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Geotechnical Report

HAVEN HILL ACRES
COLLINSVILLE, ILLINOIS

November 2024

HAVEN HILL ACRES, LLC
Representative

JRG HOLDINGS COMPANY
Developer

THOUVENOT WADE & MOERCHIN, INC.
Civil Engineer

ARCHITECTURAL ASSOCIATES INCORPORATED, INC.
Architect

FENDRICH ENGINEERING, INC.
Structural Engineer

SCI No. 2023-1906.10

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11/22/2024

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November 22, 2024

James Mundloch
Haven Hill Acres, LLC
518 Leonard Avenue
St. Louis, Missouri 63119

RE: Geotechnical Report
Haven Hill Acres
Collinsville, Illinois
SCI No. 2023-1906.10

Dear James Mundloch:

Attached is SCI Engineering, Inc.'s (SCI's) *Geotechnical Report*, dated November 2024. It should be read in its entirety, and our recommendations applied to the design and construction of the project. Selected excerpts from the report are highlighted below:

- Silt was encountered throughout the native soils onsite. These soils are susceptible to excessive pumping or rutting with disturbance such as that caused by construction traffic. Measures should also be taken to limit traffic over them, particularly during the wetter portions of the year. Construction of haul roads should be considered to minimize damage by limited traffic to areas designed to support those loads. In some cases, silty soils with low plasticity indices do not react well with hydrated lime. As such, consideration should be given to using cement to create a stable working platform. We would recommend a contingency fund be established to provide stabilization methods listed above throughout the duration of the project, particularly if construction will take place during the wetter times of the year.

- Shallow spread foundations can be sized for maximum net allowable bearing pressures of 2,000 pounds per square foot (psf) for both continuous wall footings and isolated spread footings.

- The site is classified as Site Class D with the following seismic design parameters: $F_a = 1.43$, $F_v = 2.28$, $S_{DS} = 0.44$, and $S_{D1} = 0.24$, resulting in a Seismic Design Category (SDC) of D.

We appreciate the opportunity to be of service and look forward to working with you during the construction phase of the project. If you have any questions or comments, please do not hesitate to contact me.

I can be reached at 314-780-3600 or gkowalik@sciengineering.com.

Respectfully,

SCI ENGINEERING, INC.

Gabrielle M. Kowalik
Staff Engineer

Julie A. Miller, P.E.
Senior Geotechnical Engineer

GMK/JAM/hgs

Enclosure
Geotechnical Report

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HAVEN HILL ACRES
COLLINSVILLE, ILLINOIS

1.0 INTRODUCTION

At the request of James Mundloch of Haven Hill Acres, LLC, SCI Engineering, Inc. (SCI) performed a geotechnical study for the proposed multi-family development. The purpose of our exploration was to characterize and evaluate the subsurface conditions, provide recommendations for foundations and pavements, and address other geotechnical aspects for the project. Our services were provided in general accordance with our proposal, dated July 18, 2024, and authorized on September 25, 2024.

2.0 SITE AND PROJECT DESCRIPTION

A multi-family development is planned for three parcels located near Ramada Boulevard and Reese Drive in Collinsville, Illinois. The Madison County Assessor's Office identifies these parcels as the following:

- Parcel Number 13-1-21-29-12-201-006, a vacant parcel addressed as Reese Drive, and is 14.4000-acres;
- Parcel Number 13-1-21-29-12-201-006.002, a vacant parcel addressed as Ramada Boulevard and is 6.6118-acres; and
- Parcel Number 13-2-21-29-12-201-002, a vacant parcel addressed as Ramada Boulevard.

The location of the site is shown on the *Vicinity and Topographic Map*, Figure 1. The site is currently undeveloped and densely vegetated/wooded. Based on the existing site topography the site slopes from the southwest downwards to the northeast with approximately 80 feet of relief. Existing site conditions are shown on the *Aerial Photograph*, Figure 2.

Based on the *concept plan* provided within the Haven Hill Acres Architectural Packet, and dated October 7, 2024, and provided by Architectural Associates Incorporated, Inc. (AAIC), the proposed development will consist of four three to four-story buildings located along the southwestern portion of the site. Associated parking lots are proposed between the buildings and one new drive lane will connect to Ramada Boulevard. We understand that future development may occur on the northern portion of the site, but that is currently outside of our scope of work.

Finished floor elevations provided by AAIC and approximate required cut and fill for the apartment buildings are shown in Table 2.1 below.

Table 2.1 – Finished Floor Elevation and Cut/Fill Summary

Apartment Building	Provided Finished Floor Elevation	Approximate Fill Depth to Reach FFE (feet)	Approximate Cut Depth to Reach FFE (feet)
Building #1	547	2 to 10	N/A
Building #2	547	N/A	0 to 2
Building #3	547	0 to 2	0 to 3
Building #4 (First Floor) *	547	N/A	0 to 7
Building #4 (Basement)*	539	0 to 3	0 to 5

*Transition between first floor and basement level was not available. Fill and cut depths are estimates.

The structural loads provided by AAIC indicate wall loads will be on the order of 2200 to 3340 pounds per lineal foot. The proposed construction is shown on the *Site Plan,* Figure 3.

We are not aware of any previous studies on this specific site, by SCI or others, that would affect the preparation of this report.

3.0 SUBSURFACE CONDITIONS

Eleven Piezocone Penetration Test (CPTu) soundings (B-1 through B-11) were advanced at the approximate locations shown on the *Aerial Photograph* and *Site Plan*. To supplement the CPTu data and aid in classification, Continuous Direct Push (CDP) samples were taken adjacent to each of the soundings. The testing locations were staked in the field by SCI personnel using a handheld global positioning system. Approximate ground surface elevations at the testing locations were interpolated from the *Concept Plan* by TWM, Inc. Detailed information regarding the nature and thickness of the soils encountered, and the results of the field sampling and laboratory testing are shown on the *CPTu Logs* in Appendix A and *CDP Logs* in Appendix B.

3.1 Native Soil

The native soils consisted of interbedded layers of silt (ML in accordance with Unified Soil Classification System and ASTM D-2487) and lean clay extending to the maximum depth of sounding termination at 40 feet.

The tip resistance (qt) within the native soil ranged from 2 to 127 tsf, averaging 60 tsf, classifying the soil as soft to hard in consistency. Associated correlated Standard Penetration Test (SPT) N-values within the clay soils generally ranged from 1 to 35 blows per foot (bpf), with an average of 17 bpf. In general, the native soils were soft in the upper 2 feet, then became very stiff on average with occasional soft and hard zones at depths of 25 or deeper. Moisture contents ranged from 4 to 21 percent, averaging approximately 13 percent.

3.2 Groundwater

Groundwater was not observed in any of the direct push borings. CPTu soundings suggest groundwater may be present in the other four of the eleven soundings at depths of 12 to 30 feet as summarized in Table 3.1. The groundwater level depends on seasonal and climatic variations and may be present at different depths in the future. In addition, without extended periods of observation, accurate groundwater level measurements may not be possible, particularly in low permeability soils.

Table 3.1 – Groundwater Summary

Boring	Approximate Surface Elevation (feet)	Approximate Groundwater Depth (feet)	Approximate Groundwater Elevation (feet)
B-1	551	NE	--
B-2	546	NE	--
B-3	554	30	524
B-4	550	26	524
B-5	543	NE	--
B-6	541	20	521
B-7	548	NE	--
B-8	547	NE	--
B-9	547	NE	--
B-10	546	12	534
B-11	550	NE	--

3.3 Site Geology

Documented geology, including the *Geologic Map of Illinois* indicated the bedrock at the site consists of Pennsylvanian-aged, undivided, Shelburn-Patoka Formations. The Shelburn-Patoka Formations are characterized by shale, sandstone, clay, limestone, and coal.

3.4 Underground Mining Research

The ISGS *Directory of Coal Mines in Illinois, Monks Mound Quadrangle,* dated 2020, indicated the site is undermined by the Collinsville Mine (ISGS No. 2768) as shown on the *ISGS Mine Map,* Figure 4. SCI previously discussed the risks related to undermining for this project site in our *Undermining Assessment* dated January 5, 2024. In summary, there is a risk of mine subsidence associated with the Collinsville Mine impacting the project site. However, it is difficult to fully quantify this risk without information regarding the current condition of the mine. Based on subsidence history near the project site, the risk for mine subsidence is high.

4.0 DESIGN RECOMMENDATIONS

4.1 Silty Soil Considerations

Silt was encountered throughout the native soils onsite. These soils are susceptible to excessive pumping or rutting with disturbance such as that caused by construction traffic. Some of the silty soils observed are 5 to 10 percent above anticipated optimum moisture contents to achieve compaction. As such, these soils will need additional drying prior to placement as fill. During drier times of the year, these soils can be disced and allowed to air dry. In wetter times of the year, additives will be needed to help the soils dry.

Measures should also be taken to limit traffic over them, particularly during the wetter portions of the year, such as limiting proofroll passes, use of smooth-edged buckets for excavation, and use of a pump truck to place concrete. Disturbed areas should be undercut and replaced with properly compacted structural fill. A combination of crushed rock and geogrid reinforcement could be used on particularly unstable areas. Construction of haul roads will minimize damage by limiting traffic to areas designed to support such loads. For estimating purposes, the contractor may elect to place a geogrid on the underlying subgrade with 12 inches of 2-inch-clean crushed stone (IDOT gradation RR-1). Stabilization of the soils can also be performed as further discussed below.

In some cases, silty soils with low plasticity indices do not react well with hydrated lime. As such, consideration should be given to using cement to create a stable working platform. Typically, cement can be used to stabilize the silty soils by adding it at a rate of 4 to 8 percent. Mud mats can be utilized at the base of the basement foundation excavations during the wetter times of the year to reduce the potential for disturbance of the foundation subgrade soil. We would recommend a contingency fund be established to provide stabilization methods listed above throughout the duration of the project, particularly if construction will take place during the wetter times of the year.

In addition, silty soils are very susceptible to erosion. Care should be taken to provide proper erosion control for the development to reduce the potential for damage related to erosion.

4.2 Shallow Foundations

Shallow spread footing foundations bearing on newly placed, low plastic structural fill or native lean clay or silt, are appropriate for support of the proposed building. Based on the soils encountered during our exploration, the shallow foundations can be sized for a maximum net allowable bearing pressure of 2,000 pounds per square foot (psf) for both continuous spread footings and isolated column pads. Foundations may be designed with an ultimate coefficient of friction between the base of the concrete footing and the soil subgrade of 0.3.

We anticipate that localized areas of inadequate bearing materials may be encountered during construction and, subsequently, require remediation to achieve this capacity. If encountered, inadequate bearing materials should be undercut and replaced properly by engineered fill in accordance with Sections 5.1 and 5.2.

Exterior footings, as well as foundations in unheated areas of the building, should be provided with at least 30 inches of soil cover for frost protection. Interior footings in heated areas can be located at nominal depths below the finished floor. For footings designed and constructed in accordance with our recommendations, total settlement should be less than 1 inch, and differential settlement between adjacent footings should be less than ¾ inch.

4.3 Seismic Considerations

Ground shaking at the foundation of structures and liquefaction of the soil under the foundation are the principle seismic hazards to be considered in design of earthquake-resistant structures. Liquefaction occurs when a rapid buildup in water pressure, caused by the ground motion, pushes sand particles apart, resulting in a loss of strength and later densification as the water pressure dissipates. This loss of strength can cause bearing capacity failure, while the densification can cause excessive settlement. Potential earthquake damage can be mitigated by structural and/or geotechnical measures or procedures common to earthquake resistant design.

4.3.1 Design Earthquake

According to International Building Code (IBC) 2021 edition, structures such as those proposed for this project are required to be designed to a design earthquake with a 2 percent Probability of Exceedance over a 50-year exposure period (i.e., a 2,475-year design earthquake).

4.3.2 International Building Code Site Classification

Seismic design parameters for the site as determined from data provided by the 2021 IBC and the United States Geological Survey National Seismic Hazard Mapping Project are shown in Table 4.1.

Table 4.1 – Seismic Design Parameters

Site Class	D
M_w	5.84
PGA	0.27
F_{PGA}	1.34
Site Modified PGA_M	0.35
S_S	0.46
S_1	0.16
F_a	1.43
F_v	2.28
S_{DS} (Design Spectral Acceleration at 0.2 sec)	0.44
S_{D1} (Design Spectral Acceleration at 1.0 sec)	0.24
Seismic Design Category	D

4.3.3 Liquefaction Potential Analysis

The liquefaction potential analysis for the site was conducted using data from the field exploration and laboratory test results and the techniques outlined in the National Center for Earthquake Engineering (NCEER) Technical Report NCEER-97-0022. Based on our analyses, the soils at the project site have sufficient strength values to resist liquefaction and/or a plasticity index that make the threat of liquefaction minimal during the design earthquake. While the amount of the seismically induced settlement is dependent on the magnitude and distance from the seismic event, we estimate that the settlements from the design earthquake will be negligible and relatively uniform in nature, so liquefaction mitigation techniques are not required.

4.4 Floor Slabs

The following recommendations assume an average uniform floor load of less than 150 psf. If sections of the floor slabs will support higher loads, the underlying soils below these sections may need to be removed and replaced with compacted/engineered fill. If the proposed buildings will include more heavily loaded floor slab sections, SCI should be provided the opportunity to review the final design plans and specifications to determine if the underlying subsurface soils can adequately support the loads. Proofrolling, as discussed in Section 5.1, should be accomplished to identify soft or unstable soils that should be removed from the floor slab areas prior to fill placement and/or floor slab construction.

We recommend that the floor slabs be designed using a modulus of subgrade reaction, *k* value, of 150 pounds per cubic inch based on values typically obtained from 1-foot by 1-foot plate load tests. This value assumes the slabs will bear on newly placed, low plastic structural fill or suitable native soils. Depending on how the slab load is applied, the value will have to be geometrically modified. The value should be adjusted for larger areas using the following expression for cohesive and cohesionless soil:

Modulus of Subgrade Reaction, $k_s = \left(\frac{k}{B}\right)$ for cohesive soil and

$k_s = k\left(\frac{B+1}{2B}\right)^2$ for cohesionless soil.

Where: k_s = coefficient of vertical subgrade reaction for loaded area;
k = coefficient of vertical subgrade reaction for 1x1 square foot area; and
B = width of area loaded, in feet.

The floor slabs should be supported on a minimum 4-inch-thick layer of crushed stone. This will help to distribute concentrated loads and equalize moisture conditions beneath the slab. There likely will be some differential settlement between the foundations and the floor slabs. It is generally preferable to maintain structural separation between the floor slabs and the foundation walls and column pads using isolation joints. We also suggest that joints be placed in the floor slabs with spacing (in feet) equal to approximately three times the thickness of the slab (in inches) in both directions. Such joints permit slight movements of the independent elements and help reduce random cracking that might otherwise be caused by restraint of shrinkage, slight rotations, heave, or settlement.

