4 Material for Fill

The on-site soils are, in general, suitable for use as compacted fill. Clayey soils, if encountered, should not be placed within the upper 1 foot below slab-on-grade floor or hardscape, or used as retaining wall backfill. Retaining wall backfill and select fill below flatwork and slabs should consist of on-site or imported granular (containing no more than 40 percent fines – portion passing the No. 200 sieve) and relatively non-expansive (Expansion Index of 20 or less) soils.

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

Both imported and existing on-site soils to be used as fill should be free of debris and should not contain material larger than 6 inches in any dimension.

The 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.

4.3.5 Placement and Compaction of Fills

Fill soils should be placed in horizontal lifts, moisture-conditioned, and mechanically compacted to at least 90 percent of the maximum dry density determined 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, heavy loaders, and large vibratory rollers	8-12 inches

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

The moisture content of the on-site soils anticipated to be used as fill material is generally below optimum moisture. Moisture-conditioning (wetting) of the on-site soils will be required to readily achieve proper compaction. Granular fills should be placed at a moisture content of 0 to 2 percent over the optimum moisture content. Fills consisting of the on-site silts or clays, if encountered, should be placed at a moisture content of 2 to 3 percent over the optimum moisture content.

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

4.3.6 Shrinkage and Subsidence

Shrinkage is the loss of soil volume caused by compaction of fills to a higher density than before grading. Subsidence is the settlement of in-place subgrade soils caused by loads generated by large earthmoving equipment. For earthwork volume estimating purposes, an average shrinkage value of 10 to 15 percent may be assumed for the soils to be recompacted. Subsidence is expected to be on the order of 0.1 to 0.2 feet. These values are estimates only and exclude losses due to removal of vegetation or debris. Actual shrinkage and subsidence will depend on the types of earthmoving equipment used and should be determined during grading.

4.3.7 Trench/Wall Backfill

Utility trench and wall backfill material should be mechanically compacted in lifts. Clayey soils, if encountered at the site, should not be used as retaining wall backfill. Only sandy soils, on-site or imported, should be used as wall backfill. Lift thickness should not exceed those values given in the "Compacted Fill" section of this report. Jetting or flooding of backfill materials should not be permitted. GPI should observe and test all trench and wall backfills as they are placed.

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).

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 1½ sacks of cement per cubic yard and have a maximum slump of 5 inches. When set, such a mix typically has the consistency of compacted soil. We also recommend that slurry be used as bedding backfill material for trenches containing multiple lines.

4.4 FOUNDATIONS

4.4.1 Foundation Type

The proposed apartment building may be supported on conventional spread footings with slab-on-grade floor or a post-tensioned slab foundation subsequent to completion of the grading recommendations as provided in “Excavations” section of this report.

Lightly-loaded minor structures such as retaining walls, trash enclosures, recreational structures, monument signs and screen walls may be supported on lightly loaded spread footing.

4.4.2 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 3,000 pounds per square foot (psf) may be used for both continuous footings and isolated column footings for the proposed 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.

Minimum Footing Width and Embedment

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

STATIC BEARING PRESSURE (psf)	MINIMUM FOOTING WIDTH (inches)	MINIMUM FOOTING* EMBEDMENT (inches)
3,000	36	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.

Estimated Settlements

At the apartment building, total static settlement of the column footings (150 kips maximum column load) and wall footings (5 kip/lf) is expected to be on the order of 1-inch. Maximum differential settlements between similarly loaded adjacent footings or along a 40-foot span are expected to be less than ½-inch.

At the single-story shops building, total static settlement of the column footings (50 kips maximum column load) and wall footings (2 to 3 kip/lf) is expected to be on the order of 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 estimated static settlements should be included with the anticipated seismic settlement (½-inch total and ¼-inch differential) caused by the combination of liquefaction and seismic ground subsidence when evaluating the total settlement of the buildings.

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

For minor structures supported at-grade on properly compacted fill, total static settlement of is expected to be less than 1 inch. Maximum differential settlements between similarly loaded adjacent footings or along a 40-foot span are expected to be less than ½-inch.

4.4.3 Post-Tensioned Slabs

A post-tensioned slab foundation can be used to support the apartment buildings. The post-tensioned slab can be placed directly on the recompacted sandy soils at site with a very low expansion potential (E.I. < 20).

Based upon the similar projects, the post-tension slabs for the apartment buildings will impose an average pressure of approximately 200 to 300 psf. We anticipate the foundation of the portion of the apartment building with the largest footprint will be approximately 70 feet by 380 feet in plan dimension.

With the post-tensioned mat supported on engineered fill over the existing natural soils at the site, we estimate that the ground surface under the center of a loaded area, having the same dimensions and applied pressure as described above, will settle approximately ½-inch. The edge of this area under the same loading conditions is expected to settle approximately ¼-inch. The corner of this area under the same loading conditions is expected to settle less than ¼-inch.

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

The post-tensioned slab can be placed directly on the compacted fill derived from the on-site silty sands and sands with a very low expansion potential. The Project Structural Engineer does not need to consider the soil shrinkage or swelling in the design of the post-tensioned slab.

The allowable soil bearing pressure will be significantly greater than the average bearing pressures required for the post-tensioned slab foundation as discussed above. At localized or thickened areas of the slab, such as continuous footings, columns and point of load applications, a static allowable net bearing pressure of 2,000 pounds per square foot may be used with a minimum width of 18 inches and minimum embedment depth of 18 inches. These allowable bearing pressures are for dead-plus-live loads, and may be increased one-third for short-term, transient, wind and seismic loading.

4.4.4 Carport Footings

We assume that the carports will be supported on drilled pile footings. The design of the drilled pile footings will likely be governed by lateral force considerations. For design by the non-constrained embedded poles as presented in Section 1807.3.2.1 of the 2019 CBC, a unit passive resistance of up to 250 pounds per square foot per foot of depth may be used for the pile foundation in natural soils. We take no exception to the Structural Engineer incorporating the allowable increase (doubling of passive resistance) stated in the Section 1806.3.4 of the 2019 CBC for a single pole that can tolerate a deflection of ½-inch at the ground surface under short-term loads.

4.4.5 Lateral Load Resistance

Soil resistance to lateral loads will be provided by a combination of frictional resistance between the bottom of footings and underlying soils and by passive soil pressures acting against the embedded sides of the footings. For frictional resistance, a coefficient of friction of 0.35 may be used for design. In addition, an allowable lateral bearing pressure equal to an equivalent fluid weight of 350 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. The friction and lateral bearing values may be used in combination without reduction.

4.4.6 Foundation Concrete

Laboratory testing by HDR (Appendix C) indicates that the near surface soils exhibit a soluble sulfate content of 25 mg/kg (0.003 percent by weight). For the 2019 CBC, foundation concrete should conform to the requirements outlined in ACI 318, Section 4.3, for negligible levels of soluble sulfate exposure from the on-site soil.

4.4.7 Footing Excavation Observation

Prior to placement of steel and concrete, a representative of GPI should observe and approve all footing excavations including post-tensioned slabs.

4.5 BUILDING FLOOR SLABS

Slab-on-grade floors should be supported on granular (sandy) non-expansive, compacted fill soils as discussed in the “Subgrade Preparation” and “Placement and Compaction of Fill” sections. In general, the on-site soils in the upper 5 feet of the soil profile will meet this requirement.

A moisture vapor retarder should be placed under floor slabs including post-tensioned slabs that are to be covered with moisture-sensitive floor coverings (parquet, vinyl, tile, etc.). Currently, common practice is to use a 10 or 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. The function of the sand layer is to protect the vapor retarder during construction and to aid in the uniform curing of the concrete. 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 effective sealing of joints edges (particularly at pipe penetration) as well as excess moisture in the concrete. The manufacturer of floor coverings should be consulted for establishing acceptable criteria for the condition of floor surface prior to placing moisture-sensitive floor coverings.

For elastic design of slabs supporting sustained concentrated loads, a modulus of subgrade reaction (k) of 175 pounds per cubic inch (pounds per square inch per inch of deflection) may be used for compacted fill.

For lateral resistance design, a coefficient of friction of 0.35 can be used for concrete in direct contact with sandy fill. For slabs constructed over a visqueen or polyolefin moisture retarder, a friction coefficient of 0.1 should be used.

4.6 LATERAL EARTH PRESSURES

Based on information available to us at the time this report was prepared, retaining walls are not planned on the site. However, the following recommendations are provided for walls no more than 5 feet in height.

We recommend that on-site or imported non-expansive, granular soils (less than 40 percent fines) be used as conventional retaining 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 cantilever walls with level backfill comprised of granular soils, the magnitude of active pressures is equivalent to the pressures imposed by a fluid weighing 40 pounds per cubic foot (pcf). Should the cantilever walls be designed to resist hydrostatic pressure, a value for saturated backfill of 82 pcf may be used for level backfill. These values can also be used for design of temporary cantilevered shoring, if needed.

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 60 pounds per cubic foot should be used for granular backfill. Should the walls be designed to resist hydrostatic pressure, a value for saturated backfill of 92 pcf may be used for level backfill.

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 streets/drive aisles should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pound per square foot surcharge behind the wall due to normal street traffic. If traffic is kept at least 10 feet from the walls, the traffic surcharge may be neglected.

Unless the walls are designed to resist hydrostatic pressure, 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.

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

4.7 CORROSIVITY

Resistivity testing of a representative sample of the on-site soils indicates that they are moderately corrosive to metals. GPI does not practice corrosion protection engineering. A corrosion engineer such as HDR should be consulted for specific corrosion recommendations.

4.8 DRAINAGE

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

In order to help mitigate the potential for hydroconsolidation of the silty sands and sandy silts in the upper 10 to 15 feet of the soil profile underlying the footprints of the buildings after grading, methods such as avoiding landscaping within 10 feet of buildings or providing impermeable liners under the landscaping should be implemented.

4.9 STORM WATER INFILTRATION

Current regulations require that storm water be infiltrated into the site soils of new developments, when possible. The soil types present at the site control the ability of water to infiltrate into the subgrade. Due to an impermeable clay layer at a depth of 10 to 15 feet across the site and the soils overlying the clay impacted by a potential for hydroconsolidation under load, we recommend against shallow infiltration at this site.

