GEOTECHNICAL EVALUATION REPORT

UPLAND WWTP IMPROVEMENTS
UPLAND, INDIANA

SME Project Number: 101022.00
November 17, 2025

SME

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Fort Wayne, IN 46825

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November 17, 2025

Mr. Ryan Marovich, PE (IN, KY, IL, OH, MI, WI)
Project Engineer
Commonwealth Engineers, Inc.
19 North Court Street
Crown Point, Indiana 46307

Via E-Mail: rmarovich@contactcei.com

RE: Geotechnical Evaluation Report
Upland WWTP Improvements
500 East Bragg Avenue
Upland, Indiana
SME Project No. 101022.00

Dear Mr. Marovich,

We have completed our geotechnical evaluation for the referenced project. This report presents the results of our observations and analyses, our geotechnical and pavement subgrade recommendations, and a discussion of potential construction considerations based on the information disclosed by the borings.

We appreciate this opportunity to be of service. If you have questions or require additional information, please contact me.

Sincerely,

SME

Jamie M. Bates

Jamie M. Bates, PE
Project Manager

Enclosure: SME Geotechnical Evaluation Report; Dated November 17, 2025

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1. INTRODUCTION

This report presents the results of the geotechnical evaluation performed by SME for the proposed Upland WWTP Improvements Project. This evaluation was conducted in general accordance with the scope of services outlined in SME Proposal No. P03010.25, dated July 25, 2025. Our services for this evaluation were authorized by Commonwealth Engineers, Inc. (CEI) under “Exhibit K” the Agreement between Engineer and Consultant for Professional Services, Amendment No. 2025-38.

SME received the following information which was used in the evaluation and preparation of this report:

- A preliminary site plan and floor plan titled “Upland Wastewater Treatment Utility Storage Facility”, dated September 26, 2025, and prepared by Halstead Architects.
- An e-mail sent by Greg Lange on October 15, 2025, providing anticipated structural loading conditions for the project provided by Mr. Michael W. Halstead with Halstead Architects.
- A preliminary grading plan titled “Town of Upland, Grant County, Indiana – Wastewater Treatment Plant Utility Storage Building”, dated November 3, 2025, and prepared by CEI.
- An aerial image titled “Upland WWTP – 500 East Bragg Avenue”, provided by CEI on July 22, 2025, and included the site location, anticipated limits of the proposed project area, approximate locations of six suggested borings.
- A historic geotechnical evaluation completed by SME for the Town of Upland Wastewater Improvements project at the Upland WWTP (SME Project No. 087061.00) dated July 29, 2021.

SME completed six borings (B1 through B6) at the project site on October 24, 2025. Refer to the boring logs included in Appendix A for the specific depth of each individual boring. The approximate boring locations are depicted on Image No. 1 and on the Boring Location Diagram located in Appendix A (Figure No. 1). Soil descriptions and the field and laboratory test results are presented on the boring logs. Exploration and laboratory testing procedures are presented in Section 4.

IMAGE NO. 1: Excerpt from Figure No. 1 in Appendix A – Boring Location Diagram (north facing upward)

1.1 SITE CONDITIONS

The project site is at the existing Upland Wastewater Treatment Plant (WWTP) located at 500 East Bragg Avenue in Upland, Indiana. The Upland WWTP facility consists of existing structures associated with the wastewater treatment process surrounded by grass-covered areas, gravel covered access drives. The site is enclosed by a chain-linked fence. The subject project area is located outside the chain-linked fence and is on the southeast side of the facility. The project area consists of both grass-covered and wooded/brush covered areas. Based on the referenced grading plan, the site generally slopes downward from the north to south and east to west and the existing ground surface elevations across the site range from about 909 to 919 feet.

1.2 PROJECT DESCRIPTION

The project will include constructing a new storage facility building, a new salt storage barn, and corresponding pavement drives and parking areas. The storage facility building will include a single-story, slab-on-grade structure, with a plan area of about 10,000 square feet. The salt storage barn will include a single-story, slab-on-grade structure, with a plan area of about 1,250 square feet. The storage facility building will be constructed within the existing wooded/brushed covered area, and the salt storage barn will be constructed within a grass-covered area. The approximate locations of the proposed buildings are depicted on Image No. 1 and Figure No. 1. Based on the information provided to us by Halstead Architects (project structural engineer), the design structural loads will include maximum column loads of 20 kips and maximum wall loads of 1 kip per linear foot. Anticipated slab loading was not provided at the time this report was written. We anticipate floor slab loading will include maximum of 250 pounds per square foot (psf). No anticipated structural loading conditions were provided for the proposed salt storage barn.