Where occupied space or moisture sensitive floor coverings are planned, we recommend a 6-mil-thick polyethylene sheeting be placed immediately beneath the floor slabs and above the crushed rock or gravel to reduce the transfer of capillary moisture to the slab. However, without careful attention to curing of the floor slab, the polyethylene sheeting can cause excessive shrinkage cracking and "curling."

The precautions listed below should be followed for construction of slab-on-grade pads. These details will not reduce the amount of movement but are intended to reduce potential damage should some settlement of the supporting subgrade take place. Some increase in moisture content is inevitable as a result of development and associated landscaping. However, extreme moisture content increases can be largely controlled by proper and responsible site drainage, building maintenance and irrigation practices.

- Cracking of slab-on-grade concrete is normal and should be expected. Cracking can occur not only as a result of heaving of the supporting soil, but also as a result of concrete curing stresses. The occurrence of concrete shrinkage cracking, and problems associated with concrete curing may be reduced and/or controlled by limiting the slump of the concrete, proper concrete placement, finishing, curing, and by the placement of crack control joints at frequent intervals, particularly where re-entrant slab corners occur. The American Concrete Institute recommends a maximum panel size (in feet) equal to approximately three times the thickness of the slab (in inches) in both directions. For example, joints are recommended at a maximum spacing of 12 feet, based on having a 4-inch slab. SCI also recommends that the slab be independent of the foundation walls.

- Areas supporting slabs should be properly moisture conditioned and compacted. Backfill in all interior and exterior water and sewer line trenches should be carefully compacted to reduce the shear stress in the concrete extending over these areas.

Exterior slabs should be isolated from the building. These slabs should be reinforced to function as independent units. Movement of these slabs should not be transmitted to the building foundation or superstructure.

4.5 Below-Grade Walls

Below-grade walls will include the basement walls in Building 4. Below-grade walls should be designed to withstand lateral earth pressures caused by the weight of the backfill, including slopes behind the walls; and any surcharge, such as adjacent floor loads. We recommend the equivalent fluid unit weights shown in Table 4.2 for lateral earth pressures, in pounds per cubic foot (pcf), be used in the design of below-grade walls. We typically recommend that positive drainage is provided to prevent buildup of hydrostatic pressure. If drainage cannot be provided, hydrostatic pressures will need to be taken into consideration. Fat clay should not be used to backfill the wall excavations. Values for granular material should only be used if the granular backfill extends upwards and outwards the full height of the wall at a slope of 45 degrees or flatter from its base. In this case, exterior granular backfill should be capped with approximately 2 feet of cohesive soil to reduce the potential for surface water infiltration into the granular backfill. With clean granular backfill, filter fabric, such as Mirafi 140N, should be placed along the interface between the soil and granular backfill to reduce the potential for infiltration of the soil into the granular material.

Table 4.2 – Recommended Lateral Earth Pressures

Backfill Type	Equivalent Fluid Unit Weights	
	At-Rest Earth Pressures (pcf)	Active Earth Pressures (pcf)
Cohesive Soil	70	50
Granular Material (1-inch minus)	60	40
Free-Draining, Granular Material (1-inch clean)	50	30

At rest earth pressures should be used for restrained or fixed-head walls that are restricted from rotation, such as loading dock or basement walls connected to floor joists or beams, or a wing wall attached to a basement wall. Active earth pressures should be used for free-head walls where the base remains fixed and deflection at the top of the wall of approximately 1 inch for each 10 feet of wall height is allowed, such as a retaining wall.

The above values are applicable when the surface of the backfill behind the wall is horizontal. Upward sloped or loaded backfill will result in increased values. In addition to lateral earth pressures, below-grade walls should be designed to resist any surcharge loads, including shallow building foundations and traffic. These surface loads can be modeled as uniform lateral loads, equivalent to one-half of the surface load, acting at the halfway point on the wall.

A passive soil resistance modeled by an equivalent fluid unit weight of 250 pcf may be used for native soil against the face of the exterior base or a key below the base of the wall. The upper 2 feet of soil backfilled against the exterior face of the walls and uncontrolled backfill soils should be ignored when calculating the lateral resistance. Lower passive pressure should be used if the ground surface slopes downward away from the face of the wall.

We typically recommend that all below-grade walls be provided with a drainage system. If drainage of the walls is feasible then we recommend a minimum 4-inch diameter, perforated drainpipe should be used, and placed at foundation level. Granular drainage material, consisting of 1-inch clean crushed rock, classified as GP by ASTM D 2487, with less than 5 percent of the rock passing the No. 200 sieve, should be placed a minimum of 6 inches to each side and above the drainage pipe. Synthetic filter fabric, such as Mirafi 140N or equivalent, should encapsulate the drainpipe and granular drainage material. The pipe should be sloped to drain by gravity or through weepholes located on approximately 10-foot centers for above-grade retaining walls, or to a sump with a pump for below-grade walls where positive drainage by gravity cannot be achieved. Alternately, drainage can be provided directly through the weepholes without a drainpipe, provided that filter fabric is used or other measures are taken to prevent the granular backfill from migrating out through the weepholes. Any interior sumps must be isolated "watertight" from the interior subgrade to prevent the movement of moisture from the sump into the underlying soils.

4.6 Site Grading and Drainage

Positive site drainage should be provided to reduce surface water infiltration around the perimeter of the building and beneath the floor slab. All grades should be sloped away from the building. Roof and surface drainage should be collected and discharged such that water is not permitted to infiltrate the backfill of the building.

Large trees and shrubs should be planted away from exterior footings as they may cause drying and shrinkage of the foundation soils and, with the passage of time, potentially detrimental settlement of the building floor slab and foundations. A minimum distance of 20 feet or the mature tree's dripline, whichever is greater, is suggested.

We recommend that all final slopes have a maximum inclination of 3 horizontal to 1 vertical (3H:1V) and that a crest of at least 10 feet in width or a distance equivalent to the height of the slope, whichever is less, be provided around the buildings before the surface slopes down and away. Cut and fill slopes of up to 15 feet in height should perform satisfactorily at this inclination, or flatter. We do not anticipate that steeper or taller slopes will be required. However, if they are proposed, the slopes should be brought to our attention and individually addressed and evaluated by SCI on a case-by-case basis.

Natural slopes to receive fill which are steeper than 5H:1V should be benched prior to the placement of fill. Benching will provide level surfaces for compaction and reduce the potential for development of inclined planes of weakness between the natural soil and compacted fill. The benches should be spaced such that the maximum height of cut at the up-slope end of the bench is 5 feet.

4.7 Underground Utilities

Underground utilities can provide a pathway for water to migrate below the building. Drain and utility pipes beneath floors should have tight joints to prevent leakage. If utility excavations are backfilled with free-draining granular materials, then cutoffs should be provided at the exterior walls to reduce the potential for water to migrate beneath the building. Impermeable cutoffs may consist of a 3-foot-long "plug" of cohesive soil or bentonite soil mix, or a 1-foot-long plug of lean concrete. Soil may be used for the balance of the backfill.

Groundwater is not anticipated to influence utility excavation. However, in most situations, small amounts of groundwater seepage into the excavations can be handled by means of gravity ditching and a sump pump. If greater flows are experienced, SCI should be retained to review our recommendations, and provide additional recommendations regarding dewatering.

With the exception of individual service lines to the building that intersect foundations perpendicularly, below-grade utilities should not be located within the stress influence zone of the building foundations. Accordingly, below-grade utilities should be located outside a zone extending 45 degrees downward and outward from the edge of the footings.

4.8 Pavements

Selection of the pavement section is dependent on the design life, traffic loads, subgrade strength, drainage characteristics, and the desired level of maintenance. Neither California Bearing Ratio testing, nor formal pavement design was a part of our scope for this project. However, for planning purposes, the following recommendations typically result in pavements that perform satisfactorily on similar subgrades. They are intended to roughly provide a pavement requiring routine maintenance for a 5-year period, minor repair and maintenance during the 5- to 10-year life of the pavement, and possibly major repairs and restoration after a 10-year service life.

4.8.1 Flexible Pavement

A flexible pavement section may be used for the parking lot and drives, as summarized in Table 4.3.

Table 4.3 – Recommended Flexible Pavement Thickness

Pavement Layer	Thickness (inches)	
	Automobile and Light Truck Parking Stalls	Drive Lanes
Asphaltic Concrete (IL-9.5)	3	3
Aggregate Base (CA-6)	6	8

4.8.2 Rigid Pavement

Alternately, a rigid concrete pavement section may be used, with less anticipated long-term maintenance. The recommended rigid pavement sections are shown in Table 4.4.

Table 4.4 – Recommended Rigid Pavement Thickness

Pavement Layer	Thickness (inches)		
	Automobile and Light Truck Parking Stalls		Drives, Entrance Approaches, and Dumpster Pads
Non-Reinforced Portland Cement Concrete	5	6	8
Aggregate Base (CA-6)	4	--	4

To provide resistance against salt and freeze-thaw cycles, we recommend the concrete have a minimum 28-day compressive strength of 4,000 pounds per square inch (psi) and air entrainment of 5 to 7 percent by volume. We also recommend that the maximum joint spacing be approximately 15 feet.

4.8.3 Pavement Subgrade Considerations

Pavement subgrades may be subjected to construction traffic and exposure to weather for an extended period and significant problems may be incurred. Soft areas should be selectively undercut and backfilled with properly compacted cohesive soil or otherwise stabilized in a manner approved by SCI prior to placing additional fill. As discussed in Section 4.1, a contingency fund should be established for stabilization of the soil subgrade to minimize disturbance of the silty soils.

Care should be taken to provide drains or drainable transition at locations where pavement sections of varying thickness abut, so as not to trap water within the crushed stone base, which could saturate and soften the subgrade.

5.0 SITE DEVELOPMENT AND CONSTRUCTION CONSIDERATIONS

5.1 Site Preparation

Areas to be cut or to receive fill should be stripped of any surface vegetation or organic topsoil. The strippings should be removed and stockpiled for later placement in landscaped or common ground areas, as appropriate.

Trees and brush may be burned on-site if approved by local ordinances. Burn pits should be located in cut areas such that the ashes are completely removed during site grading. If this is not practical, burn pits must be located outside of building, street, and areas designated as slopes steeper than 5H:1V. Stumps that cannot be burned should be removed from the site.

Silty native soils were encountered throughout the site. As an option, these soils may be cement stabilized to create a stable working platform as further discussed in Section 5.2. Soft areas or otherwise unacceptable materials, if encountered, should be removed and replaced with structural fill or stabilized prior to placing additional fill. If removal of soft soils is impractical due to their excessive depth, they should be stabilized or “bridged over” in a manner approved by SCI. “Bridging” of the soft soils can often be accomplished by placing a geofabric, such as Mirafi HP270, or a geogrid such as Tensar TX140, or equivalents, then placing compacted crushed minus gradation rock. The thickness and gradation of the crushed rock should be evaluated by SCI personnel in the field on a case-by-case basis; however, thicknesses of 1 to 2 feet are common.

Natural slopes to receive fill which are steeper than 5H:1V should be benched prior to the placement of fill. Benching will provide level surfaces for compaction and reduce the potential for development of inclined planes of weakness between the natural soil and compacted fill. The benches should be spaced such that the height of the cut at the up-slope end of the bench is less than 5 feet.

5.2 Fill Materials and Compaction

Prior to fill placement and compaction, the upper 8 inches of the exposed subgrade should be scarified, moisture conditioned, and recompacted. Structural fill should be placed in maximum 8-inch-thick loose lifts and mechanically compacted in accordance with Table 5.1. We recommend that any fill placed in building areas have a liquid limit less than 45 and a plasticity index less than 30. If higher plasticity soils are placed within 3 feet of the floor slab subgrade, or 2 feet of the bottom of the footings, then remediation will be required. Acceptable non-organic fill soils include materials designated CL, ML, GC, GM, and GC-GM by ASTM D 2487-11.

Table 5.1 – Typical Compaction Requirements for Fill

Material Tested	Proctor Type	Minimum Percentage Dry Density
Structural Fill (Cohesive)	Modified (ASTM D 1557)	90
	Standard (ASTM D 698)	95
Structural Fill (Granular)	Modified	95
	Standard	98
Landscaped Areas (Non-Load Bearing)	Modified	88
	Standard	92
Utility Trench Backfill	Modified	90
	Standard	95

Prior to compaction, the soil may require moisture adjustment. During warm weather, moisture reduction can generally be accomplished by disking or otherwise aerating the soil. When air drying is not feasible, a moisture reducing chemical additive, such as hydrated lime, could be incorporated into the soil. As previously stated, silty soils do not react well with hydrated lime. As such, consideration should be given to using cement, cement kiln dust, or fly ash if the soils require stabilization. During dry weather, some addition of moisture may be required to facilitate compaction. This should also be done in a controlled manner using a tank truck with a spray bar. The moistened soil should be thoroughly blended with a disk or pulverizer to produce a uniform moisture content. If construction is performed during the winter season, fill materials should be carefully observed to see that no frozen soil is placed as fill or remains in the base materials upon which fill is placed.

Backfill for foundation walls may consist of lean clay, 1-inch minus crushed limestone, or controlled low strength material. We advise performing field density tests on at least every other lift to monitor compaction. As an alternate, we suggest using 1-inch clean crushed limestone to provide improved drainage and to reduce lateral pressures on the walls. Due to a slight risk of migration of soil fines into the clean rock, a synthetic filter fabric, such as Mirafi 140N or equivalent, should be placed between the soil face of the excavation and the crushed limestone. If clean rock is used, it may be placed in 2-foot-thick lifts and tamped or tracked to achieve adequate densification. Exterior clean rock backfill should be capped with cohesive soil to reduce the potential for surface water infiltration and should also be drained to daylight, or to a sump with a pump.

Backfill placed next to walls should be compacted with hand operated equipment and not large, self-propelled or machine operated equipment, which could result in potential overcompaction and higher lateral pressures. Compaction should be reduced within approximately 1 foot of the walls, and the walls should be observed periodically for signs of movement. If movement is detected, it may be necessary to provide bracing and/or change backfill procedures.

In addition to the minimum density requirements listed above, the soil must be stable, i.e., not “pumping” or rutting excessively under construction traffic, prior to placing additional fill or constructing foundations, floor slabs, or pavements. Field density tests should be performed on each lift of fill to document that proper compaction is achieved.

5.3 Shallow Foundation Excavations

SCI should observe all footing and floor slab excavations for problem areas such as soft or disturbed zones prior to placing concrete. Excessive disturbance of siltier soils in footing excavations should be avoided and could complicate construction. The potential for such disturbance will increase during wetter times of the year. Footing excavations that have been excessively disturbed should be overdeepened to approved undisturbed soils. Overexcavation and replacement with structural fill should be performed where inadequate bearing materials are present in footing excavations.