We performed infiltration testing at the site to determine the suitability for infiltration of storm water into the site soils using deeper dry wells. We determined that the deeper sandy soils at the site are suitable for storm water infiltration. We issued a separate stand-alone report dated December 20, 2019 (Reference 2) providing our findings.

4.10 EXTERIOR CONCRETE AND MASONRY FLATWORK

Exterior concrete and masonry flatwork should be supported on non-expansive, compacted granular fill. While not anticipated near the existing grade, clayey soils should not be used in the upper 1-foot of hardscape subgrade. Prior to placement of concrete, the subgrade should be prepared as recommended in "Subgrade Preparation" section.

4.11 PAVED AREAS

Preliminary pavement design recommendations have been based on an R-value of 40 for the on-site sandy soils and conventional Traffic Indices (TI's) typically used for commercial developments. The California Division of Highways Design Method was used for design of the recommended preliminary pavement sections. These recommendations are based on the assumption that the pavement subgrades will consist of existing near-surface silty sand soils. Final pavement design should be based on R-value testing performed near the conclusion of grading activities.

The following pavement sections are recommended for planning purposes only.

PAVEMENT AREA	TRAFFIC INDEX	SECTION THICKNESS (inches)	
		Asphalt Concrete	**Aggregate Base Course**
Auto Parking	4	3	4
Circulation Drives	5	3	4
Truck Drives	6	3	7
		Portland Cement Concrete	**Aggregate Base Course**
Auto Parking	4	5.5	4
Circulation Drives	5	6	4
Truck Drives	6	6.5	5

The 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 traffic).

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

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 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.

4.12 GEOTECHNICAL OBSERVATION AND TESTING

We recommend that a representative of GPI observe all earthwork during construction to confirm that the recommendations provided in our report are applicable during construction. The earthwork activities include shoring, 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 the Magnolia Partnership 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 expressed or implied, is included or intended in our report.

Respectfully submitted,
Geotechnical Professionals Inc.

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2. Geotechnical Professionals Inc., "Percolation/Infiltration Testing, Proposed Mixed-Use Apartment Development, NEC Magnolia Avenue and Banbury Drive, Riverside, California," GPI Project No. 2924.1I, dated December 20, 2019.

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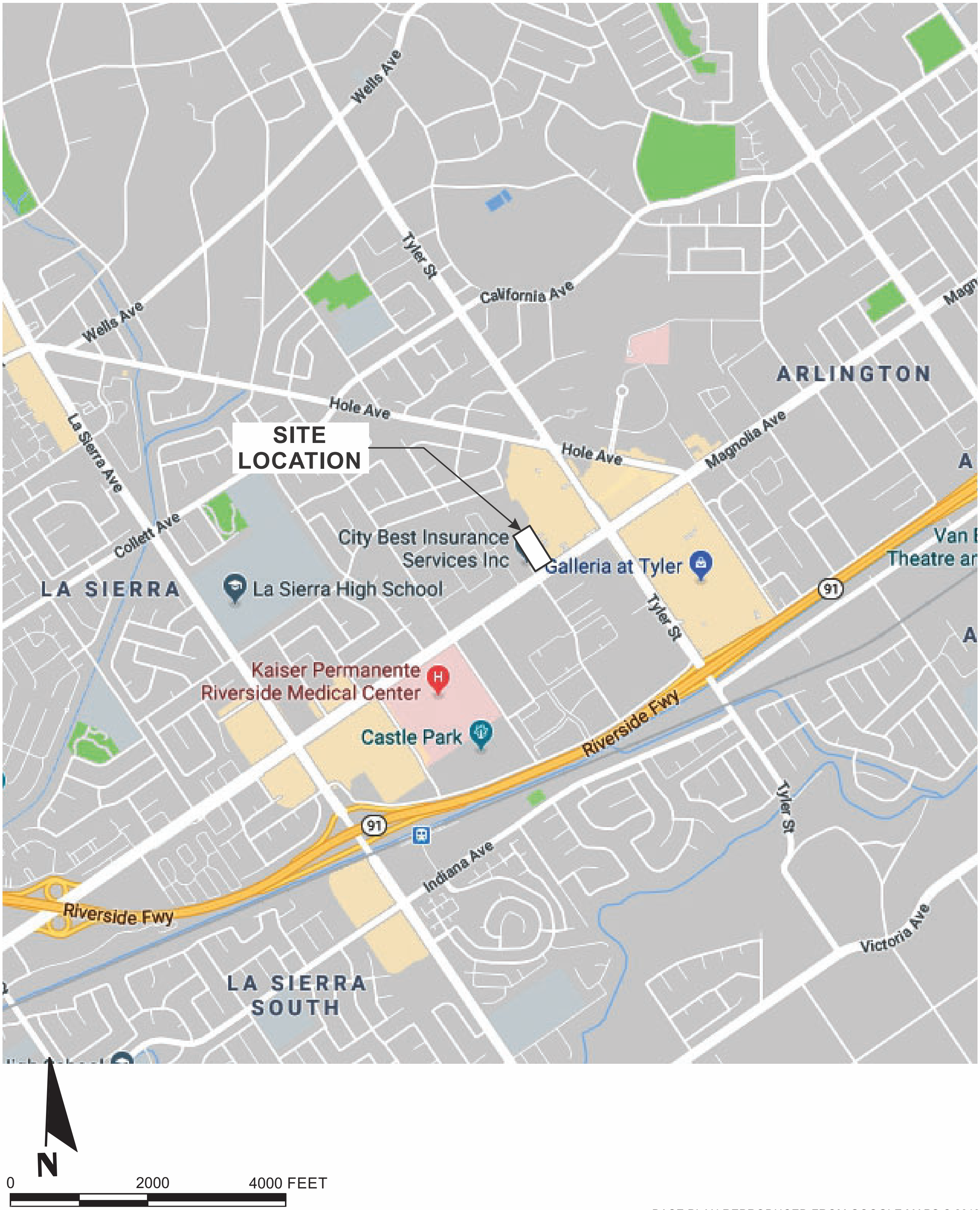


BASE PLAN REPRODUCED FROM GOOGLE MAPS © 2019

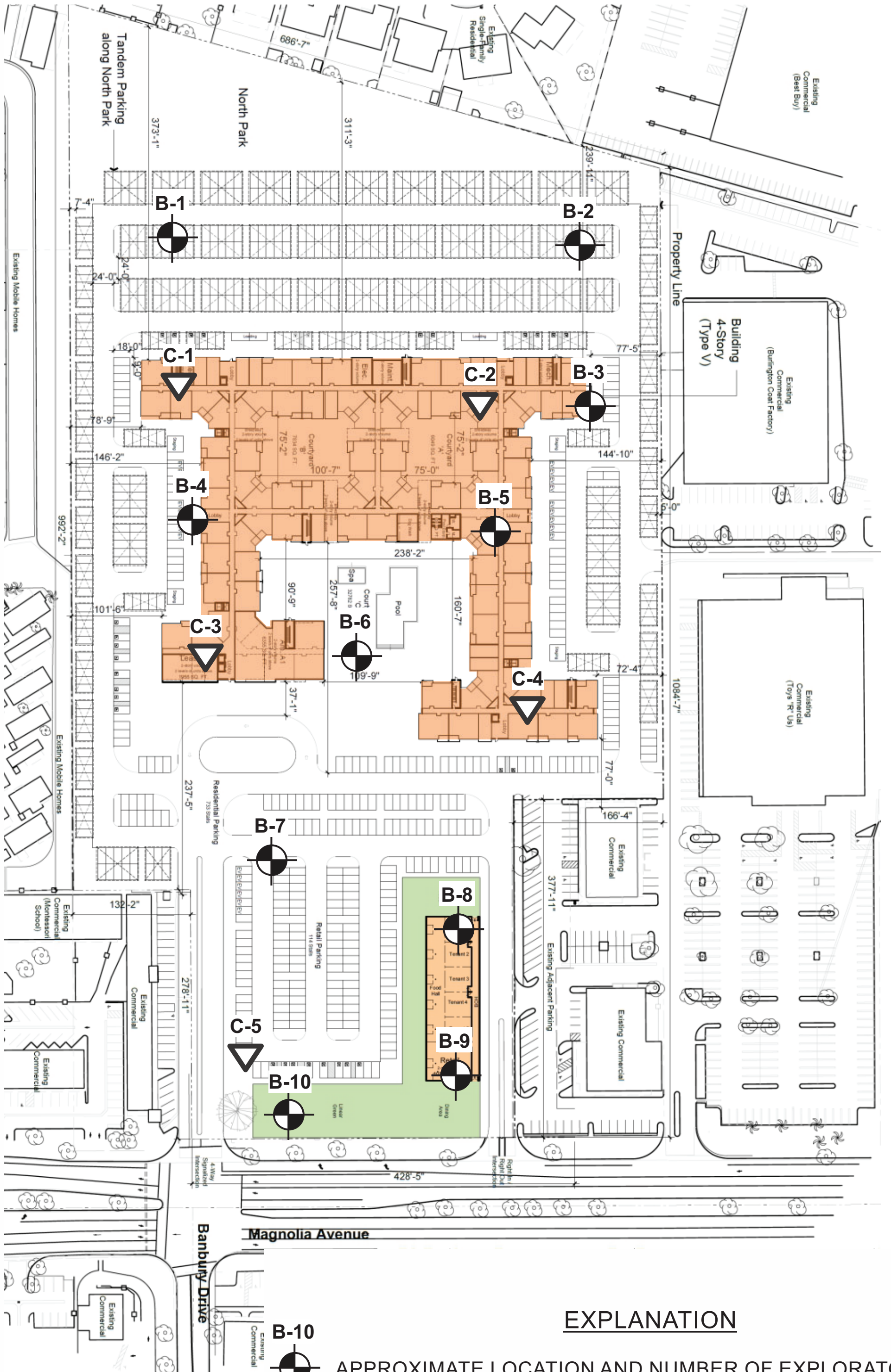
GPI GEOTECHNICAL PROFESSIONALS, INC.