We understand a design finish floor elevation (FFE) of 915 feet has been established for the proposed storage facility and salt storage barn. Based on the referenced grading plan, about 1 to 3 feet of fill will be required to establish the design subgrade level for the storage facility building and about 1 to 3 feet of fill or cut will be required to establish the design subgrade level for the salt storage barn.

New pavements will be constructed west of the new storage facility building. Based on the referenced grading plan, up to 3 feet of fill to 5 feet of cut will be required to establish the design subgrade level across the proposed pavement areas.

The recommendations of this report are based on the information provided above and the results of the field and laboratory evaluation. Contact SME if the final design information is different than discussed herein.

1.3 SUBSURFACE CONDITIONS

Refer to the boring logs for the soil conditions at the specific boring locations. In summary, the soil conditions observed in the borings generally consisted of surficial topsoil overlying natural clay which extended to the explored depths of the borings. An exception was observed at boring B1 where a layer of clay fill was encountered below the topsoil and extended to a depth of 2 feet below the existing grade, corresponding to elevation 914 feet. The clay fill and natural clay was encountered in a medium to hard condition and had moisture contents ranging from 10 to 23 percent.

Groundwater was not observed in the borings during or upon completion of drilling.

Refer to the Field Testing Procedures in Appendix B for additional information about groundwater level measurements. If more information regarding groundwater levels at this site is required, then we recommend performing additional subsurface assessment(s).

2. ANALYSIS AND RECOMMENDATIONS

2.1 SITE PREPARATION AND EARTHWORK

2.1.1 EXISTING FILL CONSIDERATIONS

Clay fill was encountered in boring B1 extending to approximate elevation 914 feet and was readily identifiable from the root fibers within the material. We consider the fill to be undocumented or uncontrolled since the origin of the existing fill is not known, we are not aware of records documenting the fill placement and any compaction operations, and because of the variable composition and density of the fill. We anticipate most of the existing fill within the proposed salt storage building footprint will be removed during earthwork operations based on the estimated FFE of 915 feet. Based on the borings, the fill consistency and moisture content is mostly uniform with the natural clays encountered on site.

Based on the depth of the fill observed in boring B1, we anticipate the foundations for the proposed salt storage barn will extend below the fill depth. The existing clay fill can be considered for support of lightly-loaded floor slabs and pavements, provided the subgrade soils are evaluated by SME at the time of construction, and prepared as described below, prior to floor slab and pavement construction. Subgrade improvements including compaction in-place and/or removal of unsuitable materials and replacement with engineered fill could be necessary to achieve suitable subgrade conditions.

There are inherent risks of greater than typical settlement and poor structural performance associated with constructing structures over undocumented fills. We believe the associated risks at this site could include visible cracking and differential movements of floor slabs on-grade. The risks of constructing pavements over the existing fills include differential settlements that could lead to cracking, areas of ponded surface water (i.e., birdbaths), and the need for greater than typical maintenance. These risks can be significantly reduced by removing the fill from beneath floor slabs and pavements and replacing it with engineered fill. The incidence and severity of cracking and movements associated with constructing the proposed floor slabs and pavements over the existing fill at this site can be reduced, but not eliminated, by thoroughly evaluating the condition of the fill during construction, remediating unsuitable materials that are identified, and improving the subgrade as recommended in the following report sections.

Evaluation of the existing fill during construction must be conducted by SME and may include probing the existing fill areas with a hand auger, testing several feet below the subgrade surface using a cone penetrometer, observing the condition of the fill in shallow test pits and in foundation excavations, and observing the response of the surface of the fill when subjected to a proofroll. Further evaluate suspect materials observed during the evaluation and testing processes. The contractor needs to be prepared to assist SME by excavating test pits, as needed. Unsuitable fills must be remediated as recommended in the report sections below.

The following report sections are based on the assumption that the Owner is willing to accept the increased risks described above, and that the existing fills will remain in-place and be used to support floor slabs and pavements. If the Owner does not accept the stated risks, then undercut the existing fill from beneath the building footprint and pavements and replace it with engineered fill per the requirements outlined in Section 2.1.5 of this report.