The base of all excavations should be clean, free of loose soil or uncompacted fill, relatively dry, and maintained near their optimum moisture content. Excavations should be protected from extreme temperatures, precipitation, and construction disturbances. To reduce the possibility of desiccation or saturation of the foundation soils, we recommend that the concrete be placed as soon as possible after excavations are made.

Groundwater is not anticipated to be encountered in the foundation excavations. However, in most situations, small amounts of groundwater seepage into the excavations can be handled by means of gravity ditching and a sump pump. If greater flows are experienced, SCI should be retained to provide additional consultation.

5.4 Floor Slab and Pavement Subgrades

Floor slab and pavement subgrades may be subjected to construction traffic and exposure to weather for an extended period. It may be necessary to proofroll the subgrade, in both cut and fill areas, and recompact the subgrade immediately prior to placing base rock for the floor slab or pavement. In addition, subgrade covered with base rock may be very slow to dry if precipitation occurs after placing the base rock. Therefore, the subgrade should be sloped to provide drainage and placement of the base rock be done as close to the time of pouring the floor slabs or paving as is practical. **Proofroll passes should be limited, particularly on silty subgrades, to reduce the potential for pumping of moisture from deeper within the soil profile.**

Special measures may be required to facilitate construction during wet or cold weather, or where excessive areas of soft/loose soils are identified. These measures may include, but are not limited to, the addition of lime to the subgrade soils for drying purposes, or the removal of soft spongy soils and their replacement with crushed limestone. Soft areas should be selectively undercut and backfilled with properly compacted cohesive soil. A geotechnical stabilization fabric, such as Mirafi HP270, Tensar TX140, or equivalent, may be used to help stabilize particularly soft areas. Where possible, the subgrade should be sloped to provide drainage.

5.5 Excavation Bracing Requirements

In the *Federal Register*, Volume 54, No. 209 (October 1989), the United States Department of Labor, OSHA amended its "Construction Standards for Excavations, 29 CFR, Part 1926, Subpart P." This document was issued to provide for the safety of workers entering excavations, including utility trenches, basements, footings and others. All operations should be performed under the supervision of qualified site personnel in accordance with OSHA regulations.

The contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations, as required to maintain stability of both the excavation sides and bottom. The contractor's "responsible person", as defined in 29 CFR Part 1926, should evaluate the soil exposed in the excavations as part of the contractor's safety procedures. In no case should slope height, sloped inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal safety regulations.

SCI is providing this information solely as a service to our client. SCI does not assume responsibility for construction site safety or the contractor's or other party's compliance with local, state, and federal safety or other regulations.

5.6 Erosion Control and Land Disturbance Monitoring Program

Appropriate erosion and sediment control measures, such as proper contouring during site grading activities, the installation of siltation fences, and/or inlet protection, should be used during construction to keep eroded materials from being carried onto adjacent properties or waterbodies. Depending on the length of time the subgrade is exposed and the amount of siltation that occurs, it may be necessary to periodically remove materials collected by the sediment control systems.

SCI recommends following the procedures detailed in the Stormwater Pollution Prevention Plan (SWPPP). Any site disturbing more than one acre of ground must obtain a Land Disturbance Permit from the Illinois Environmental Protection Agency. As part of the permit compliance procedures, weekly and rain-event site observations must be performed to document the changing site conditions and maintenance of control measures.

6.0 CONSTRUCTION MONITORING PROGRAM

The following list summarizes SCI's recommendations for a construction monitoring program. These services are recommended to provide quality assurance in assessing design assumptions and to document earth-related construction procedures for compliance with plans, specifications, and good engineering practice. SCI should be retained to:

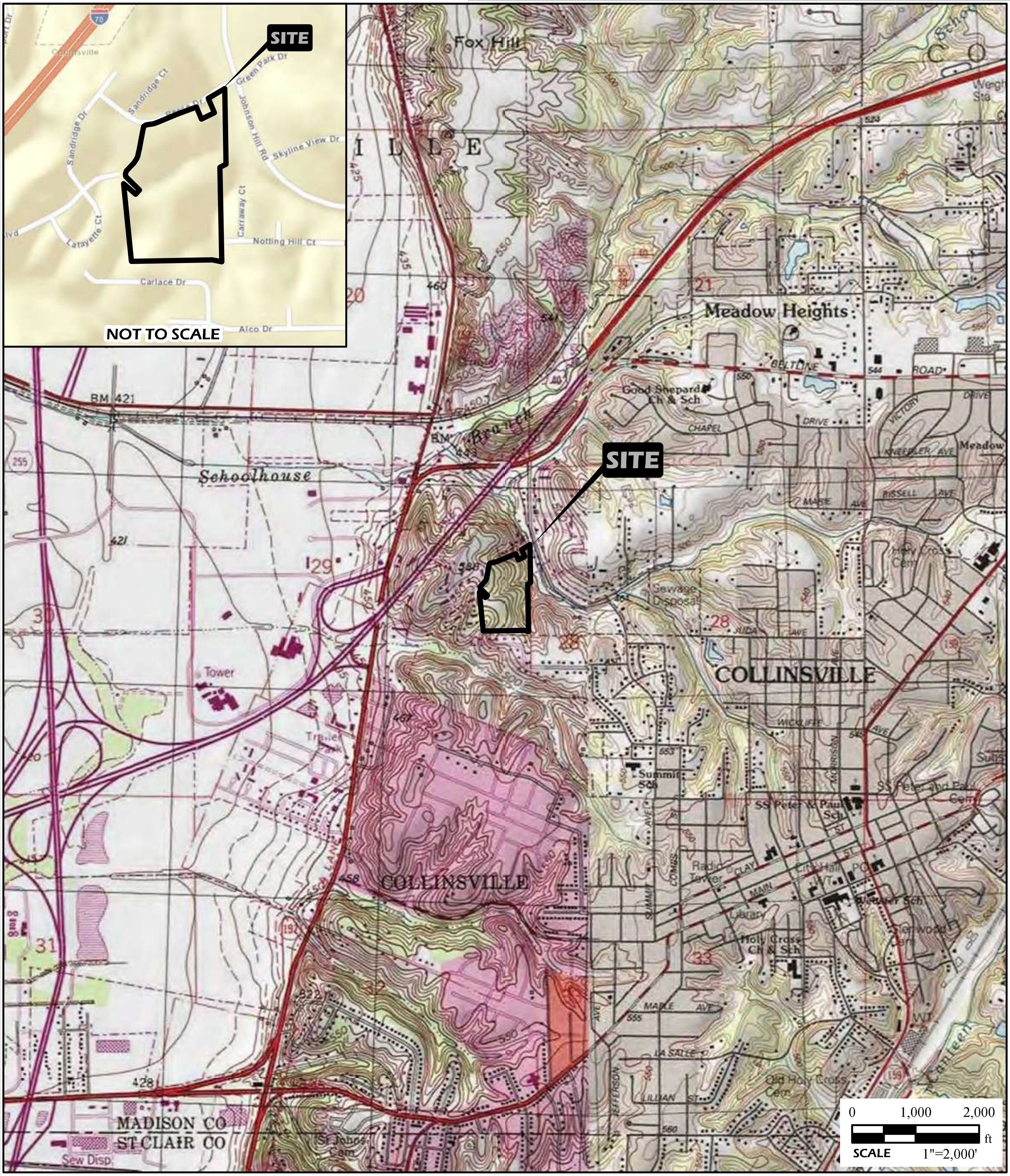
- Participate in a formal preconstruction meeting with the Owner's Representative, Civil Engineer, and Contractor, prior to construction at the site;
- Observe site preparation activities prior to construction, including stripping and proofrolling;
- Conduct and document weekly and rain-event observations at the site, maintain and update on-site paperwork, and provide submittals required by the SWPPP and Land Disturbance Permit;
- Assess the suitability of potential fill materials, including both on-site and off-site sources;
- Monitor placement and compaction of structural fill and backfill;
- Observe the overexcavation of soft, disturbed soils if encountered;
- Observe footing excavations for adequacy of bearing materials;
- Observe foundation excavations and the floor slab subgrades to assess the impact of soft/disturbed soils and to recommend the extent of remedial measures;
- Observe the floor slab subgrades prior to placing base rock;
- Observe backfilling of below-grade utility excavations;
- Observe pavement subgrade preparation and provide observation and testing services for the base course and pavement section;
- Check the thickness of pavement sections and, for asphaltic concrete, its density; and
- Provide quality assurance testing of structural concrete and pavement materials.

7.0 LIMITATIONS

The recommendations provided herein are for the exclusive use of Haven Hill Acres, LLC. It is imperative that SCI be contacted by any third-party interests to evaluate the applicability of this report relative to use by anyone other than Haven Hill Acres, LLC. Our recommendations are specific only to the project described and are not meant to supersede more stringent requirements of local ordinances. They are based on subsurface information obtained at eleven widely spaced sounding and direct push boring locations within the project area performed by SCI; our understanding of the project as presented in Section 2.0, “Site and Project Description”; and geotechnical engineering practice consistent with the standard of care. No other warranty is expressed or implied. SCI should be contacted if conditions encountered are not consistent with those described.

We should also be provided with a set of final development plans, once they are available, to review whether our recommendations have been understood and applied correctly. Failure to provide these documents to SCI may nullify some or all of the recommendations provided herein. In addition, any changes in the planned project or changed site conditions may require revised or additional recommendations on our part.

The final part of our geotechnical service should consist of direct observation during construction, to observe that conditions actually encountered are consistent with those described in this report, and to assess the appropriateness of the analyses and recommendations contained herein. SCI cannot assume responsibility or liability for the adequacy of its recommendations without being retained to observe construction.



PROJECT NAME	GENERAL NOTES/LEGEND
HAVEN HILL ACRES COLLINSVILLE, ILLINOIS	USGS TOPOGRAPHIC MAP MONKS MOUND, ILLINOIS QUADRANGLE DATED 1993 10' CONTOURS COLLINSVILLE, ILLINOIS QUADRANGLE DATED 1991 10' CONTOURS
VICINITY AND TOPOGRAPHIC MAP	STREET MAP HTTP://GOTO.ARCGISONLINE.COM/MAPS/WORLD_STREET_MAP

DRAWN BY	ACV	FIGURE DATE	JOB NUMBER	FIGURE
CHECKED BY	GMK	11/07/2024	2023-1906.10	1

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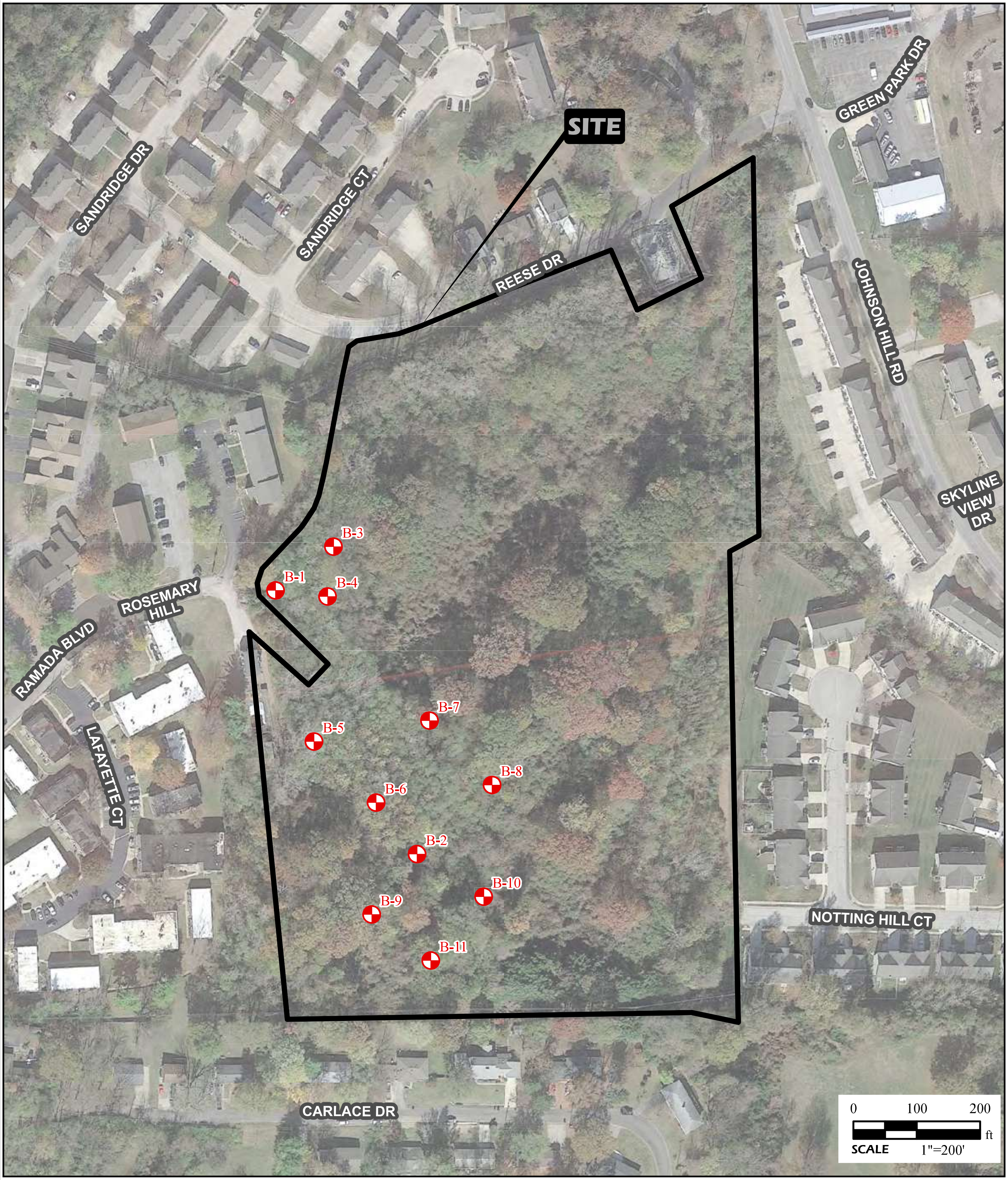


PROJECT NAME	GENERAL NOTES/LEGEND	
HAVEN HILL ACRES COLLINSVILLE, ILLINOIS	APPROXIMATE SOIL BORING LOCATIONS	N S W E
SITE/SURROUNDING PROPERTIES MAP	AERIAL PHOTOGRAPH OBTAINED FROM GOOGLE EARTH 11/2022.	
DRAWN BY ACV / CHECKED BY GMK; FIGURES DATE 11/07/2024; JOB NUMBER 2023-1906.10	DIMENSIONS AND LOCATIONS ARE APPROXIMATE; ACTUAL MAY VARY. DRAWING SHALL NOT BE USED OUTSIDE THE CONTEXT OF THE REPORT FOR WHICH IT WAS GENERATED.	FIGURE 2

Docusign Envelope ID: 1A4113B6-4DFC-4552-858F-E64A9145D632

GENERAL NOTES/LEGEND

APPROXIMATE SOIL BORING LOCATIONS

PLAN PROVIDED 11/05/2024 BY TWM.