MAGNOLIA AVENUE & BANBURY DRIVE

GPI PROJECT NO.:2924.1I | SCALE: 1" = 2000'

SITE LOCATION MAP

FIGURE 1



EXPLANATION

B-10 APPROXIMATE LOCATION AND NUMBER OF EXPLORATORY BORING

C-10 APPROXIMATE LOCATION AND NUMBER OF CONE PENETRATION TEST

N

0 200 400 FEET

BASE PLAN REPRODUCED FROM SITE PLAN, CONCEPTUAL DESIGN, PRODUCED BY KTGY ARCHITECTURE, DATED SEPTEMBER 18,2019

GPI GEOTECHNICAL PROFESSIONALS, INC.

MAGNOLIA FLATS

GPI PROJECT NO.:2924.1I	SCALE: 1" = 200'

SITE PLAN

FIGURE 2

APPENDIX A

APPENDIX A

CONE PENETRATION TESTS

The subsurface conditions were investigated by performing five Cone Penetration Tests (CPT's) at the site. These soundings were advanced to depths ranging from approximately 22 to 60 feet below existing grades. The locations of the CPT's are shown on the Site Plan, Figure 2.

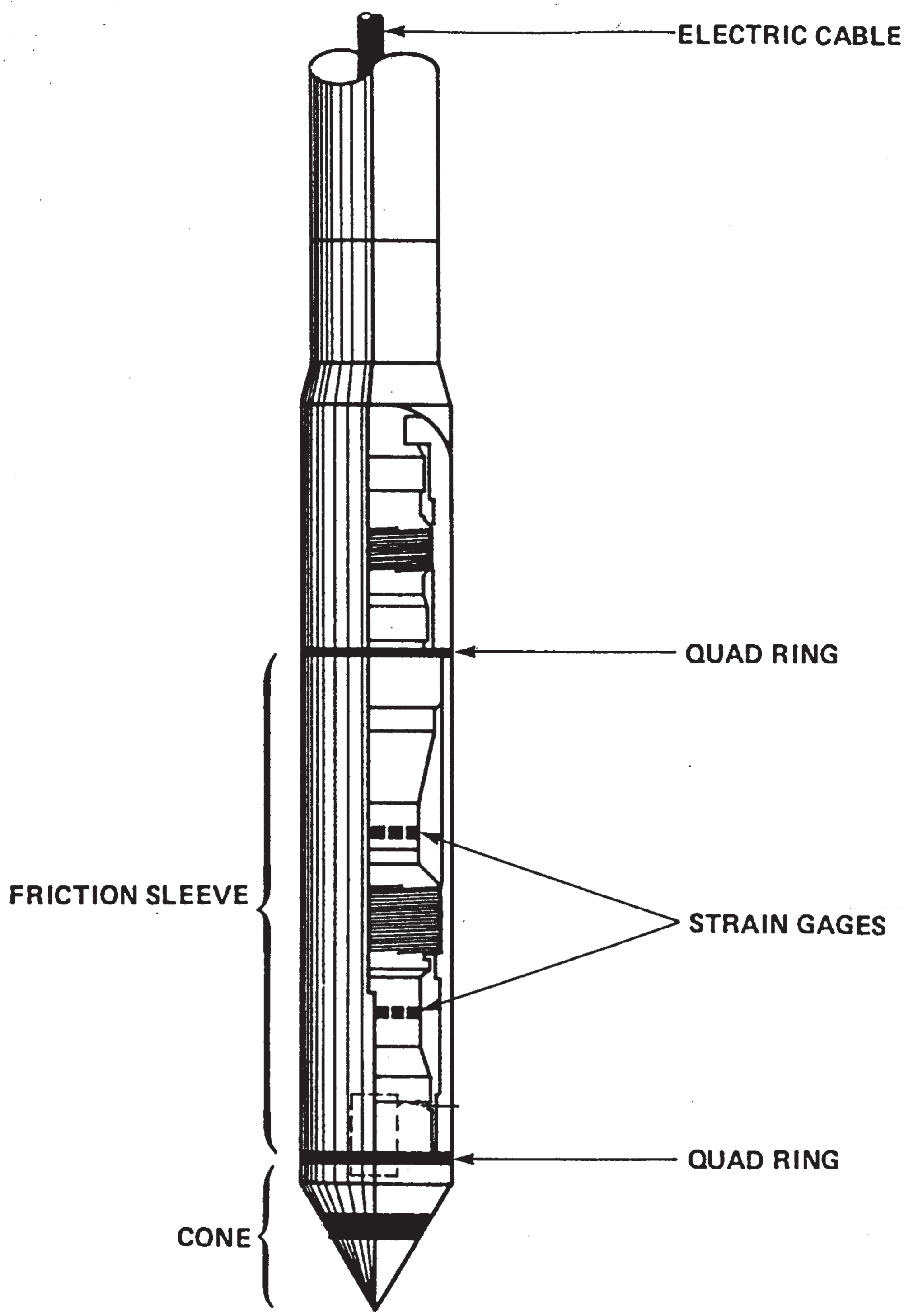
The Cone Penetration Test consists of pushing a cone-tipped probe into the soil deposit while simultaneously recording the cone tip resistance and side friction resistance of the soil to penetration (refer to Figure A-1). The CPT's described in this report were conducted in general accordance with ASTM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

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

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

Computer plots of the reduced CPT data acquired for this investigation are presented in Figures A-2 to A-6 of this appendix. The field testing and computer processing for the current investigation was performed by Kehoe Testing and Engineering, Inc. 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. The ground surface elevations at the CPT locations were estimated from a preliminary grading plan provided by KHR Associates and should be considered approximate.



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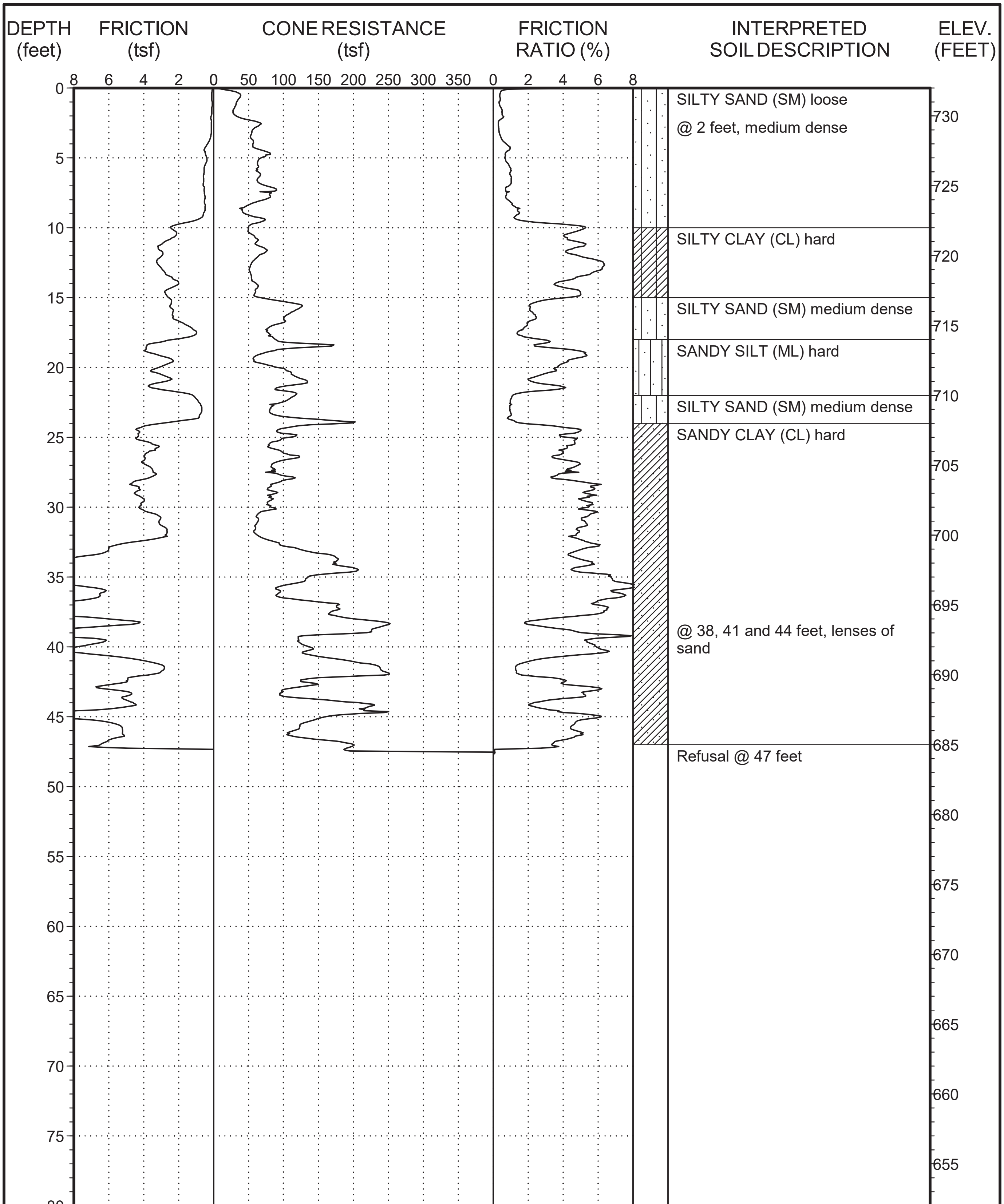
CONE PENET O ETER