2.1.2 GENERAL SITE SUBGRADE PREPARATION

Clear the proposed building area of vegetation, topsoil, and other deleterious materials to expose suitable underlying inorganic subgrade soils. Completely remove tree root mats.

Extend the clearing and stripping a minimum of 5 feet beyond the limits of the proposed buildings, pavement areas, and areas to receive engineered fill. Based on the boring information, we expect both clay fill and natural clay will be exposed after clearing and stripping the project site.

Reroute existing utilities that are within the building footprints to outside of the building areas. Remove abandoned utilities and backfill the resulting excavations with granular engineered fill. If it is necessary for existing utilities to be abandoned in-place, the utilities must not conflict with the proposed construction, they must be fully grouted, and the suitability of the existing backfill must be verified for structural support. Do not abandon utilities in-place within the zone of influence of proposed foundations.

The contractor needs to remove ponded surface water and prevent run-off from reaching foundation excavations and areas of prepared subgrade. The contractor needs to also establish positive surface drainage at the onset of construction to mitigate the potential for subgrade disturbance. To reduce the potential of subgrade disturbance across the site, restrict construction traffic to dedicated areas of the site that can be controlled and stabilized as needed.

After site stripping/clearing for the proposed slab-on-grade areas the subgrade is expected to consist mostly of existing clay fill or natural clays. The on-site subgrade clay soils are sensitive to disturbances during construction and the overall success of the subgrade preparation during mass earthwork operations will directly affect the suitability of pavement/slab bearing soils. As such, take care during site earthwork operations to prepare the subgrade for support of the building slab-on-grades and new pavements.

The subgrade may become softened/loosened and disturbed depending on the season and the amount of precipitation received immediately prior to the subgrade preparation activities, and the degree of construction equipment trafficking over the subgrade. For areas of disturbed subgrade, it will be necessary to improve the disturbed soils or remove and replace them with engineered fill. Improvement of the exposed subgrade may consist of aerating and drying disturbed soils, recompacting them in place, and placement of crushed aggregate, possibly with a geotextile for separation.

After stripping the existing topsoil, and after cutting to design grades, the exposed subgrade needs to be uniformly compacted using large construction equipment. Take care during compaction not to damage nearby existing structures and underground utilities. As predominantly clay soil conditions are expected, we recommend using large sheepsfoot or padfoot rollers for the compaction operations. We recommend at least several passes be made with the compaction equipment. In some areas, moisture conditioning and/or undercutting may be necessary to enhance the effectiveness of the compaction operation.

Prior to raising site grades, proofroll the exposed subgrade in the presence of SME. The purpose of the proofroll is to locate areas of unsuitably loose or disturbed subgrade. Mechanically improve areas of unsuitable subgrade revealed during proofrolling by compaction in-place, if feasible. Soils that cannot be improved in-place must be removed and replaced with engineered fill or chemically stabilized. Additional laboratory testing is required to verify the suitability of chemical stabilization on this project site. Perform proofrolling with a fully loaded, tandem-axle truck or other suitable-sized pneumatic-tire construction equipment.

After the exposed subgrade is proofrolled and improved as needed, and after the surface is thoroughly compacted, engineered fill may be placed on the prepared subgrade to establish final design subgrade levels. Refer to Section 2.1.5 of this report for materials and compaction requirements for engineered fill.

2.1.3 SUBGRADE PREPARATION FOR FLOOR SLABS

The final floor slab subgrade for the proposed buildings needs to consist of natural clay or existing clay fill evaluated and passed proofroll testing, or engineered fill placed over those soils. We recommend a subgrade modulus of 100 pci to design floor slabs supported on properly prepared subgrade described herein. This value of subgrade modulus is based on correlations with soil type from plate load tests and is the ratio of load in pounds per square-inch to a 0.05-inch deflection of a 30-inch-diameter bearing plate.

Prior to concrete placement for floor slabs, SME must observe and test the building pads subgrade to identify areas disturbed during construction activities and verify the final subgrade conditions are suitable for floor slab support. Recompact unsuitable subgrade identified by SME or remove and replace it with engineered fill. Proofroll the final subgrade in areas accessible with large equipment. SME needs to test areas inaccessible to proofrolling equipment using hand-operated equipment such as cone penetrometers, hand auger probes, and density gauges.