DIMENSIONS AND LOCATIONS ARE APPROXIMATE; ACTUAL MAY VARY. DRAWING SHALL NOT BE USED OUTSIDE THE CONTEXT OF THE REPORT FOR WHICH IT WAS GENERATED.

PROJECT NAME

HAVEN HILL ACRES
COLLINSVILLE, ILLINOIS

SITE PLAN

JOB NUMBER
2023-1906.10

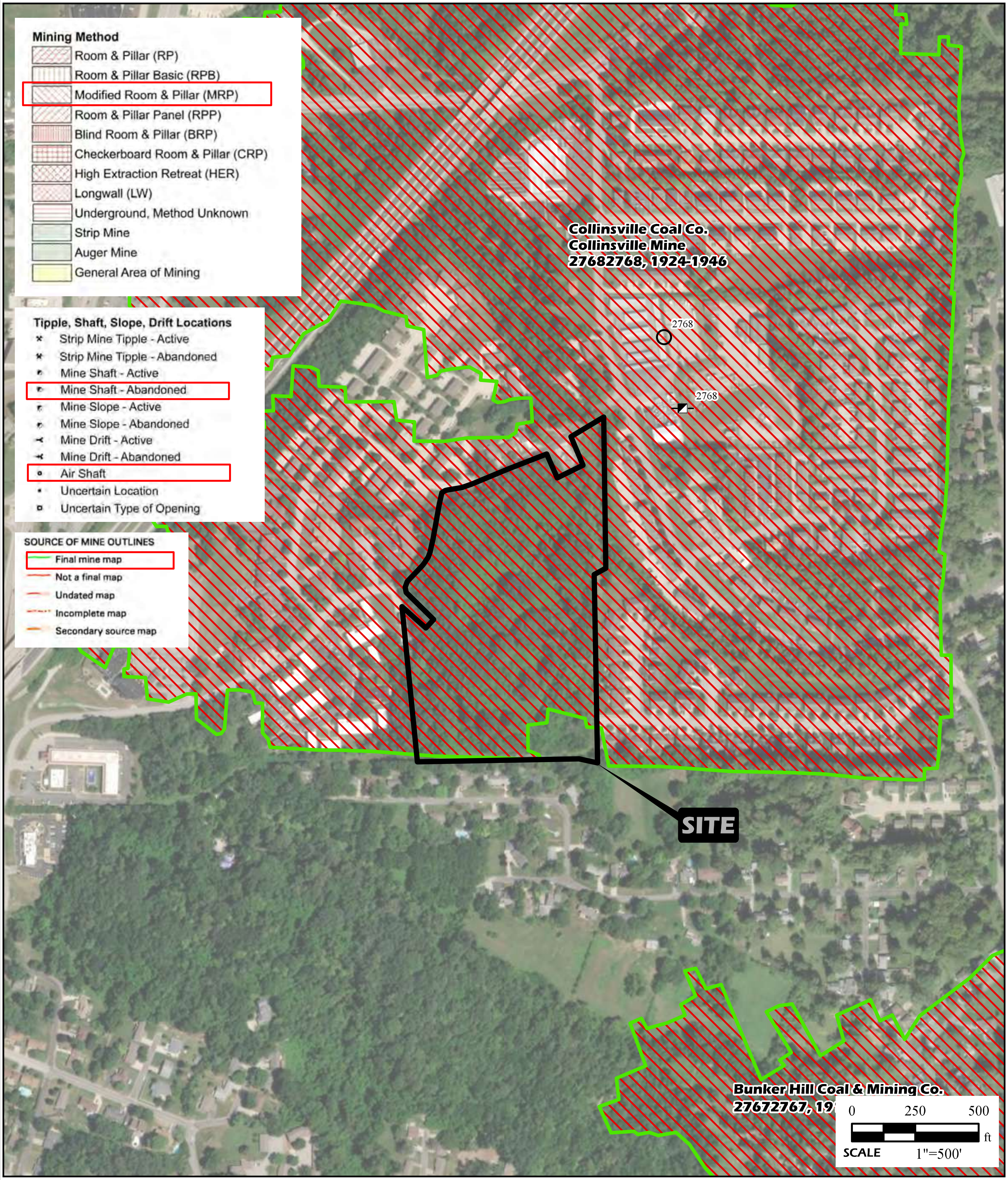
FIGURE DATE
11/19/2024

DRAWN BY
ACV

CHECKED BY
GMK

FIGURE
3

Docusign Envelope ID: 1A4113B6-4DFC-4552-858F-E64A9145D632



PROJECT NAME

HAVEN HILL ACRES
COLLINSVILLE, ILLINOIS

ISGS MINE MAP

DRAWN BY	ACV	FIGURE DATE	JOB NUMBER
CHECKED BY	GMK	11/07/2024	2023-1906.10

GENERAL NOTES/LEGEND

AERIAL PHOTOGRAPH OBTAINED FROM ARCGIS ONLINE, WORLD IMAGERY.

MINE LOCATION INFORMATION OBTAINED FROM:
HTTP://WWW.ISGS.UIUC.EDU
ILLINOIS STATE GEOLOGICAL SURVEY DISCLAIMS THAT LOCATIONS OF SOME FEATURES ON THE MINE MAP MAY BE OFFSET BY 500 OR MORE FEET DUE TO ERRORS IN THE ORIGINAL SOURCE MAPS, THE COMPILATION PROCESS, DIGITIZING, OR A COMBINATION OF THESE FACTORS. DIMENSIONS AND LOCATIONS ARE APPROXIMATE; ACTUAL MAY VARY. DRAWING SHALL NOT BE USED OUTSIDE THE CONTEXT OF THE REPORT FOR WHICH IT WAS GENERATED.

FIGURE
4

Appendix A

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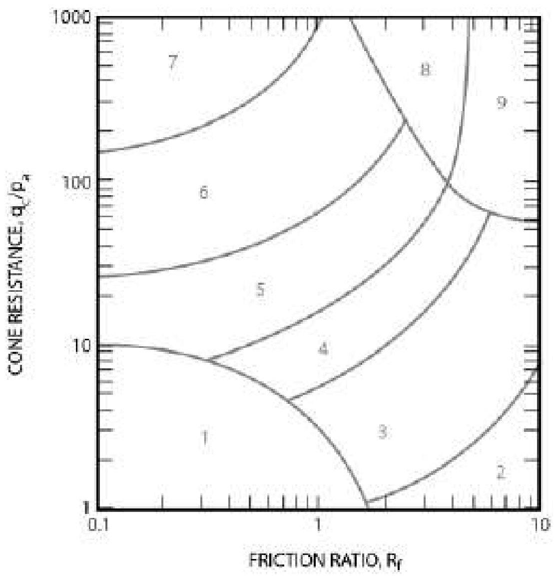
USE and UNDERSTANDING of CPTu Logs

The CPTu logs show the corrected Tip Resistance (q_t), Friction (f_s), Porewater Pressure (U_2), SPT N_{60} correlation (N_{60}), and the Soil Behavior interpretation results. The corrected cone tip resistance (q_t) is measured as the maximum force over the projected area of the cone tip. It is a point stress related to the bearing capacity of the soil. The measured uncorrected tip value (q_c) must be corrected for porewater pressure effects (Lunne et al, 1997), especially in clays and silts where porewater pressures typically vary greatly from hydrostatic. The sleeve friction (fs) is used as a measure of soil type and can be expressed by friction ratio (R_F) which is used in the soil behavior classification. The u_2 position element is required for the measurement of penetration porewater pressures and the correction of tip resistance. Calculations of q_t, R_F, and the SPT N_{60} calculation are discussed below.

The estimated stratigraphic profiles included in the CPTu logs are based on relationships between q_t, f_s, and U2 as shown graphically in the figure below.

Non-normalized CPT Soil Behavior Type (SBT) chart



Z	*Soil Behavior Type*
1	*Sensitive, fine grained*
2	*Organic soils - clay*
3	*Clay – silty clay to clay*
4	*Silt mixtures – clayey silt to silty clay*
5	*Sand mixtures – silty sand to sandy silt*
6	*Sands – clean sand to silty sand*
7	*Gravelly sand to dense sand*
8	*Very stiff sand to clayey sand**
9	*Very stiff fine grained**

** Heavily overconsolidated or cemented*

P_a = atmospheric pressure = 100 kPa = 1 tsf

Derived Values from CPT

Corrected cone resistance: $q_t = q_c + u_2(1\text{-}a)$
Friction ratio: $R_f = (f_s/q_t) \times 100\%$

Equivalent SPT N60, (blows/ft) Lunne et al. (1997)

$$\frac{(q_t/p_a)}{N_{60}} = 8.5 \text{ X } \left(1 - \frac{I_c}{4.6}\right)$$

Where $I_c = ((3.47 - \log Q_{t1})2 + (\log R_F + 1.22)2)0.5$
And $Qt_n = ((q_t - sv_o)/p_a) \text{ x } (p_a/(s'v_o)n$, and recalculate I_c, then iterate for n:

$$n = 0.381 \times I_c + 0.05 \times \left(\frac{s_{vo}'}{p_a}\right) - 0.15$$

Iterate until the change in n, $\Delta_n < 0.01$

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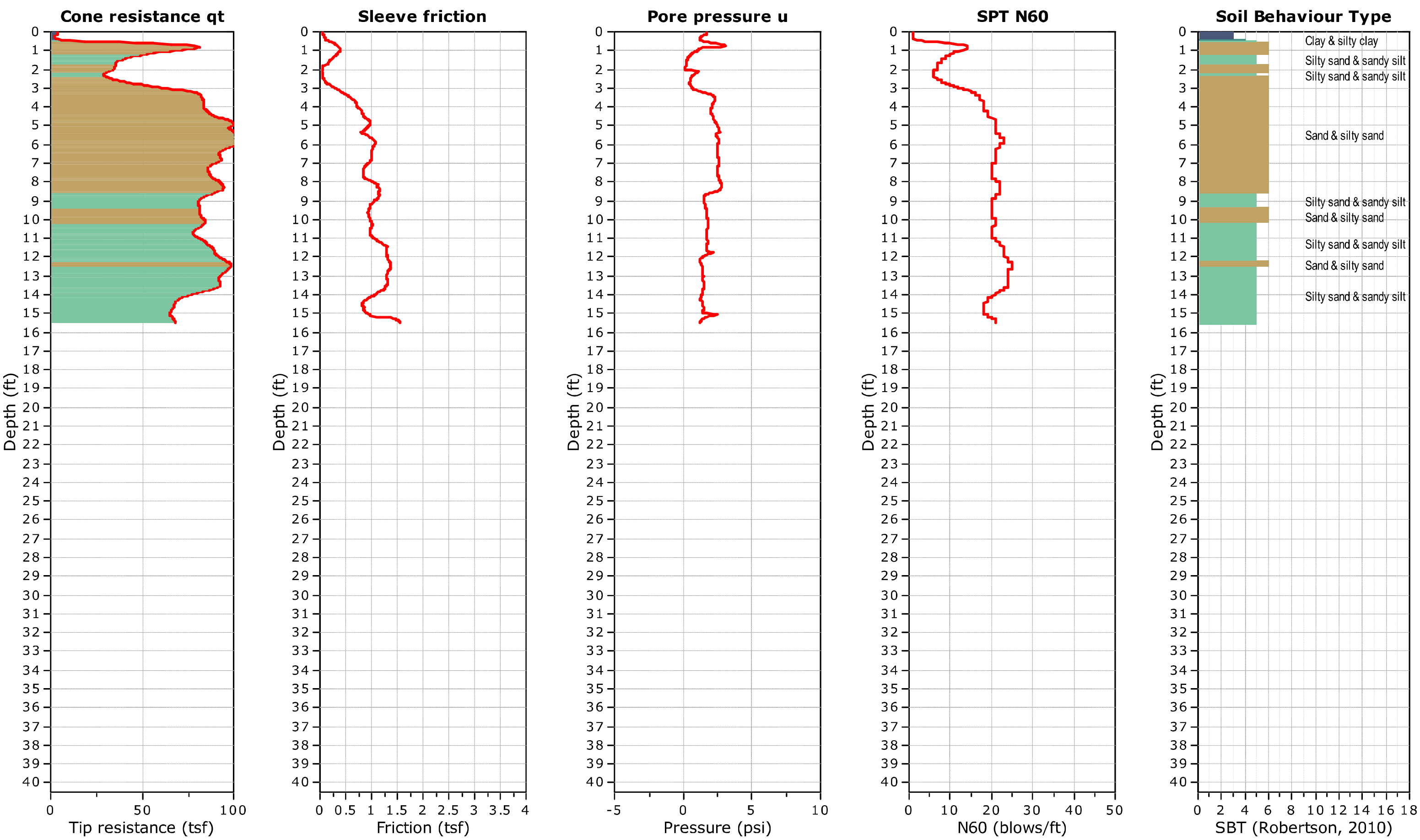
CPT: B-01