FIGURE



Date performed: 1-24-19

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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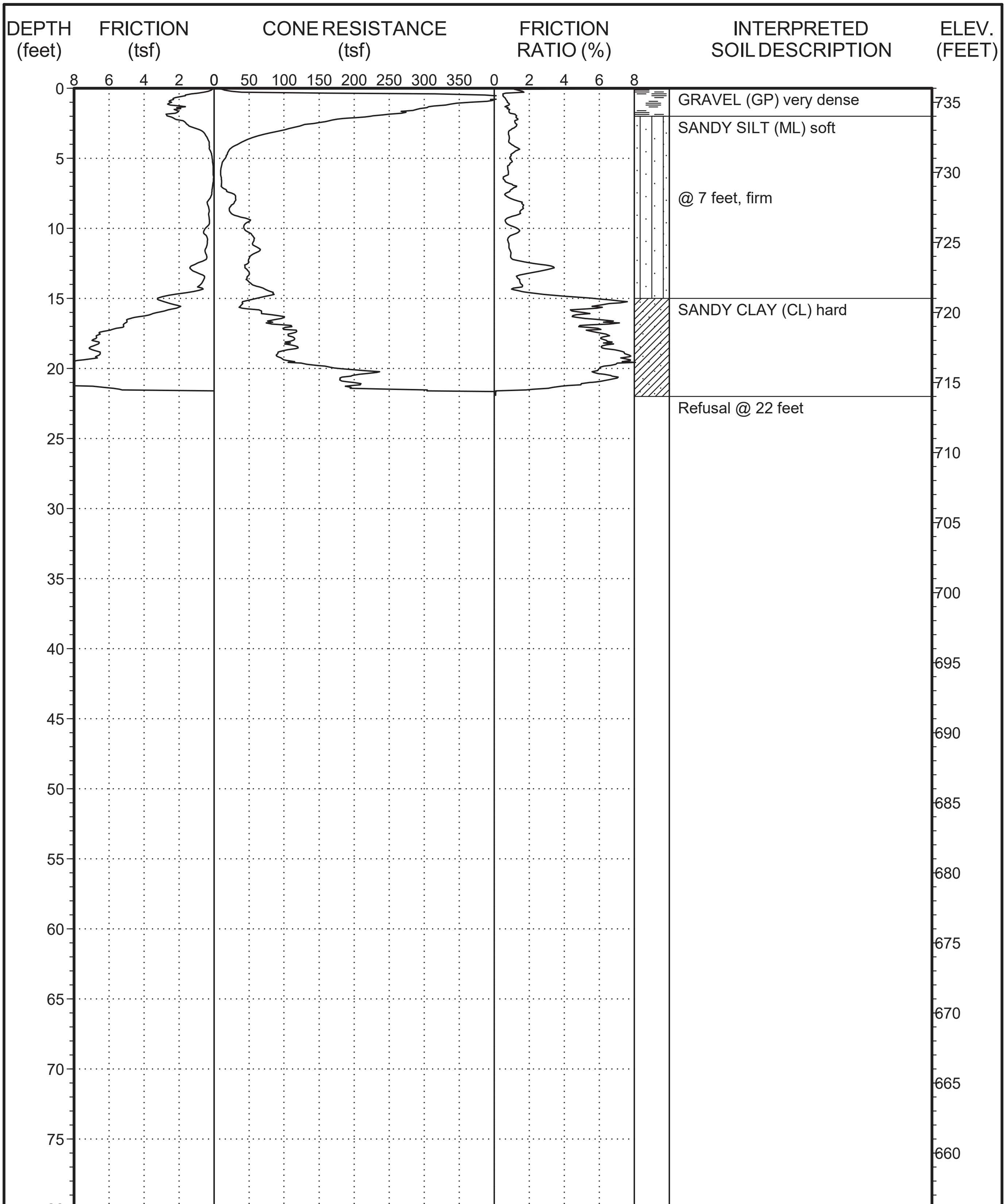
PROJECT NO.: 2924.1I

MAGNOLIA FLATS

LOG OF CPT NO. C-1

FIGURE A-2



Date performed: 1-24-19

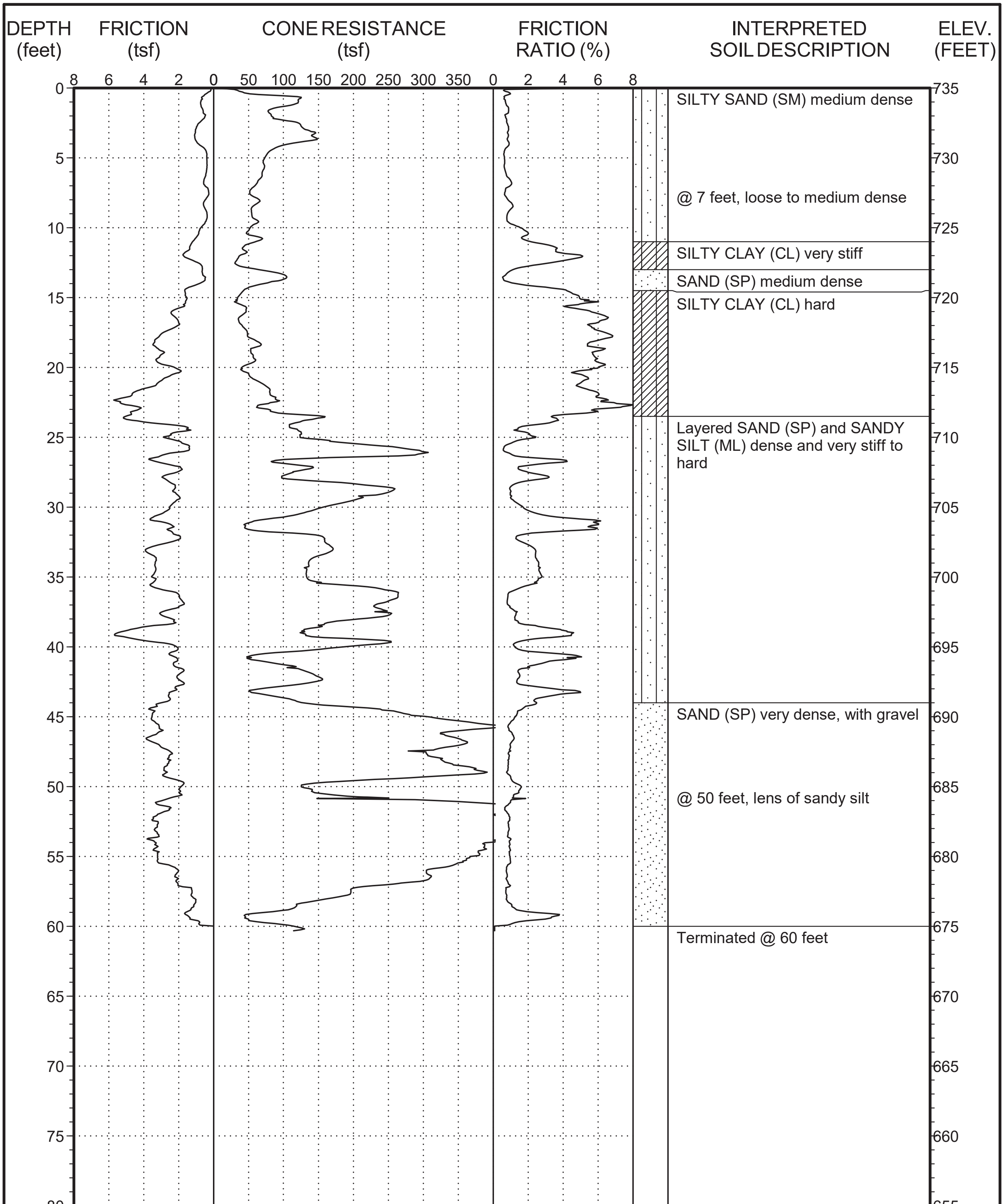
This summary applies only at the location of this cone penetration test and at the time of the exploration. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The interpreted soil description is derived from the friction ratio and cone resistance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

PROJECT NO.: 2924.1I

MAGNOLIA FLATS

LOG OF CPT NO. C-2

FIGURE A-3



Date performed: 1-24-19

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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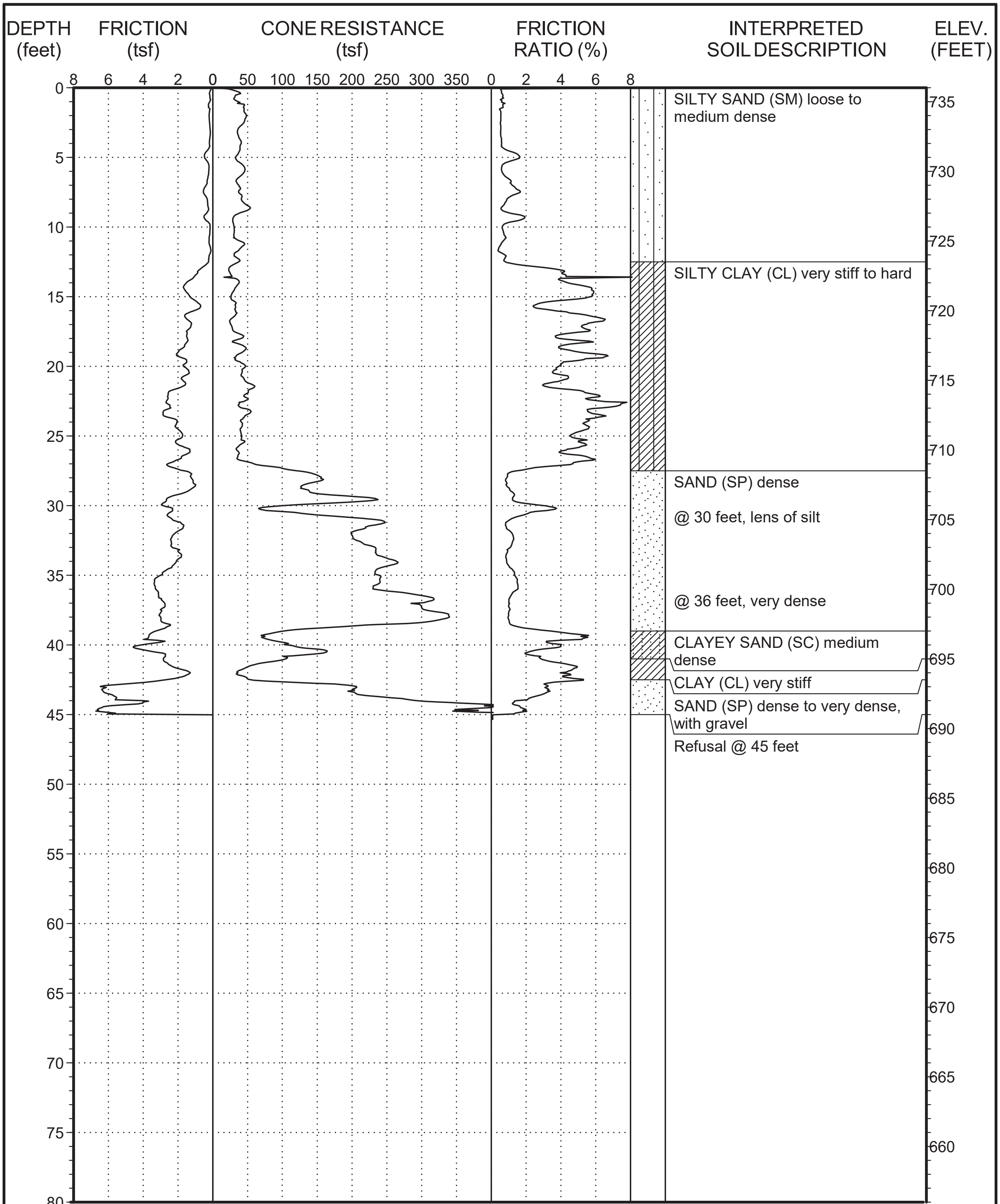
PROJECT NO.: 2924.1I
MAGNOLIA FLATS