We recommend the top 6 inches of the slab subbase consist of an approved INDOT No. 53 crushed aggregate to provide a leveling surface for construction of the slab and a moisture capillary break between the slab and the underlying soils. The thickness of aggregate needed to provide a stable construction platform will depend on the condition of subgrade soils during construction and the type and volume of construction equipment trafficking the prepared subgrade. We expect a thicker aggregate profile will be needed for heavily trafficked areas and/or dedicated crane pads. The dense graded aggregate must be compacted per Section 2.1.5.

Place a vapor retarder below floor slabs to receive an impermeable floor finish/seal or a floor covering which would retard vapor transmission. The location of the vapor retarder (relative to the subbase) needs to be determined by the design Architect/Engineer based on the intended floor usage, planned finishes, and ACI recommendations.

Separate floor slabs by isolation joints from structural walls and columns to permit relative movement. Allow a minimum of 6 inches of engineered fill between the bottom of the slab and the top of the shallow foundation below.

Protect the slab-on-grade subgrade soils from frost action during winter construction. Frozen soils must be thawed and compacted or removed and replaced prior to slab-on-grade construction.

2.1.4 PAVEMENT SUBGRADE PREPERATION RECOMMENDATIONS

Based on the borings performed in the proposed pavement areas (borings B5 and B6) we expect the exposed subgrade soils at the design subgrade level in the pavement areas will consist predominantly of natural clay. Subgrades need to be prepared in accordance with Sections 2.1.2 and 2.1.5 of this report, and as follows. Subgrade preparation and the aggregate base layers need to extend out to at least 18 inches beyond the edge of pavement or beyond the back of curbs to provide support for the outer edges of pavement.

After stripping surficial materials, cutting to design subgrade levels, removing/replacing any organics or deleterious materials (including tree mats), but prior to raising grades, compact the exposed subgrade to a minimum of 95 percent of the maximum dry density determined by the Modified Proctor test. Proofroll the resulting subgrade using a fully loaded tandem axle dump truck. Improve areas exhibiting deflection greater than ½ inch as described in Section 2.1.2. Place and compact engineered fill as presented in Section 2.1.5.

Prior to the placement of the aggregate base, we recommend fine-grading the subgrade to slope downward toward the stormwater drainage structures. Fine-grading of the underlying subgrade will be critical to minimize low-spots below the aggregate base where water can pond, likely resulting in moisture changes and undesirable early pavement distress. Fine-grading the subgrade is important for drainage of perched groundwater, and to achieve a uniform thickness of base course to be placed throughout each of the pavement sections. Also, we recommend installing underdrains at/through low spots in the prepared subgrade to facilitate drainage of perched groundwater.

Prior to placement of the aggregate base, proofroll the subgrade again and repair any areas of disturbed or unstable subgrade identified by proofrolling. Once the subgrade passes the final proofroll test, we recommend fine-grading the subgrade again and then placing the pavement layers soon thereafter to avoid further subgrade disturbance. Place and compact aggregate base to a minimum of 95 percent of the maximum dry density determined by the Modified Proctor test. Just prior to placing the HMA layers, proofroll the aggregate base and recompact areas exhibiting deflection greater than ¼ inch as described in Section 2.1.2. Place and compact engineered fill as presented in Section 2.1.5.

Limit construction traffic on the prepared subgrade once the aggregate base layer is placed. Use designated haul roads and staging areas for heavy construction vehicles and equipment/materials storage. Haul roads should be constructed using a layer of crushed stone, possibly in combination with a high-strength woven geotextile fabric or geogrid, if necessary to stabilize and protect the subgrade. Contact SME for assistance with the type and quantity of stabilization required based on field conditions during construction.

Based on the borings, laboratory tests, climate moisture/temperature effects, and assumed implementation of our recommended subgrade preparations, we have estimated a subgrade resilient modulus of 3,500 psi for determining the pavement design sections.

2.1.5 ENGINEERED FILL REQUIREMENTS

The fill needs to be free of frozen soil, significant organics and construction debris, potentially expansive and/or chemically active materials, particle sizes hindering compaction, cobbles, boulders, and tree roots. If the proposed engineered fill soils contain more than four percent organics, or debris larger than 6 inches in nominal diameter, do not use soils for engineered fill. Also, if debris material is significantly variable in nature, suspect in origin, or greater than about five percent of the soil (by weight or volume), do not use soils for engineered fill.