Project: **Haven Hill Acres: 2023-1906.10**
Location: **Collinsville, Illinois**

Total depth: 15.52 ft, Date: 11/1/2024
Surface Elevation: 551.00 ft



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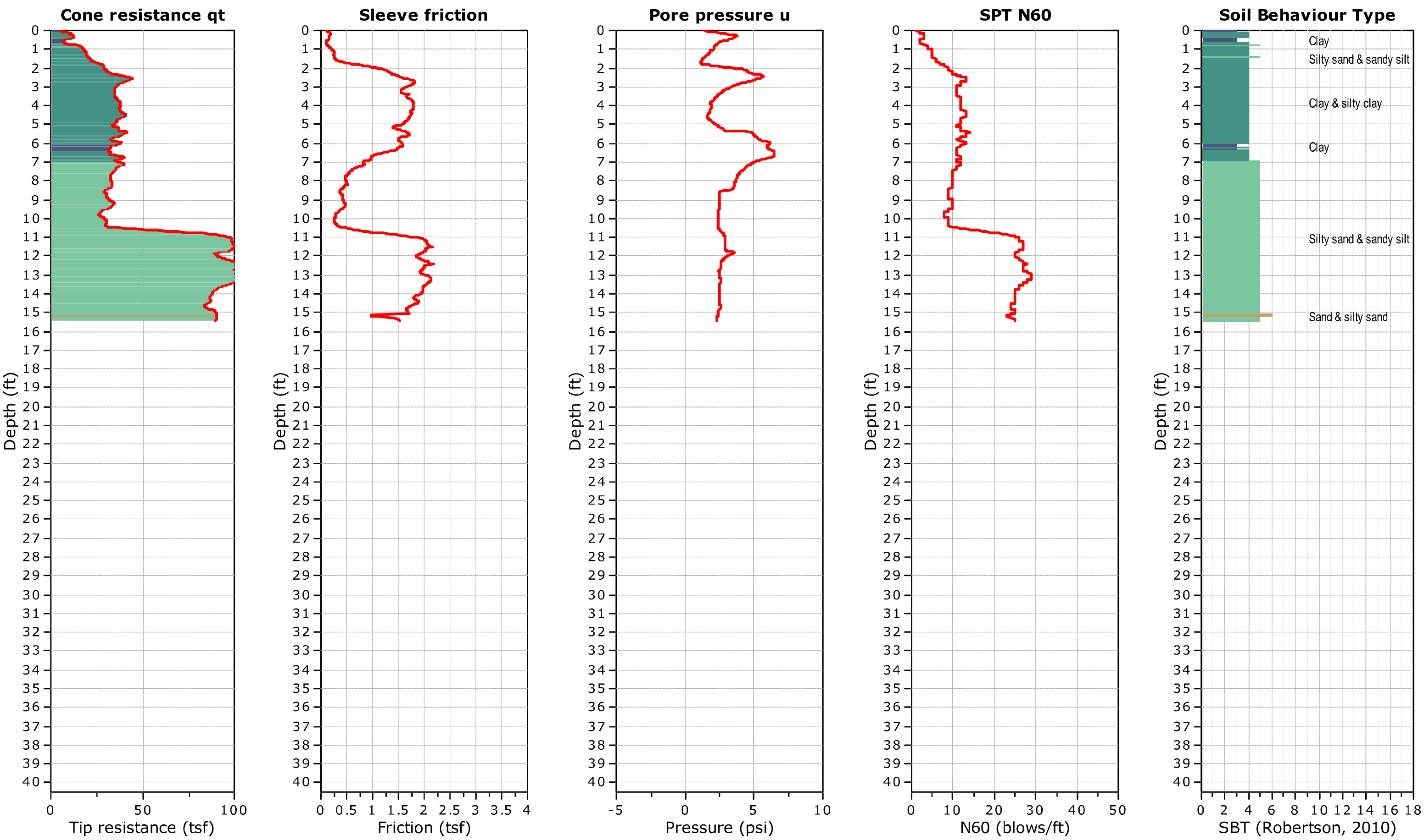
CPT: B-02

Total depth: 15.42 ft, Date: 11/1/2024
Surface Elevation: 546.00 ft

Project: **Haven Hill Acres: 2023-1906.10**
Location: **Collinsville, Illinois**



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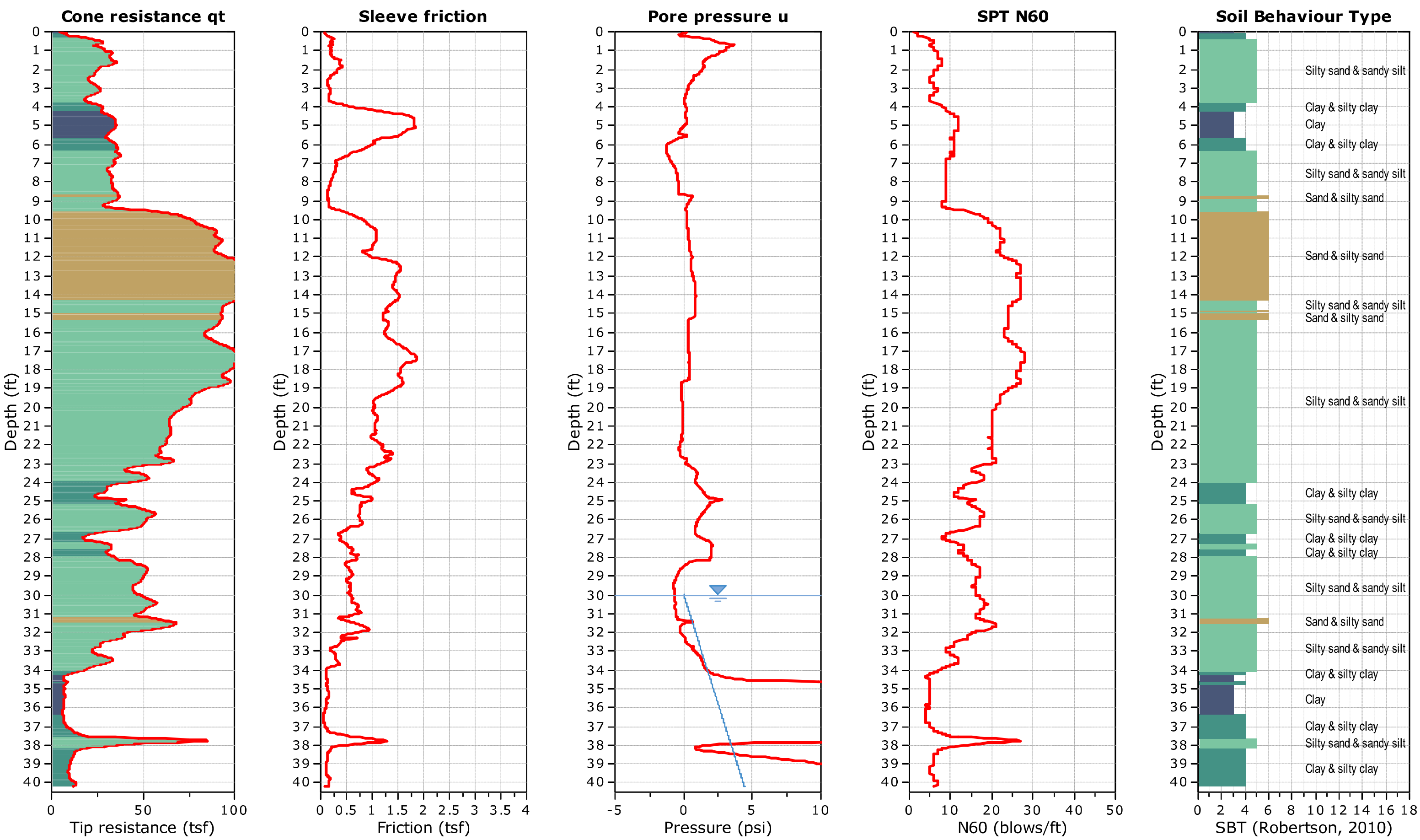
CPT: B-03
Total depth: 40.22 ft, Date: 11/1/2024
Surface Elevation: 554.00 ft

Project: **Haven Hill Acres: 2023-1906.10**
Location: **Collinsville, Illinois**



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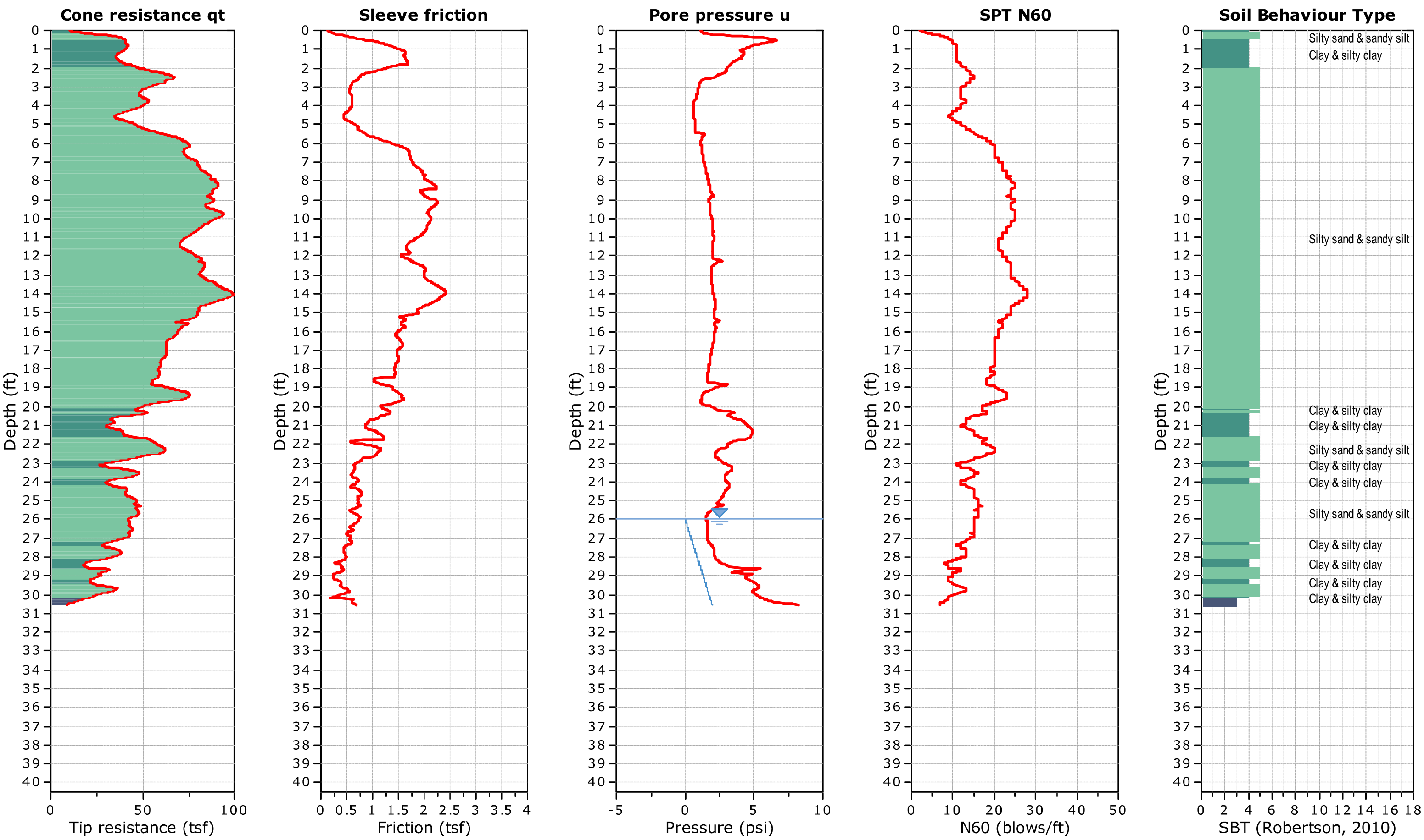
CPT: B-04

Project: **Haven Hill Acres: 2023-1906.10**
Location: **Collinsville, Illinois**

Total depth: 30.55 ft, Date: 11/1/2024
Surface Elevation: 550.00 ft



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Geotechnical Engineering
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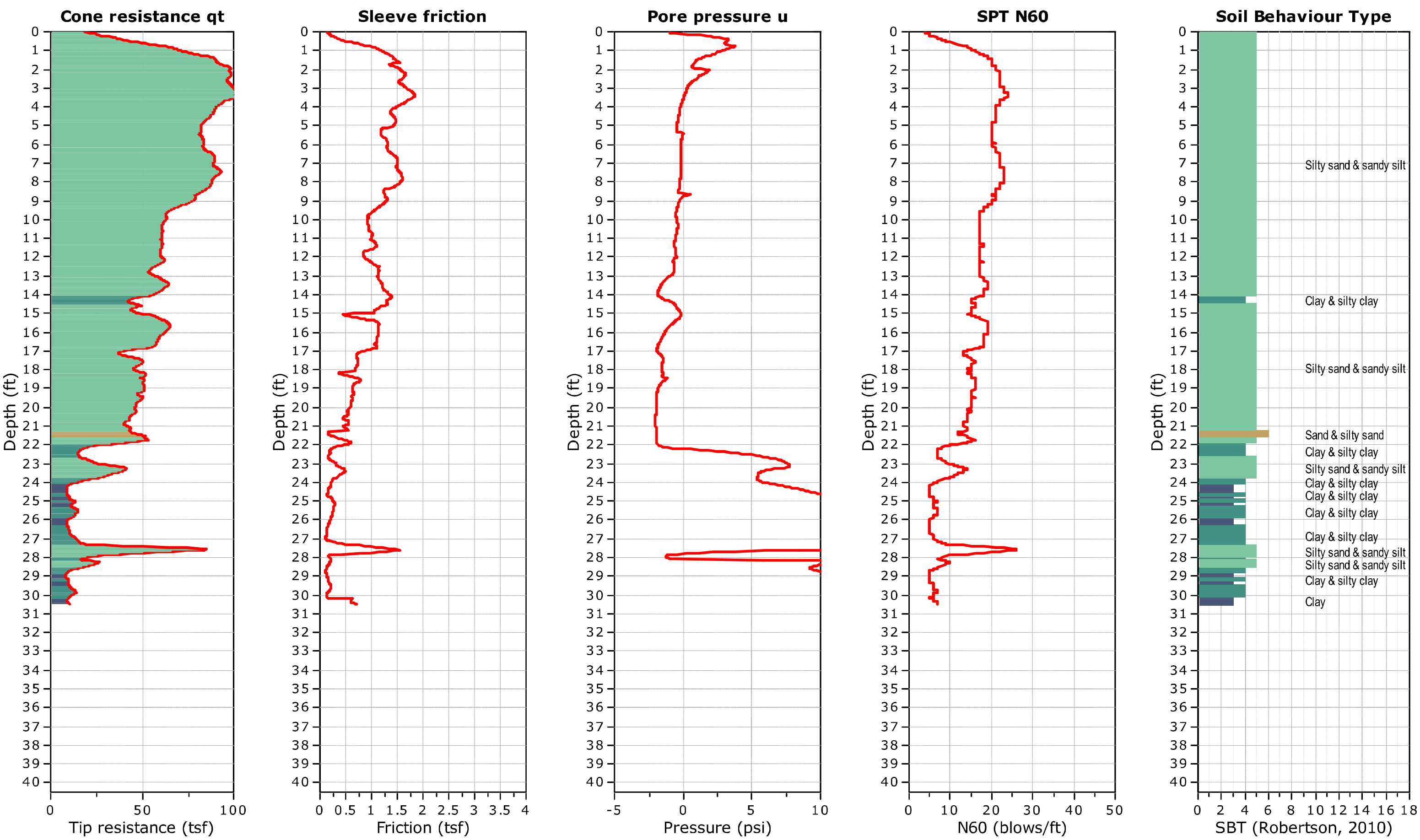
CPT: B-05