LOG OF CPT NO. C-3

FIGURE A-4



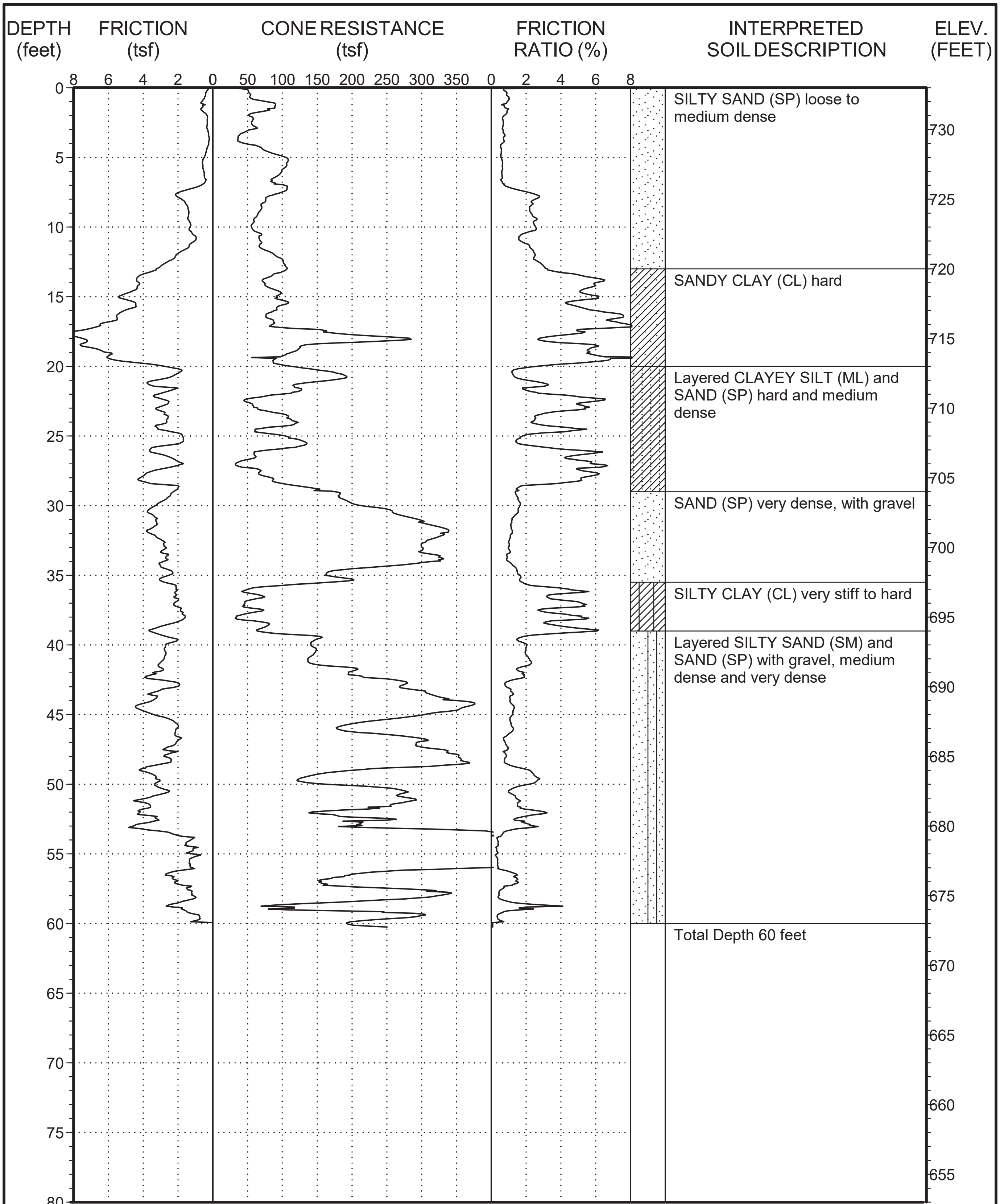
Date performed:1-24-19

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.: 2924.1I
MAGNOLIA FLATS

LOG OF CPT NO. C-4

FIGURE A-5



Date performed: 1-24-19

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.: 2924.1I

MAGNOLIA FLATS

LOG OF CPT NO. C-5

FIGURE A-6

APPENDIX B

APPENDIX B

EXPLORATORY BORINGS

The subsurface conditions at the site were investigated by drilling and sampling ten exploratory borings. The borings were advanced to depths of 21 to 61 feet below the existing ground surface. The location of the exploration is shown on the Site Plan, Figure 2.

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

At selected locations, disturbed samples were obtained using a split-spoon sampler by means of the Standard Penetration Test (SPT, ASTM D 6066). The spoon sampler was driven into the soil by a 140-pound hammer dropping 30 inches, employing 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. The borings were backfilled with soil cuttings. The ground surface elevations at the boring locations were estimated from a preliminary grading plan provided by KHR Associates and should be considered approximate.

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	*DESCRIPTION OF SUBSURFACE MATERIALS* This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
				B	0	Fill: **SILTY SAND (SM)** brown, dry, medium dense	
	3.6	110	15	D			
	6.3	100	11	D		@ 3 feet, slightly moist	725
					5		
	6.9	99	9	D		Natural: **SILTY SAND (SM)** brown, slightly moist, loose	
						SANDY SILT (ML) brown, slightly moist, firm	
	16.0	108	44	D	10	**SILTY CLAY (CL)** brown, moist, very stiff	720
	12.9	109	42	D		**CLAY (CL)** brown, moist, very stiff	715
	7.8	114	41	D	15	**SILTY SAND (SM)** brown, moist, medium dense	710
	7.0	106	59	D	20	@ 20 feet, light brown, slightly moist, dense	
						Total Depth 21 feet	

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

DATE DRILLED:
11-4-19

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2924.1I

MAGNOLIA FLATS

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, slightly moist, trace trash	
1.4	111	25	D		Natural: **SAND WITH SILT (SP-SM)** light brown, dry, medium dense	730
2.5	110	23	D	5	**SILTY SAND (SM)** brown, dry, medium dense	
					SAND (SP) light brown, dry, medium dense	
2.5	102	24	D		@ 7 feet, medium to coarse grained	725
22.1	98	19	D	10	**SILTY CLAY (CL)** brown, very moist, stiff	720
18.2	110	25	D	15	**CLAYEY SILT (ML)** brown, wet, very stiff	715
5.3 19.1	120 106	50/4"	D	20	**SAND WITH SILT (SP-SM)** light brown, slightly moist, very dense	
					Total Depth 21 feet	

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

DATE DRILLED:
11-4-19

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2924.1I

MAGNOLIA FLATS

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	8" Pulverized Concrete/ Gravel	
3.8	120	76/10"	D		Fill: **SILTY SAND (SM)** brown, dry, very dense	
3.4	116	50/6"	D		Natural: **SILTY SAND (SM)** brown, dry, very dense, trace cementation	
				5		730
2.3	104	50/5"	D		**SAND (SP)** brown, dry to slighty moist, very dense, coarse grained	
2.1	107	50/4"	D			
				10		725
1.7	83	50/3"	D		@ 12 feet, grey	
3.2	108	50/3"	D	15	@ 15 feet, trace silty sand	720
1.6	115	50/1"	D	20		715
		50/0.5"	D	25	@ 25 feet, no recovery	710
					Refusal @ 27 feet	

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

DATE DRILLED:
11-4-19

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2924.1I
MAGNOLIA FLATS

LOG OF BORING NO. B-3

FIGURE B-3

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
						This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	
				B	0	Pulverized concrete 8-Inch over silty sand, brown, moist	
						Fill: **SILTY SAND (SM)** brown, moist	
					5	5 to 35 feet, not sampled	730
					10		725
					15		720
					20		715
					25		710
					30		705
					35		700
	21.9		22	S		Natural: **CLAYEY SILT (ML)** brown, very moist, very stiff	
	19.9		19	S		**SANDY SILT (ML)** brown, wet, very stiff	
	2.4		42	S		**SAND (SP)** light brown, dry to slightly moist, dense	

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

DATE DRILLED:
10-31-19

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2924.1I

MAGNOLIA FLATS

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)
					40	**SAND WITH SILT (SP-SM)** light brown, dry, dense, medium to coarse grained	
	4.0		29	S		@ 41 feet, medium dense	
	2.3		28	S			
	2.7		34	S	45	@ 45 feet, dense, coarse grained	690
	9.9		38	S		@ 47 feet, fine to medium grained, trace gravel, 3-Inch lens of silt	
	9.4	99	77	D	50	@ 50 feet, very dense, sandy silt @ tip	685
						Total Depth 51 feet	

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

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

DATE DRILLED: 10-31-19

EQUIPMENT USED: 8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 2924.1I

MAGNOLIA FLATS

LOG OF BORING NO. B-4

FIGURE B-4

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS	ELEVATION (FEET)
					0	Pulverized concrete 8-Inch	
						Fill: **SILTY SAND (SM)** brown, slightly moist, dense	735
	5.6	119	23	D			
	2.4	101	18	D	5	Natural: **SAND (SP)** brown, dry to slightly moist, medium dense	730
	6.0	109	17	D		**SILTY SAND (SM)** brown, slightly moist, medium dense	
	9.8	107	19	D	10	@ 10 feet, moist	725
	6.4	108	85/11"	D	15	**SANDY SILT (ML)** brown, moist to very moist, hard	720
						SAND (SP) light brown-orange, dry to slightly moist, very dense, medium to coarse grained	
	3.2	104	50/3"	D	20		715
	12.9		75/9"	D	25	@ 25 feet, very moist	710
			50/1"	D	30	@ 30 feet, no recovery	705
						Refusal @ 32 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:
11-1-19