Project: **Haven Hill Acres: 2023-1906.10**
Location: **Collinsville, Illinois**

Total depth: 30.51 ft, Date: 11/1/2024
Surface Elevation: 543.00 ft

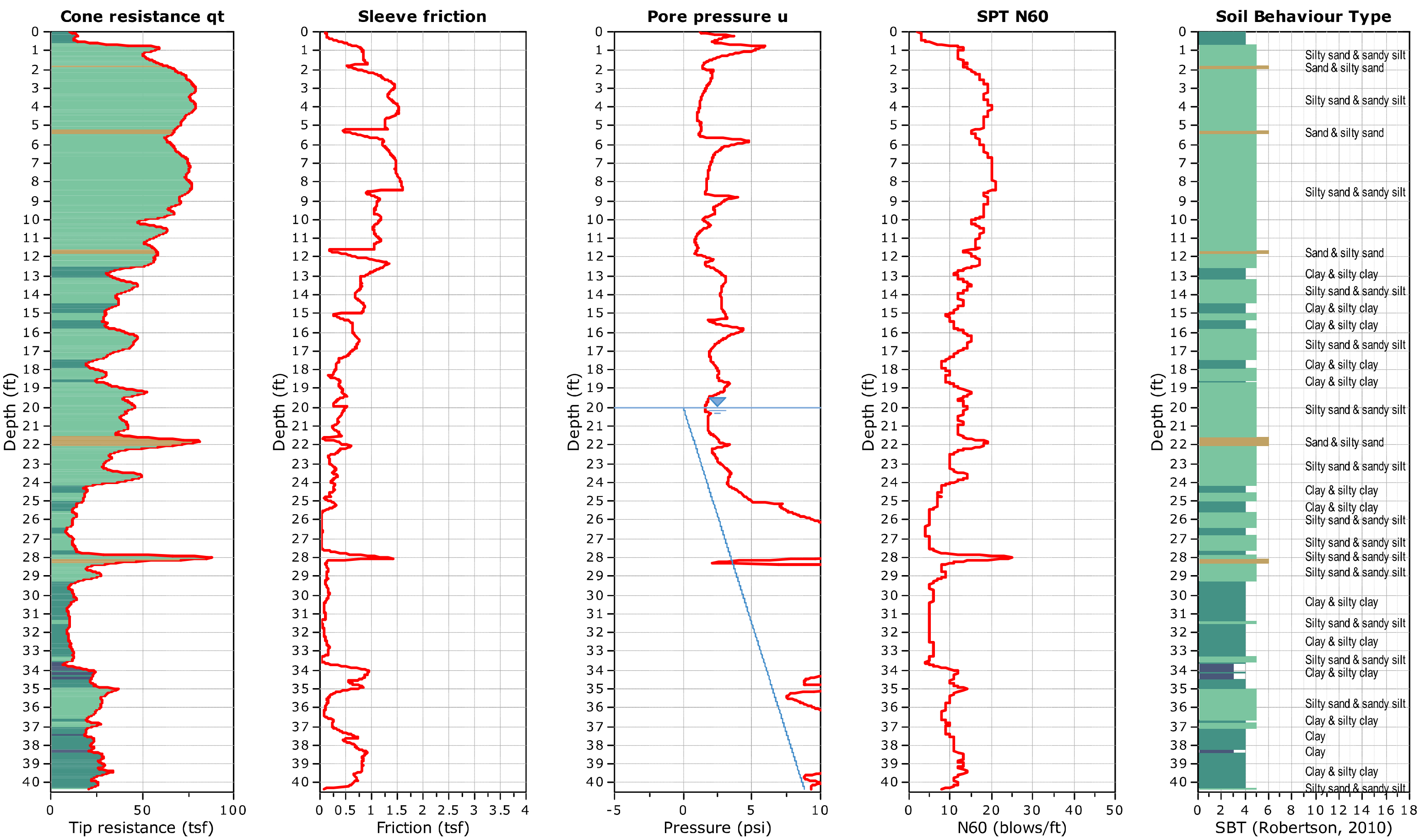


SCI Engineering Inc.
650 Pierce Blvd, O'Fallon, IL 62269
Geotechnical Engineering
http://www.sciengineering.com

CPT: B-06

Project: **Haven Hill Acres: 2023-1906.10**
Location: **Collinsville, Illinois**

Total depth: 40.32 ft, Date: 11/1/2024
Surface Elevation: 541.00 ft



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Geotechnical Engineering
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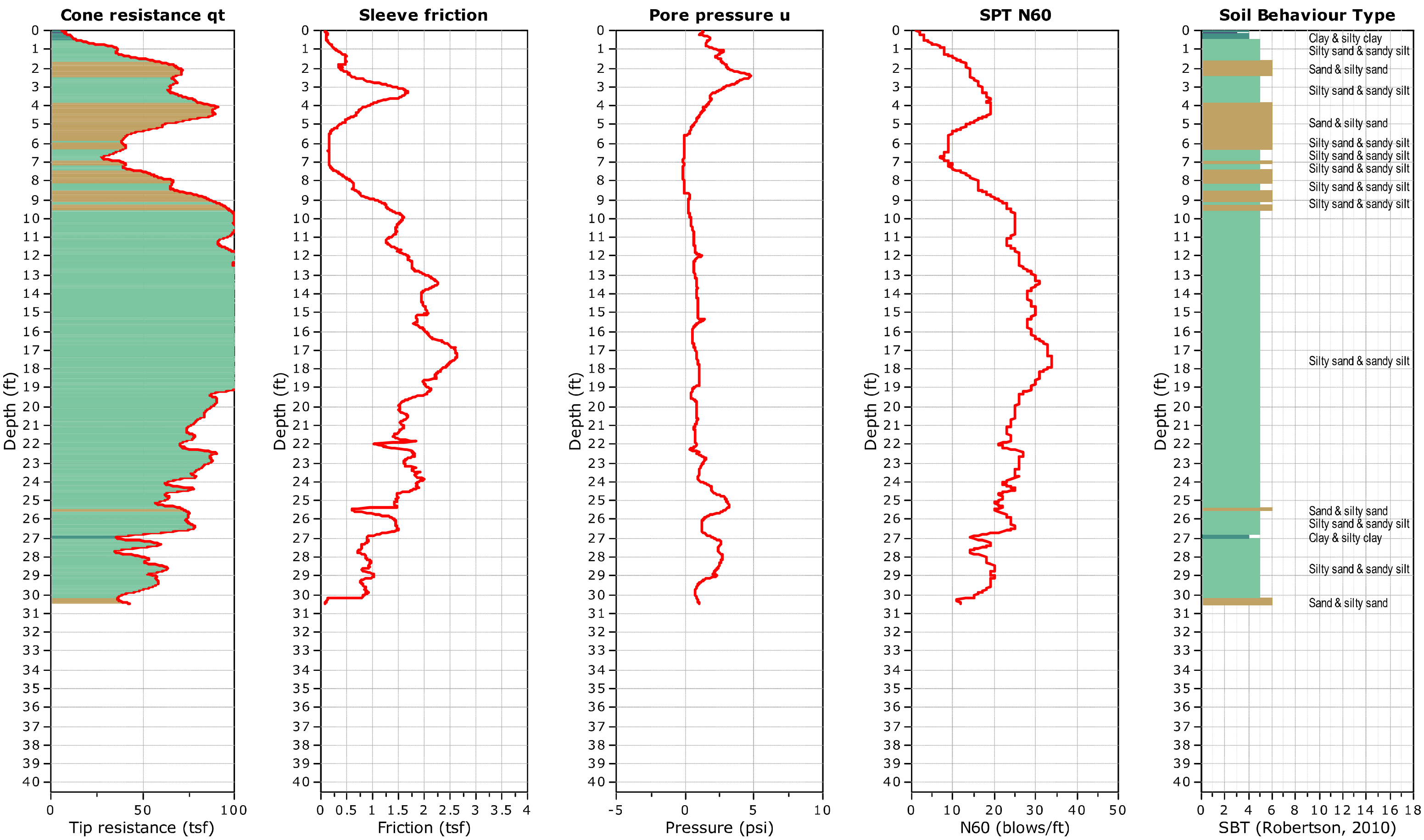
CPT: B-07

Project: **Haven Hill Acres: 2023-1906.10**
Location: **Collinsville, Illinois**

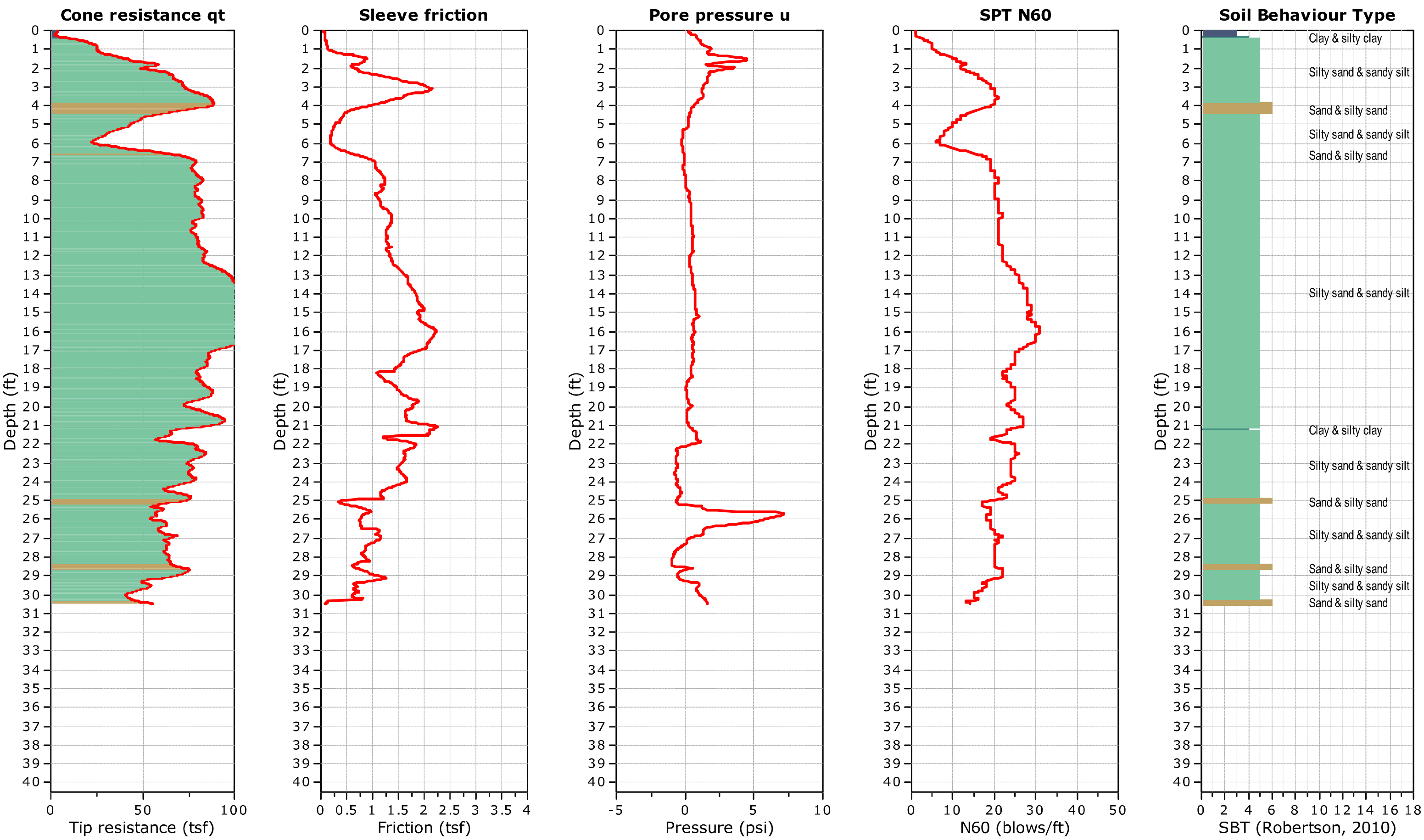
Total depth: 30.51 ft, Date: 11/1/2024
Surface Elevation: 548.00 ft



SCI Engineering Inc.
650 Pierce Blvd, O'Fallon, IL 62269
Geotechnical Engineering
http://www.sciengineering.com

CPT: B-08
Total depth: 30.51 ft, Date: 11/1/2024
Surface Elevation: 547.00 ft

Project: Haven Hill Acres: 2023-1906.10
Location: Collinsville, Illinois



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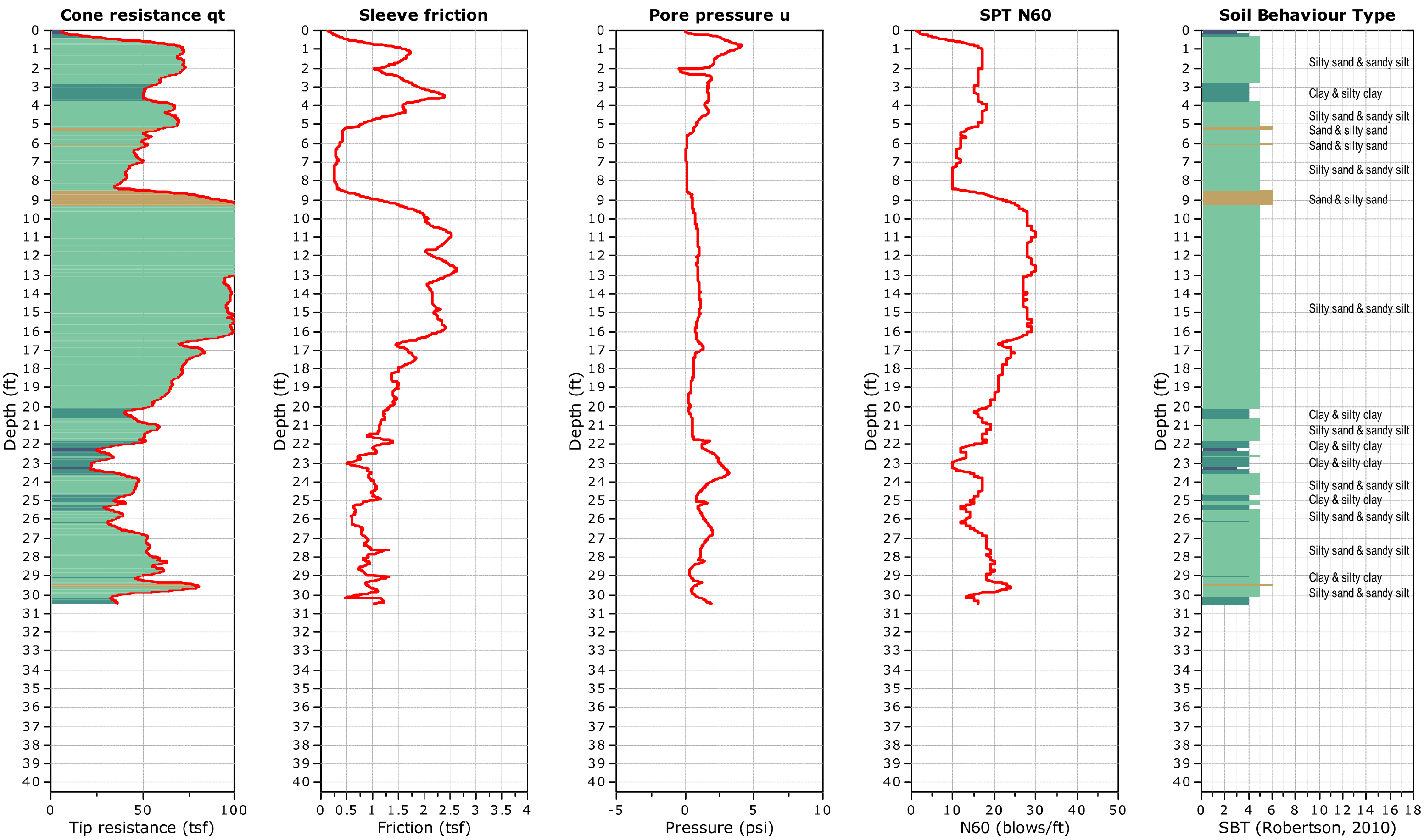
Geotechnical Engineering
http://www.sciengineering.com

CPT: B-09

Project: **Haven Hill Acres: 2023-1906.10**
Location: **Collinsville, Illinois**

Total depth: 30.51 ft, Date: 11/1/2024
Surface Elevation: 547.00 ft



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Geotechnical Engineering
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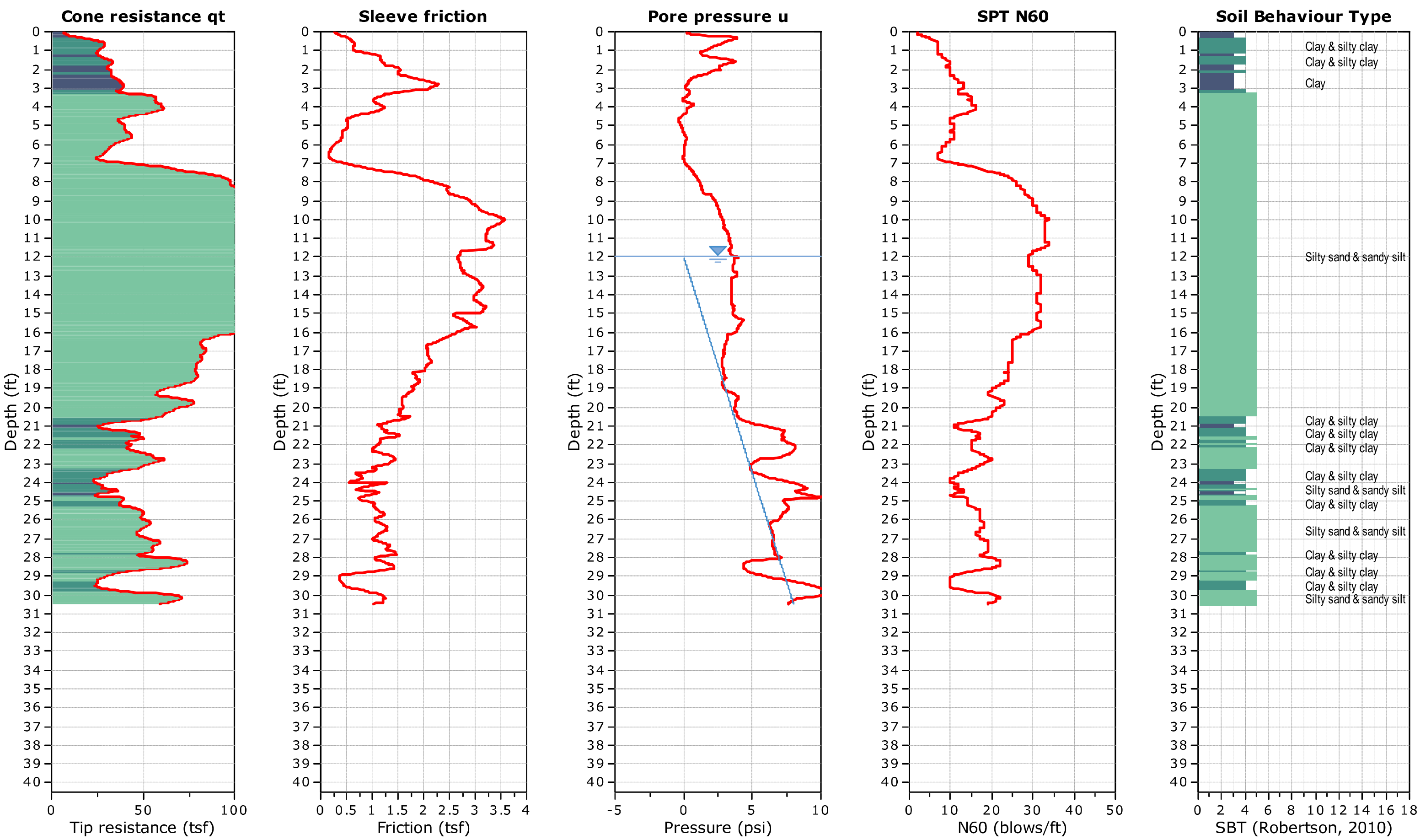
CPT: B-10

Project: **Haven Hill Acres: 2023-1906.10**
Location: **Collinsville, Illinois**

Total depth: 30.48 ft, Date: 11/1/2024
Surface Elevation: 546.00 ft



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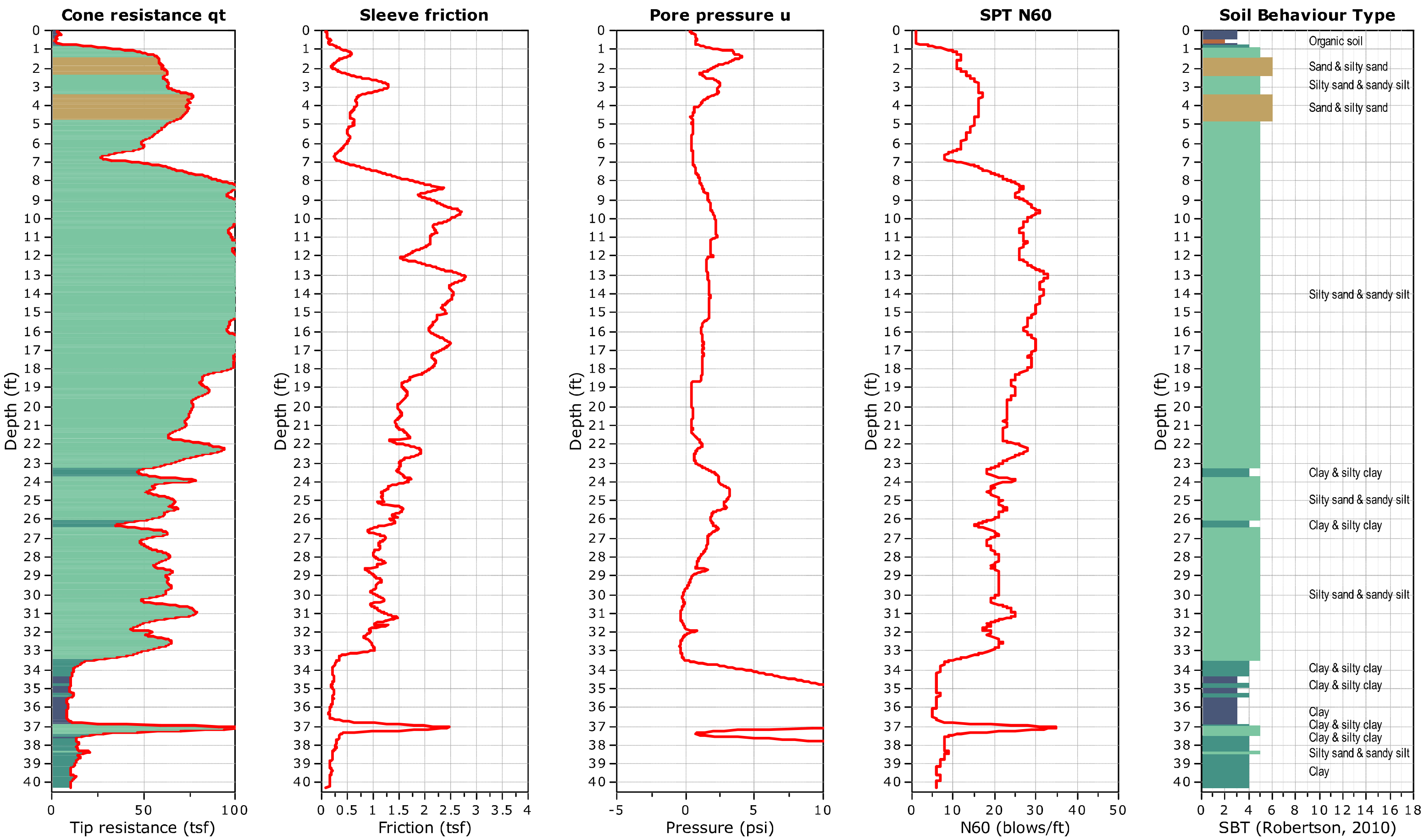
Geotechnical Engineering
http://www.sciengineering.com

CPT: B-11

Project: **Haven Hill Acres: 2023-1906.10**
Total depth: 40.26 ft, Date: 11/1/2024

Location: **Collinsville, Illinois**
Surface Elevation: 550.00 ft



Appendix B

SCI ENGINEERING, INC.
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BORING LOG LEGEND AND NOMENCLATURE

Depth is in feet below ground surface. **Elevation** is in feet mean sea level, site datum, or as otherwise noted.

Sample Type

- **SS** Split-spoon sample, disturbed, obtained by driving a 2-inch-O.D. split-spoon sampler (ASTM D 1586).
- **NX** Diamond core bit, nominal 2-inch-diameter rock sample (ASTM D 2113).
- **ST** Thin-walled (Shelby) tube sample, relatively undisturbed, obtained by pushing a 3-inch-diameter, tube (ASTM D 1587).
- **CS** Continuous sample tube system, relatively undisturbed, obtained by split-barrel sampler in conjunction with auger advancement.
- **SV** Shear vane, field test to determine strength of cohesive soil by pushing or driving a 2-inch-diameter vane, and then shearing by torquing soil in existing and remolded states (ASTM D 2573).
- **BS** Bag sample, disturbed, obtained from cuttings.

Recovery is expressed as a ratio of the length recovered to the total length pushed, driven, cored.

Blows Numbers indicate blows per 6 inches of split-spoon sampler penetration when driven with a 140-pound hammer falling freely 30 inches. The number of total blows obtained for the second and third 6-inch increments is the N value (Standard Penetration Test or SPT) in blows per foot (ASTM D 1586). Practical refusal is considered to be 50 or more blows without achieving 6 inches of penetration, and is expressed as a ratio of 50 to actual penetration, e.g., 50/2 (50 blows for 2 inches).

For analysis, the N value is used when obtained by a cathead and rope system. When obtained by an automatic hammer, the N value may be increased by a factor of 1.3.

Vane Shear Strength is expressed as the peak strength (existing state) / the residual strength (remolded state).

Description indicates soil constituents and other classification characteristics (ASTM D 2488) and the Unified Soil Classification (ASTM D 2487). Secondary soil constituents (expressed as a percentage) are described as follows:

Trace	<5
Few	5-15
With	>15-30

Stratigraphic Breaks may be observed or interpreted, and are indicated by a dashed line. Transition between described materials may be gradual.

Laboratory Test Results

- Natural moisture content (ASTM D 2216) in percent.
- Dry density in pounds per cubic foot (pcf).
- Hand penetrometer value of apparently intact cohesive sample in kips per square foot (ksf).
- Unconfined compressive strength (ASTM D 2166) in kips per square foot (ksf).
- Liquid and Plastic Limits (ASTM D 4318) in percent.

RQD (Rock Quality Designation) is the ratio between the total length of core segments 4 inches or more in length and the total length of core drilled. RQD (expressed as a percentage) indicates insitu rock quality as follows:

Excellent	90 to 100
Good	75 to 90
Fair	50 to 75
Poor	25 to 50
Very Poor	0 to 25

BORING LOG

PROJECT Haven Hill Acres
LOCATION Collinsville, Illinois
DRILLER SCI Engineering, Inc. HAMMER Automatic
EQUIPMENT Vertek S4 CPT w/Discrete Sampler ELEVATION 551±

BORING NUMBER B-1
SHEET 1 of 1
PROJECT NO. 2023-1906.10
DATE DRILLED 10/31/2024

DEPTH (ft)	SAMPLE NUMBER	SAMPLE TYPE	RECOVERY (in/in)	BLOWS (per 6 in)	DESCRIPTION (UNIFIED SOIL CLASSIFICATION)	GRAPHIC	SEE REMARK NO.	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	HAND PENETROMETER (ksf)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	LIQUID LIMIT	PLASTICITY INDEX	ELEVATION (ft)
					6" TOPSOIL									
					SILT (ML): Brown			9		5.0				
	1	DP	42/48											549
3								7		--				
								8		4.5				546
6	2	DP	47/48											
								8		--				
					Boring terminated at 8 feet.									543
9														
														540
12														
														537
15														
														534
18														

WATER LEVEL:

X NONE OBSERVED WHILE DRILLING
_____ ft WHILE DRILLING
_____ ft ___ HRS AFTER DRILLING
_____ ft ___ DAYS AFTER DRILLING

REMARKS:

BORING LOG

PROJECT Haven Hill Acres
BORING NUMBER B-2
LOCATION Collinsville, Illinois
SHEET 1 of 1
DRILLER SCI Engineering, Inc.
HAMMER Automatic
PROJECT NO. 2023-1906.10
EQUIPMENT Vertek S4 CPT w/Discrete Sampler
ELEVATION 546±
DATE DRILLED 10/31/2024

DEPTH (ft)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE RECOVERY (in/in)	SAMPLE BLOWS (per 6 in)	DESCRIPTION (UNIFIED SOIL CLASSIFICATION)	GRAPHIC	SEE REMARK NO.	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	HAND PENETROMETER (ksf)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	LIQUID LIMIT	PLASTICITY INDEX	ELEVATION (ft)
					SILT (ML): Brown									
								19		2.5		30	6	
	1	DP	48/48											
3								21		5.5				543
								20		3.5				
6	2	DP	48/48											540
								22		1.5				
					Boring terminated at 8 feet.									
9														537
12														534
15														531
18														528

WATER LEVEL:

X NONE OBSERVED WHILE DRILLING
___ ft WHILE DRILLING
___ ft ___ HRS AFTER DRILLING
___ ft ___ DAYS AFTER DRILLING

REMARKS:

BORING LOG

PROJECT Haven Hill Acres		**BORING NUMBER**	B-3
LOCATION Collinsville, Illinois		**SHEET** 1 **of** 1	
DRILLER SCI Engineering, Inc.	**HAMMER** Automatic	**PROJECT NO.**	2023-1906.10
EQUIPMENT Vertek S4 CPT w/Discrete Sampler	**ELEVATION** 554±	**DATE DRILLED**	10/31/2024