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2924.1I

MAGNOLIA FLATS

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	6-Inch Pulverized Concrete	
5.5	125	29	D		Fill: **SILTY SAND (SM)** brown, slightly moist, medium dense	
5.5	113	15	D			
				5		730
8.8	108	14	D		Natural: **SILTY SAND (SM)** brown, moist, loose	
8.5	102	14	D			
				10		725
4.2	106	17	D		@ 12 feet, dry to slightly moist	
					CLAYEY SILT (ML) brown, moist to very moist, stiff, trace sand	
15.7	108	22	D	15		720
21.3	109	48	D	20	**CLAY (CL)** light brown, very moist, hard	715
25.4	96	43	D	25	@ 25 feet, wet	710
2.9	117	76	D	30	**SILTY SAND (SM)** brown, dry, very dense	705
					SAND (SP) light brown, dry to slightly moist, very dense	
3.1	105	76/11"	D	35		700

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

DATE DRILLED:
11-1-19

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2924.1I

MAGNOLIA FLATS

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)
3.2	102	41	D	40	**SAND (SP)** light brown, dry to slightly moist, medium dense	
					Total Depth 41 feet	

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

DATE DRILLED: 11-1-19
EQUIPMENT USED: 8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 2924.1I
MAGNOLIA FLATS

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)
				B	0	1-Inch AC Fill: **SILTY SAND (SM)** brown, moist	
							730
					5	5 to 35 feet, not sampled	
							725
					10		
							720
					15		
							715
					20		
							710
					25		
							705
					30		
							700
	2.0		27	S	35	**SAND WITH SILT (SP-SM)** light brown, dry to slightly moist, medium dense, coarse grained	
	3.4		28	S		@ 37 feet, medium to coarse grained	695
	3.9		19	S			

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

DATE DRILLED:
10-30-19
EQUIPMENT USED:
8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft):
57

GPI

PROJECT NO.: 2924.1I
MAGNOLIA FLATS

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	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.	
23.8				40	**SILT (ML)** brown, wet, very stiff	
20.0		22	S		**SILTY SAND (SM)** light brown, wet, medium dense, fine to medium grained	
4.2		31	S		**SAND (SP)** light brown, dry to slightly moist, dense, coarse grained @ 44 feet, 4-Inch lens of silty sand	690
2.3		43	S	45		
2.6		31	S			685
3.7		43	S	50		
						680
5.2	125	71	D	55	**SANDY SILT (ML)** brown, dry, hard	
					SAND (SP) light brown, slightly moist, very dense, coarse grained	675
32.8	93	88/9"	D	60	**SILT (ML)** brown, wet, hard	
					SAND (SP) orange brown, wet, very dense	
					Total Depth 61 feet	

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

DATE DRILLED:
10-30-19

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
57

GPI

PROJECT NO.: 2924.1I

MAGNOLIA FLATS

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	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	2-Inch AC	
5.3	118	19	D		Fill: **SILTY SAND (SM)** brown, slightly moist, medium dense, medium to coarse grained	730
2.6	111	12	D		@ 3 feet, dry	
				5		
12.9	105	13	D		Natural: **SANDY SILT (ML)** brown, moist, stiff	725
10.1	104	13	D	10		
14.7	107	13	D			720
16.1	113	44	D	15	**CLAY (CL)** brown, moist, very stiff	715
14.7	110	64	D	20		710
9.9 2.5	105 112	51	D	25	**SAND WITH SILT (SP-SM)** light brown, dry to slightly moist, dense, medium to coarse grained	705
4.3	107	46	D	30		
					Total Depth 31 feet	

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

DATE DRILLED:
11-4-19

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2924.1I

MAGNOLIA FLATS

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)
				0	Fill: **SILTY SAND (SM)** brown, dry	735
6.1	100	32	D		Natural: **SILTY SAND (SM)** brown,slightly moist, medium dense, trace porosity	
2.4	111	21	D	5	@ 5 feet, dry, medium to coarse grained	730
6.6	90	25	D		@ 7 feet, slightly moist, fine to medium grained	
3.5	105	14	D	10		725
					SILTY CLAY (CL) brown, very moist, very stiff	
18.2	108	37	D	15		720
13.0	121	89	D	20		715
					SILTY SAND (SM) light brown, moist to very moist, very dense	
19.1	102	33	D	25	**CLAY (CL)** light brown, very moist, very stiff	710
5.5	116				**SAND (SP)** light brown, slightly moist, medium dense	
2.3	108	61	D	30	@ 30 feet, dry, dense, coarse grained, trace gravel	705
2.7	103	64	D	35		700
						695

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: 11-4-19
EQUIPMENT USED: 8 " Hollow Stem Auger
GROUNDWATER LEVEL (ft): Not Encountered

GPI

PROJECT NO.: 2924.1I
MAGNOLIA FLATS

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.	
3.3	113	62	D	40	**SAND (SP)** light brown, slightly moist, dense, interbedded silt lenses	695
24.7	99	50/6"	D	45	**SANDY SILT (ML)** brown, wet, hard	690
4.1	118				**SAND (SP)** light brown grey, slightly moist, very dense	
2.5	105	53	D	50	@ 50 feet, dry, dense, coarse grained	685

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

DATE DRILLED:
11-4-19

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2924.1I

MAGNOLIA FLATS

LOG OF BORING NO. B-9

FIGURE B-9

	MOISTURE (%)	DRY DENSITY (PCF)	PENETRATION RESISTANCE (BLOWS/FOOT)	SAMPLE TYPE	DEPTH (FEET)	DESCRIPTION OF SUBSURFACE MATERIALS This summary applies only at the location of this boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	ELEVATION (FEET)
				B	0	3-Inch AC over silty sand, brown, slightly moist Fill: **SILTY SAND (SM)** brown, slightly moist	
							730
					5	5 to 35 feet, not sampled	
							725
					10		
							720
					15		
							715
					20		
							710
					25		
							705
					30		
							700
	2.8		23	S	35	Natural: **SAND (SP)** light brown, dry to slightly moist, medium dense, medium to coarse grained	
	16.5		15	S		**SILTY SAND (SM)** brown, wet, medium dense	
							695
	1.7		19	S		**SAND (SP)** light brown, dry, medium dense	

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

DATE DRILLED:
10-30-19

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2924.1I

MAGNOLIA FLATS

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)
					40	**SILT (ML)** grey, wet, very stiff	
	34.2		10	S			
	11.0		21	S		@ 43 feet, 4-Inch lens of grey sand	690
	18.6		35	S	45	**CLAYEY SAND (SC)** grey, wet, dense	
	4.2					**SAND WITH SILT (SP-SM)** grey, slightly moist, dense, coarse grained	
	3.4		39	S		@ 47 feet, trace gravel	
							685
	3.1	112	64	D	50		
						Total Depth 51 feet	

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

DATE DRILLED:
10-30-19

EQUIPMENT USED:
8 " Hollow Stem Auger

GROUNDWATER LEVEL (ft):
Not Encountered

GPI

PROJECT NO.: 2924.1I

MAGNOLIA FLATS

LOG OF BORING NO. B-10

FIGURE B-10

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.

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.

PERCENT PASSING NO. 200 SIEVE

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

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	PERCENT PASSING No. 200 SIEVE
B-3	3	Silty Sand (SM)	17
B-4	37	Sandy Silt (ML)	59
B-4	41	Sand w/Silt (SP-SM)	6
B-7	37	Sand w/Silt (SP-SM)	8
B-7	41	Silty Sand (SM)	42
B-7	45	Sand (SP)	4
B-8	3	Silty Sand (SM)	20

-continued-

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	PERCENT PASSING No. 200 SIEVE
B-8	30	Sand (SP)	5
B-10	37	Silty Sand (SM)	45
B-10	41	Silt (ML)	96
B-10	45	Silty Sand (SM)	42
B-10	45½	Sand (SP-SM)	7

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 95 percent of the maximum dry density (ASTM D 1557). The samples were placed in the shear machine, and a normal load comparable to the in-situ overburden stress was applied. The samples were inundated, allowed to consolidate, and then were sheared to failure. The tests were repeated on additional test specimens under increased normal loads. Shear stress and sample deformation were monitored throughout the test. The results of the direct shear tests are presented in Figures C-3 and C-4.

COLLAPSE

Collapse tests were performed on relatively undisturbed samples in accordance with ASTM D 5333. After trimming the ends, the sample was placed in the consolidometer and loaded to 0.4 ksf. Thereafter, the samples were incrementally loaded to 1.6 ksf 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-5	10	Silty Sand (SM)	9.8	2.7	3.9
B-6	6	Silty Sand (SM)	8.8	3.6	4.5
B-8	6	Sandy Silt (ML)	12.9	2.8	3.2
B-8	12	Sandy Silt (ML)	14.7	2.1	2.2

CONSOLIDATION

One-dimensional consolidation tests were performed on undisturbed samples in accordance with ASTM D 2435. After trimming the ends, the sample was placed in the consolidometer and loaded to up to 0.4 ksf. Thereafter, the sample was incrementally loaded to a maximum load of up to 25.6 ksf. The sample was 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-5 and C-6.

COMPACTION TEST

Maximum dry density/optimum moisture tests were performed in accordance with ASTM D 1557 on representative bulk samples of the surficial soils. The test result is as follows:

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	MAXIMUM DRY DENSITY (pcf)	OPTIMUM MOISTURE (%)
B-3	0-5	Silty Sand (SM)	136	8.0

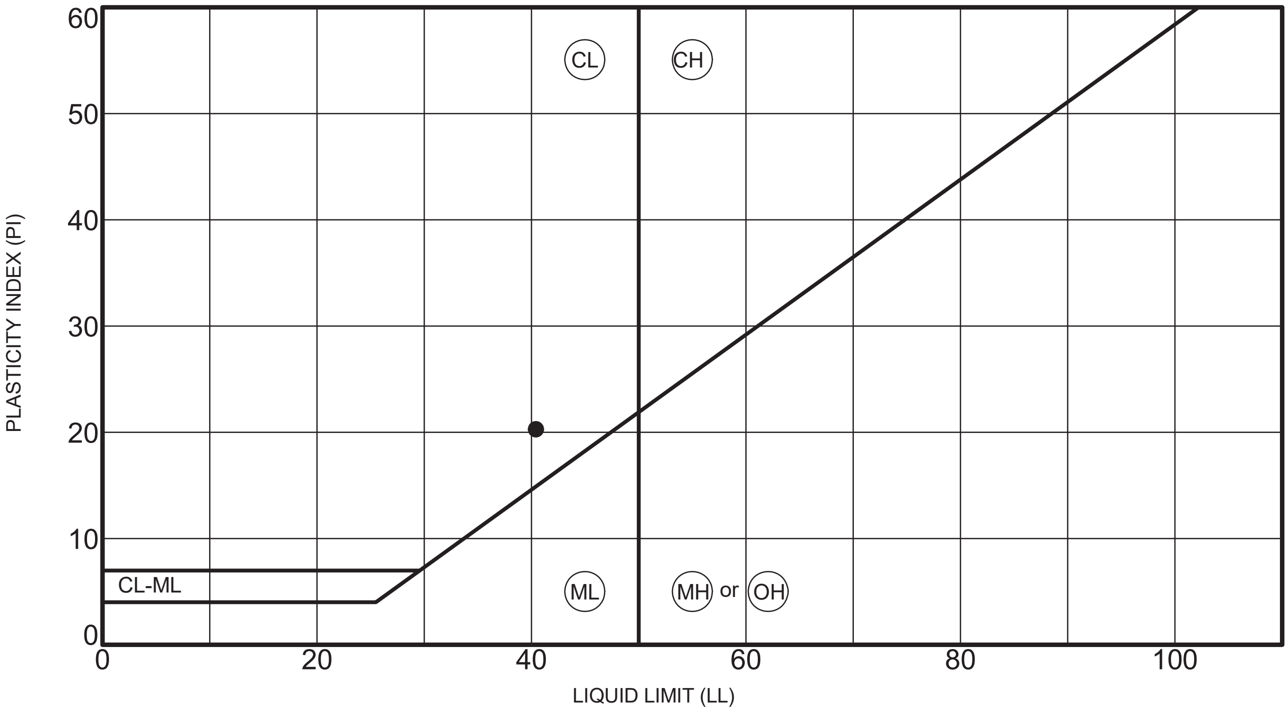
R-VALUE

Suitability of the near-surface soils for pavement was evaluated by conducting an R-value test. The test was performed in accordance with ASTM D2844. The results of the test are as follows:

BORING NO.	DEPTH (ft)	SOIL DESCRIPTION	R-VALUE
B-1	0-5	Silty Sand (SM)	61

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-6	20.0	40	20	20		CLAY (CL)

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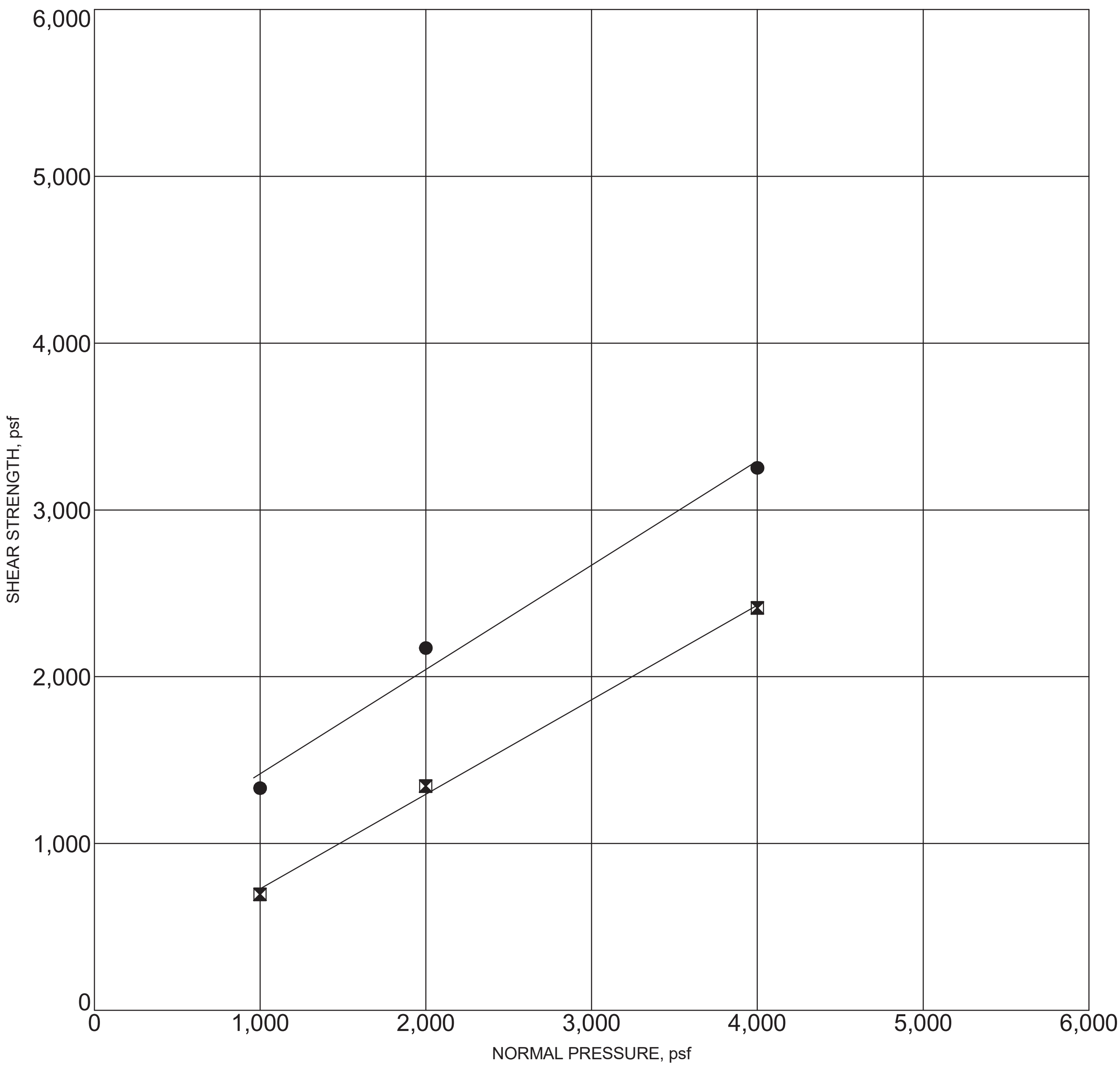
ATTERBERG LIMITS TEST RESULTS

FIGURE C-1



● **PEAK STRENGTH**
Friction Angle= 32 degrees
Cohesion= 792 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 30 degrees
Cohesion= 162 psf

Note: Samples remolded to 95% of maximum dry density

Sample Location		Classification	DD,pcf	MC,%
B-3	0-5	SILTY SAND (SM)	129	8.0

PROJECT: MAGNOLIA FLATS

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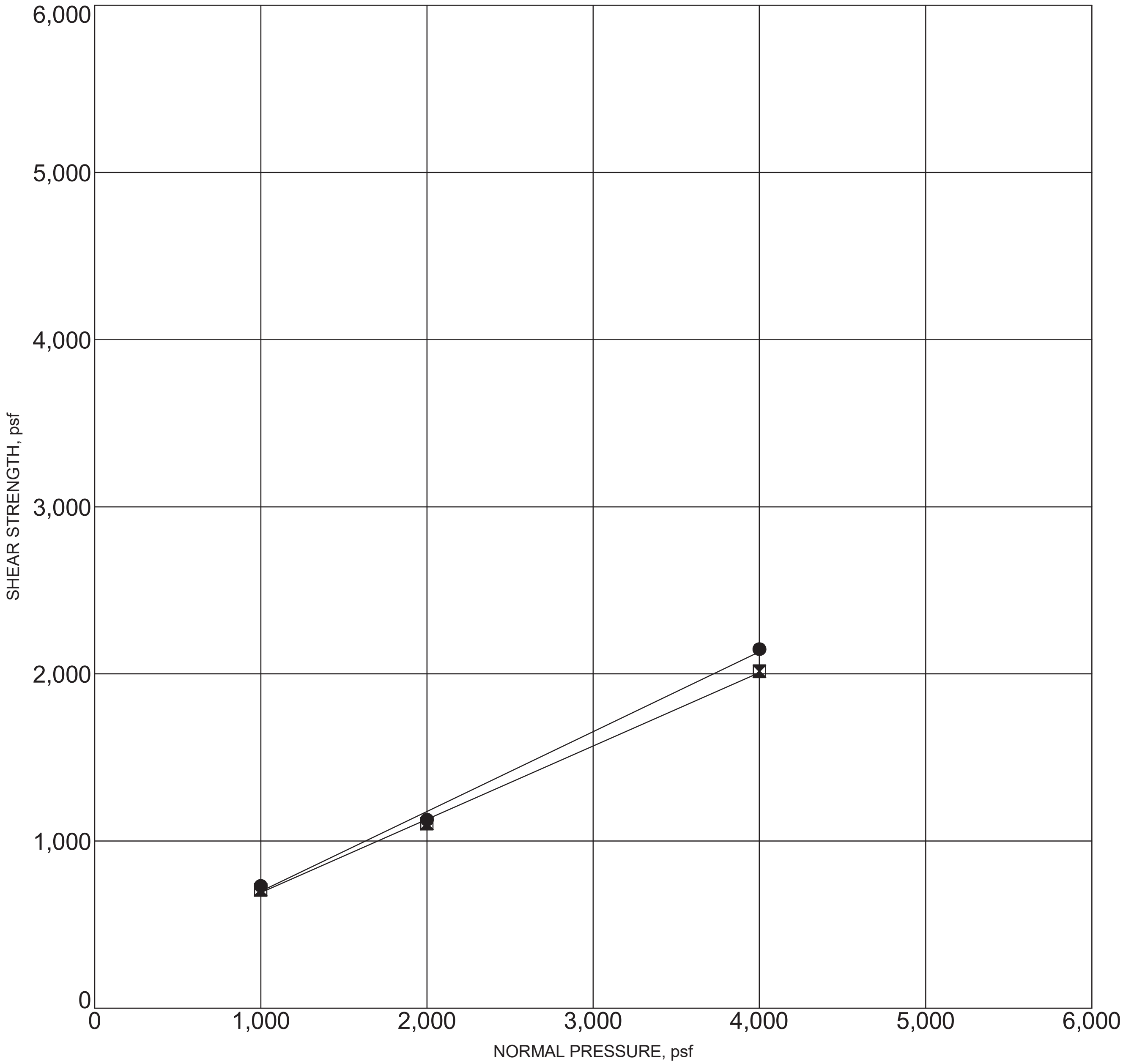
DIRECT SHEAR TEST RESULTS