DEPTH (ft)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE RECOVERY (in/in)	SAMPLE BLOWS (per 6 in)	DESCRIPTION (UNIFIED SOIL CLASSIFICATION)	GRAPHIC	SEE REMARK NO.	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	HAND PENETROMETER (ksf)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	LIQUID LIMIT	PLASTICITY INDEX	ELEVATION (ft)
					SILT (ML): Brown									
								10		1.5				
	1	DP	48/48		LEAN CLAY (CL): Brown									552
3								19		5.0				
								21		7.0		49	27	549
6	2	DP	48/48		SILT (ML): Brown									
								12		3.0				
					Boring terminated at 8 feet.									546
9														
														543
12														
														540
15														
														537
18														

WATER LEVEL:

X NONE OBSERVED WHILE DRILLING
____ ft WHILE DRILLING
____ ft ____ HRS AFTER DRILLING
____ ft ____ DAYS AFTER DRILLING

REMARKS:

BORING LOG

PROJECT Haven Hill Acres
BORING NUMBER B-4
LOCATION Collinsville, Illinois
SHEET 1 **of** 1
DRILLER SCI Engineering, Inc.
HAMMER Automatic
PROJECT NO. 2023-1906.10
EQUIPMENT Vertek S4 CPT w/Discrete Sampler
ELEVATION 550±
DATE DRILLED 10/31/2024

DEPTH (ft)	SAMPLE NUMBER	SAMPLE TYPE	RECOVERY (in/in)	BLOWS (per 6 in)	DESCRIPTION (UNIFIED SOIL CLASSIFICATION)	GRAPHIC	SEE REMARK NO.	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	HAND PENETROMETER (ksf)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	LIQUID LIMIT	PLASTICITY INDEX	ELEVATION (ft)
					SILT (ML): Brown									
								18		7.5				549
	1	DP	42/48		LEAN CLAY (CL): Brown									
3								13		6.5		38	17	
					SILT (ML): Brown									546
								7		0.5				
6	2	DP	47/48											
								10		1.0				543
					Boring terminated at 8 feet.									
9														
														540
12														
														537
15														
														534
18														
														531

WATER LEVEL:
X NONE OBSERVED WHILE DRILLING
____ ft WHILE DRILLING
____ ft ____ HRS AFTER DRILLING
____ ft ____ DAYS AFTER DRILLING

REMARKS:

BORING LOG

PROJECT Haven Hill Acres
LOCATION Collinsville, Illinois
DRILLER SCI Engineering, Inc. HAMMER Automatic
EQUIPMENT Vertek S4 CPT w/Discrete Sampler ELEVATION 543±

BORING NUMBER B-5
SHEET 1 of 1
PROJECT NO. 2023-1906.10
DATE DRILLED 10/31/2024

DEPTH (ft)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE RECOVERY (in/in)	SAMPLE BLOWS (per 6 in)	DESCRIPTION (UNIFIED SOIL CLASSIFICATION)	GRAPHIC	SEE REMARK NO.	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	HAND PENETROMETER (ksf)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	LIQUID LIMIT	PLASTICITY INDEX	ELEVATION (ft)
					SILT (ML): Brown									
								13		--				
	1	DP	44/48											
3								14		--				540
								15		--				
6	2	DP	45/48											537
								14		--				
					Boring terminated at 8 feet.									
9														534
12														531
15														528
18														525

WATER LEVEL:
X NONE OBSERVED WHILE DRILLING
____ ft WHILE DRILLING
____ ft ____ HRS AFTER DRILLING
____ ft ____ DAYS AFTER DRILLING

REMARKS:

BORING LOG

PROJECT Haven Hill Acres **BORING NUMBER** B-6
LOCATION Collinsville, Illinois **SHEET** 1 **of** 1
DRILLER SCI Engineering, Inc. **HAMMER** Automatic **PROJECT NO.** 2023-1906.10
EQUIPMENT Vertek S4 CPT w/Discrete Sampler **ELEVATION** 541± **DATE DRILLED** 10/31/2024

DEPTH (ft)	SAMPLE NUMBER	SAMPLE TYPE	RECOVERY (in/in)	BLOWS (per 6 in)	DESCRIPTION (UNIFIED SOIL CLASSIFICATION)	GRAPHIC	SEE REMARK NO.	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	HAND PENETROMETER (ksf)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	LIQUID LIMIT	PLASTICITY INDEX	ELEVATION (ft)
					SILT (ML): Brown									
								17		--				540
	1	DP	42/48											
3								12		--				
														537
								14		--		25	1	
6	2	DP	48/48											
								16		--				534
					Boring terminated at 8 feet.									
9														
														531
12														
														528
15														
														525
18														
														522

WATER LEVEL:
X NONE OBSERVED WHILE DRILLING
___ ft WHILE DRILLING
___ ft ___ HRS AFTER DRILLING
___ ft ___ DAYS AFTER DRILLING

REMARKS:

BORING LOG

PROJECT Haven Hill Acres | **BORING NUMBER** B-7
LOCATION Collinsville, Illinois | **SHEET** 1 **of** 1
DRILLER SCI Engineering, Inc. | **HAMMER** Automatic | **PROJECT NO.** 2023-1906.10
EQUIPMENT Vertek S4 CPT w/Discrete Sampler | **ELEVATION** 548± | **DATE DRILLED** 10/31/2024

DEPTH (ft)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE RECOVERY (in/in)	SAMPLE BLOWS (per 6 in)	DESCRIPTION (UNIFIED SOIL CLASSIFICATION)	GRAPHIC	SEE REMARK NO.	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	HAND PENETROMETER (ksf)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	LIQUID LIMIT	PLASTICITY INDEX	ELEVATION (ft)
					SILT (ML): Brown									
								9		2.5				
	1	DP	48/48											546
3							1	6		--				
								12		5.5				543
6	2	DP	48/48											
								16		8.5				
					Boring terminated at 8 feet.									540
9														
														537
12														
														534
15														
														531
18														

WATER LEVEL:
X NONE OBSERVED WHILE DRILLING
___ ft WHILE DRILLING
___ ft ___ HRS AFTER DRILLING
___ ft ___ DAYS AFTER DRILLING

REMARKS:
1) Atterberg limit testing performed: Non-plastic

BORING LOG

PROJECT Haven Hill Acres
LOCATION Collinsville, Illinois
DRILLER SCI Engineering, Inc. **HAMMER** Automatic
EQUIPMENT Vertek S4 CPT w/Discrete Sampler **ELEVATION** 547±

BORING NUMBER B-8
SHEET 1 **of** 1
PROJECT NO. 2023-1906.10
DATE DRILLED 10/31/2024

DEPTH (ft)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE RECOVERY (in/in)	SAMPLE BLOWS (per 6 in)	DESCRIPTION (UNIFIED SOIL CLASSIFICATION)	GRAPHIC	SEE REMARK NO.	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	HAND PENETROMETER (ksf)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	LIQUID LIMIT	PLASTICITY INDEX	ELEVATION (ft)
					SILT (ML): Brown									
								14		6.0				546
	1	DP	48/48											
3								13		3.0				
														543
								9		0.5				
6	2	DP	48/48											
								8		1.5				540
					Boring terminated at 8 feet.									
9														
														537
12														
														534
15														
														531
18														
														528

WATER LEVEL:
X NONE OBSERVED WHILE DRILLING
____ ft WHILE DRILLING
____ ft ____ HRS AFTER DRILLING
____ ft ____ DAYS AFTER DRILLING

REMARKS:

BORING LOG

PROJECT Haven Hill Acres
BORING NUMBER B-9
LOCATION Collinsville, Illinois
SHEET 1 **of** 1
DRILLER SCI Engineering, Inc.
HAMMER Automatic
PROJECT NO. 2023-1906.10
EQUIPMENT Vertek S4 CPT w/Discrete Sampler
ELEVATION 547±
DATE DRILLED 10/31/2024

DEPTH (ft)	SAMPLE NUMBER	SAMPLE TYPE	RECOVERY (in/in)	BLOWS (per 6 in)	DESCRIPTION (UNIFIED SOIL CLASSIFICATION)	GRAPHIC	SEE REMARK NO.	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	HAND PENETROMETER (ksf)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	LIQUID LIMIT	PLASTICITY INDEX	ELEVATION (ft)
					SILT (ML): Brown									
								17		5.0				546
	1	DP	42/48											
3								19		5.0				
														543
								13		3.0				
6	2	DP	42/48											
								7		--		27	3	540
					Boring terminated at 8 feet.									
9														
														537
12														
														534
15														
														531
18														
														528

WATER LEVEL:

X NONE OBSERVED WHILE DRILLING
_____ ft WHILE DRILLING
_____ ft ___ HRS AFTER DRILLING
_____ ft ___ DAYS AFTER DRILLING

REMARKS:

BORING LOG

PROJECT Haven Hill Acres
BORING NUMBER B-10
LOCATION Collinsville, Illinois
SHEET 1 **of** 1
DRILLER SCI Engineering, Inc. **HAMMER** Automatic
PROJECT NO. 2023-1906.10
EQUIPMENT Vertek S4 CPT w/Discrete Sampler **ELEVATION** 546±
DATE DRILLED 10/31/2024

DEPTH (ft)	SAMPLE NUMBER	SAMPLE TYPE	RECOVERY (in/in)	BLOWS (per 6 in)	DESCRIPTION (UNIFIED SOIL CLASSIFICATION)	GRAPHIC	SEE REMARK NO.	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	HAND PENETROMETER (ksf)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	LIQUID LIMIT	PLASTICITY INDEX	ELEVATION (ft)
					SILT (ML): Brown									
								12		4.5				
	1	DP	44/48											
3								17		5.0				543
								9		--		30	7	
6	2	DP	44/48											540
								8		--				
					Boring terminated at 8 feet.									
9														537
12														534
15														531
18														528

WATER LEVEL:

X NONE OBSERVED WHILE DRILLING
____ ft WHILE DRILLING
____ ft ____ HRS AFTER DRILLING
____ ft ____ DAYS AFTER DRILLING

REMARKS:

BORING LOG

PROJECT Haven Hill Acres **BORING NUMBER** B-11
LOCATION Collinsville, Illinois **SHEET** 1 **of** 1
DRILLER SCI Engineering, Inc. **HAMMER** Automatic **PROJECT NO.** 2023-1906.10
EQUIPMENT Vertek S4 CPT w/Discrete Sampler **ELEVATION** 550± **DATE DRILLED** 10/31/2024

DEPTH (ft)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE RECOVERY (in/in)	SAMPLE BLOWS (per 6 in)	DESCRIPTION (UNIFIED SOIL CLASSIFICATION)	GRAPHIC	SEE REMARK NO.	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	HAND PENETROMETER (ksf)	UNCONFINED COMPRESSIVE STRENGTH (ksf)	LIQUID LIMIT	PLASTICITY INDEX	ELEVATION (ft)
					SILT (ML): Brown									
								10		6.5				549
	1	DP	44/48											
3								15		6.5				
														546
								9		--				
6	2	DP	44/48											
								4		--				543
					Boring terminated at 8 feet.									
9														
														540
12														
														537
15														
														534
18														
														531

WATER LEVEL:
X NONE OBSERVED WHILE DRILLING
____ ft WHILE DRILLING
____ ft ___ HRS AFTER DRILLING
____ ft ___ DAYS AFTER DRILLING

REMARKS:

Important Information about This

Geotechnical-Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, you can benefit from a lowered exposure to problems associated with subsurface conditions at project sites and development of them that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed herein, contact your GBA-member geotechnical engineer. Active engagement in GBA exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Understand the Geotechnical-Engineering Services Provided for this Report

Geotechnical-engineering services typically include the planning, collection, interpretation, and analysis of exploratory data from widely spaced borings and/or test pits. Field data are combined with results from laboratory tests of soil and rock samples obtained from field exploration (if applicable), observations made during site reconnaissance, and historical information to form one or more models of the expected subsurface conditions beneath the site. Local geology and alterations of the site surface and subsurface by previous and proposed construction are also important considerations. Geotechnical engineers apply their engineering training, experience, and judgment to adapt the requirements of the prospective project to the subsurface model(s). Estimates are made of the subsurface conditions that will likely be exposed during construction as well as the expected performance of foundations and other structures being planned and/or affected by construction activities.

The culmination of these geotechnical-engineering services is typically a geotechnical-engineering report providing the data obtained, a discussion of the subsurface model(s), the engineering and geologic engineering assessments and analyses made, and the recommendations developed to satisfy the given requirements of the project. These reports may be titled investigations, explorations, studies, assessments, or evaluations. Regardless of the title used, the geotechnical-engineering report is an engineering interpretation of the subsurface conditions within the context of the project and does not represent a close examination, systematic inquiry, or thorough investigation of all site and subsurface conditions.

Geotechnical-Engineering Services are Performed for Specific Purposes, Persons, and Projects, and At Specific Times

Geotechnical engineers structure their services to meet the specific needs, goals, and risk management preferences of their clients. A geotechnical-engineering study conducted for a given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client.

Likewise, geotechnical-engineering services are performed for a specific project and purpose. For example, it is unlikely that a geotechnical-engineering study for a refrigerated warehouse will be the same as one prepared for a parking garage; and a few borings drilled during a preliminary study to evaluate site feasibility will not be adequate to develop geotechnical design recommendations for the project.

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project or purpose;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, the reliability of a geotechnical-engineering report can be affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying the recommendations in it. A minor amount of additional testing or analysis after the passage of time – if any is required at all – could prevent major problems.

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read the report in its entirety. Do not rely on an executive summary. Do not read selective elements only. *Read and refer to the report in full.*

You Need to Inform Your Geotechnical Engineer About Change

Your geotechnical engineer considered unique, project-specific factors when developing the scope of study behind this report and developing the confirmation-dependent recommendations the report conveys. Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- the elevation, configuration, location, orientation, function or weight of the proposed structure and the desired performance criteria;
- the composition of the design team; or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project or site changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept* *responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface using various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing is performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgement to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team through project completion to obtain informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, they are not final, because the geotechnical engineer who developed them relied heavily on judgement and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* exposed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a continuing member of the design team, to:

- confer with other design-team members;
- help develop specifications;
- review pertinent elements of other design professionals' plans and specifications; and
- be available whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction-phase observations.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for information purposes only.* To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. This happens in part because soil and rock on project sites are typically heterogeneous and not manufactured materials with well-defined engineering properties like steel and concrete. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually provide environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures.* If you have not obtained your own environmental information about the project site, ask your geotechnical consultant for a recommendation on how to find environmental risk-management guidance.

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, the engineer's services were not designed, conducted, or intended to prevent migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration. Confront the risk of moisture infiltration*** by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

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