FIGURE C-2



● **PEAK STRENGTH**
Friction Angle= 26 degrees
Cohesion= 222 psf

⧗ **ULTIMATE STRENGTH**
Friction Angle= 24 degrees
Cohesion= 252 psf

Sample Location		Classification	DD,pcf	MC,%
B-5	7.0	SILTY SAND (SM)	109	6.0

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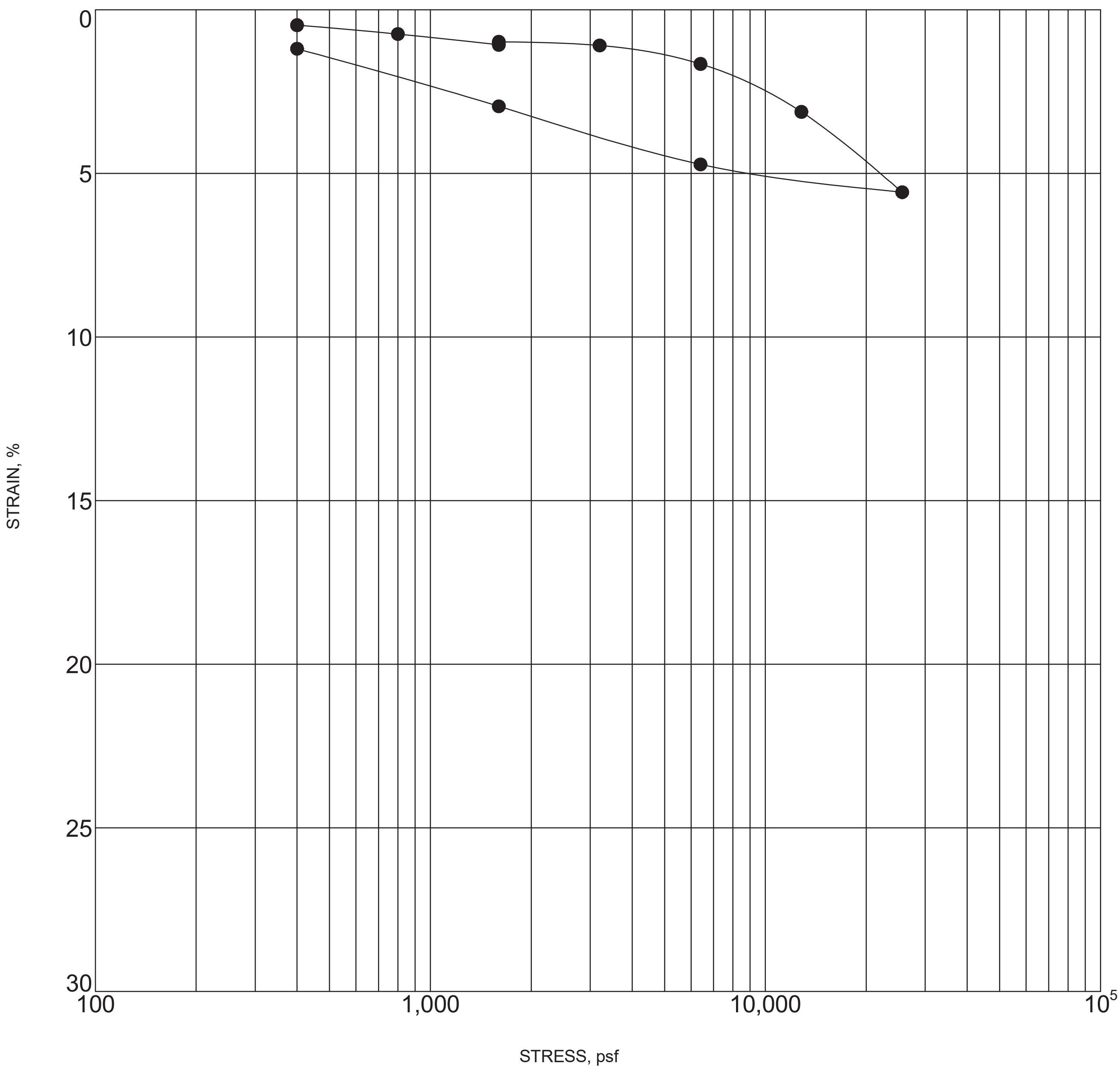
DIRECT SHEAR TEST RESULTS

FIGURE C-3



Sample inundated at 1600 psf

	Sample Location		Classification	DD,pcf	MC,%
●	B-6	20.0	CLAY (CL)	109	21.3

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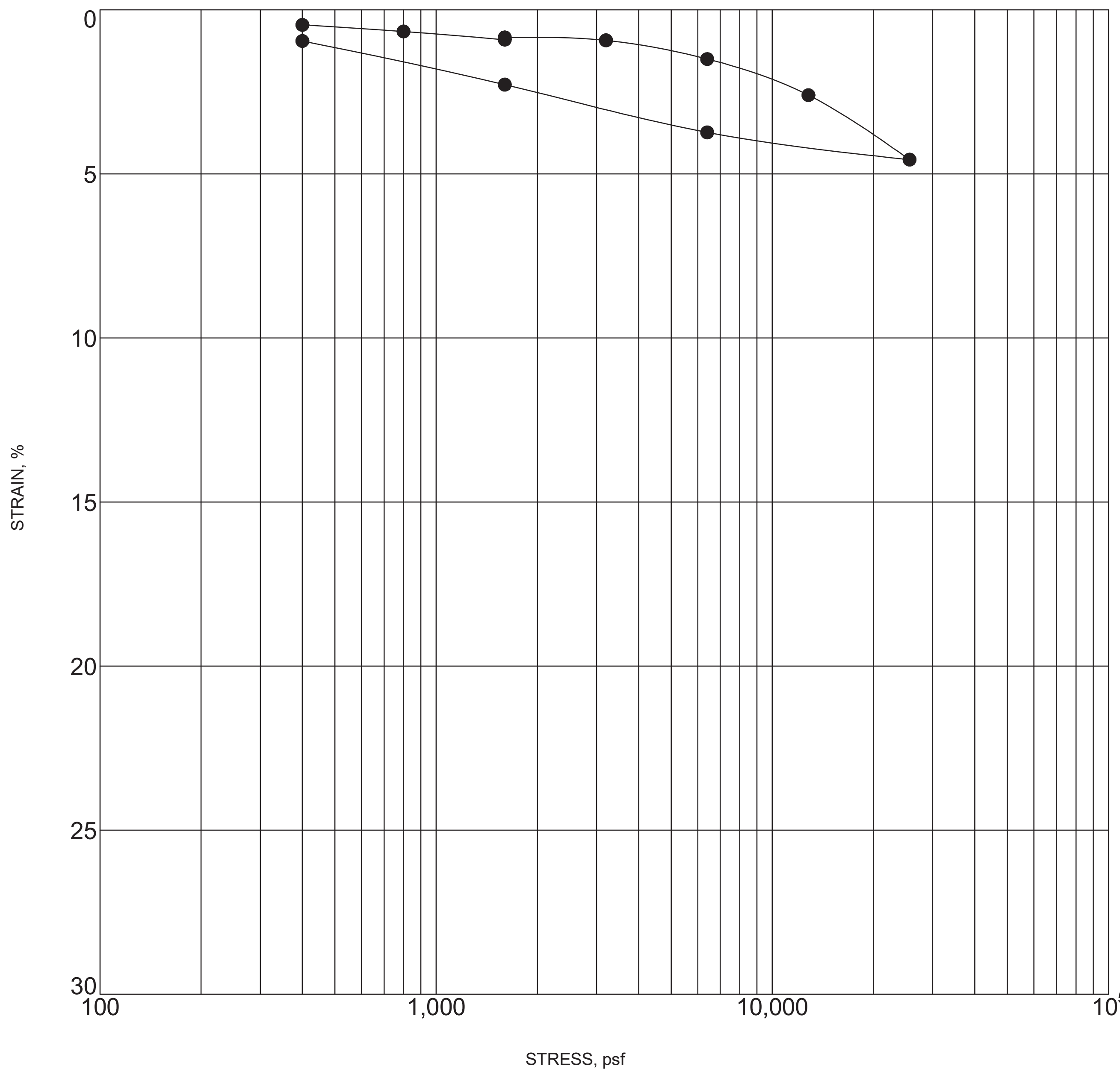
CONSOLIDATION TEST RESULTS

FIGURE C-4



Sample inundated at 1600 psf

	Sample Location		Classification	DD,pcf	MC,%
●	B-8	15.0	CLAY (CL)	113	16.1

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CONSOLIDATION TEST RESULTS

FIGURE C-5

Table 1 - Laboratory Tests on Soil Samples

Geotechnical Professionals, Inc.
Magnolia Flats
Your #2924.1I, HDR Lab #19-0783LAB
12-Nov-19

Sample ID			B-3 @ 0-5'
Resistivity		**Units**	
as-received		ohm-cm	38,400
saturated		ohm-cm	8,400
pH			8.6
Electrical Conductivity		mS/cm	0.07
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	53
magnesium	Mg^{2+}	mg/kg	8.4
sodium	Na^{1+}	mg/kg	50
potassium	K^{1+}	mg/kg	3.2
Anions			
carbonate	CO_3^{2-}	mg/kg	ND
bicarbonate	HCO_3^{1-}	mg/kg	180
fluoride	F^{1-}	mg/kg	3.7
chloride	Cl^{1-}	mg/kg	ND
sulfate	SO_4^{2-}	mg/kg	25
phosphate	PO_4^{3-}	mg/kg	ND
Other Tests			
ammonium	NH_4^{1+}	mg/kg	ND
nitrate	NO_3^{1-}	mg/kg	4.5
sulfide	S^{2-}	qual	na
Redox		mV	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

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