Spread fill in level layers with a loose thickness appropriate for the type of equipment used to obtain compaction, but not exceeding 9 inches thick. Thinner lifts will be required in confined spaces and where compaction is achieved with walk-behind rollers/compactors or plate compactors mounted on a backhoe or excavator (e.g., Ho-Pac®). Compact the fill to a minimum of 95 percent of the maximum dry density as determined in accordance with the Modified Proctor test. Compact sand fill with a smooth drum vibratory roller or vibratory plate compactors. Compact clay fill by overlapping passes with a sheepsfoot or padfoot roller at a moisture content within two percent below and above the optimum moisture.

Based on the results of our field and laboratory testing, some, but not all, of the existing inorganic natural clay soils and clay fill is considered suitable for reuse as engineered fill for general site grading. However, the clays are considered moisture-sensitive and could be difficult to compact, particularly when moisture contents are above the optimum moisture content. Clays will be difficult to compact using smaller hand-operated compaction equipment and are not expected to be suitable for reuse as backfill for foundation excavations or utility trenches. These clay soils should also not be used as fill in areas where drainage is required. In cases where the contractor must compact clayey subgrade soils, it may be necessary to moisture condition the clayey soils. Moisture conditioning is more easily performed during the warmer, drier summer months and may not be feasible during cold or wet times of the year. Unsuitable soils may be placed in non-structural areas of the site or remove them from the site. Imported fill needs to be sampled at the source, tested, and approved prior to importing to the site. We recommend imported fill consist of material meeting the gradation requirements of INDOT No. 53 crushed aggregate. Where drainage is required, INDOT No. 8 aggregate can be used for engineered fill.

Coarse crushed aggregate used to backfill undercuts or to stabilize subgrades need to consist of a well graded crushed natural aggregate ranging from 1 to 3 inches in size with no more than 7 percent by weight passing the No. 200 sieve, such as INDOT No. 2 aggregate. Spread the crushed material in layers not exceeding 12 inches and compact with a vibratory compactor until no further densification is observed. The recommended crushed aggregate must be compacted using a steel-drum vibratory roller, a static roller (in the case of disturbed subgrades), vibratory plate compactor mounted on an excavator or backhoe (hoe-pac). In cases where granular engineered fill will be placed over the crushed aggregate, choke the surface of the coarse crushed material with a layer of at least 6 inches of INDOT No. 53

crushed aggregate or cover the surface with a suitable non-woven geotextile fabric to prevent migration of the granular engineered fill into the coarser crushed material.

2.2 FOUNDATIONS

2.2.1 SUBGRADE VERIFICATION

Geotechnical foundation design recommendations are provided based on borings covering only a small area of the site. On-site observations and testing of the foundation subgrades are necessary to verify the subgrade exposed at the foundation bearing surface is consistent with the borings and is suitable for the design soil bearing pressure. By preparing this geotechnical evaluation report, SME is best suited to observe and test foundation subgrades during construction and to verify the geotechnical recommendations of this report and the geotechnical related design requirements of this project are incorporated into the construction. The recommendations of this report are based on the assumption SME will further evaluate the bearing soils during construction.

2.2.2 SHALLOW FOUNDATIONS

Shallow spread foundations are recommended for support of the proposed storage facility building and salt storage barn. A maximum net allowable bearing pressure of 3,000 pounds per square-foot (psf) can be used to design shallow isolated (column) and continuous (wall) foundations supported on suitable natural stiff to hard clays or engineered fill placed over suitable natural clays. This design bearing pressure will achieve a global safety factor of three or more for general shear failure. Suitable bearing soils were observed below the surface materials and fill at the boring locations.

Once each foundation area is exposed, SME must observe and test foundation subgrade conditions to verify suitable soils are encountered or improvements are performed, as needed, prior to foundation construction. SME will utilize a test method capable of testing the soils several feet below the design bearing level. Unsuitable soils must be mechanically improved (i.e., compacted) in-place. Where unsuitable soils cannot be mechanically improved in-place, deepen the foundation excavations (undercut unsuitable soils) to encounter suitable bearing material below. Foundations can then be constructed to bear directly at this lower level where suitable subgrade is encountered, or the design bearing level can be reestablished with engineered fill (refer to Section 2.1.5).

Oversize undercuts that are backfilled with engineered fill to the design bearing level laterally by extending excavations outward on a two vertical to one horizontal slope from the edge of the foundation as shown on Image No. 2.

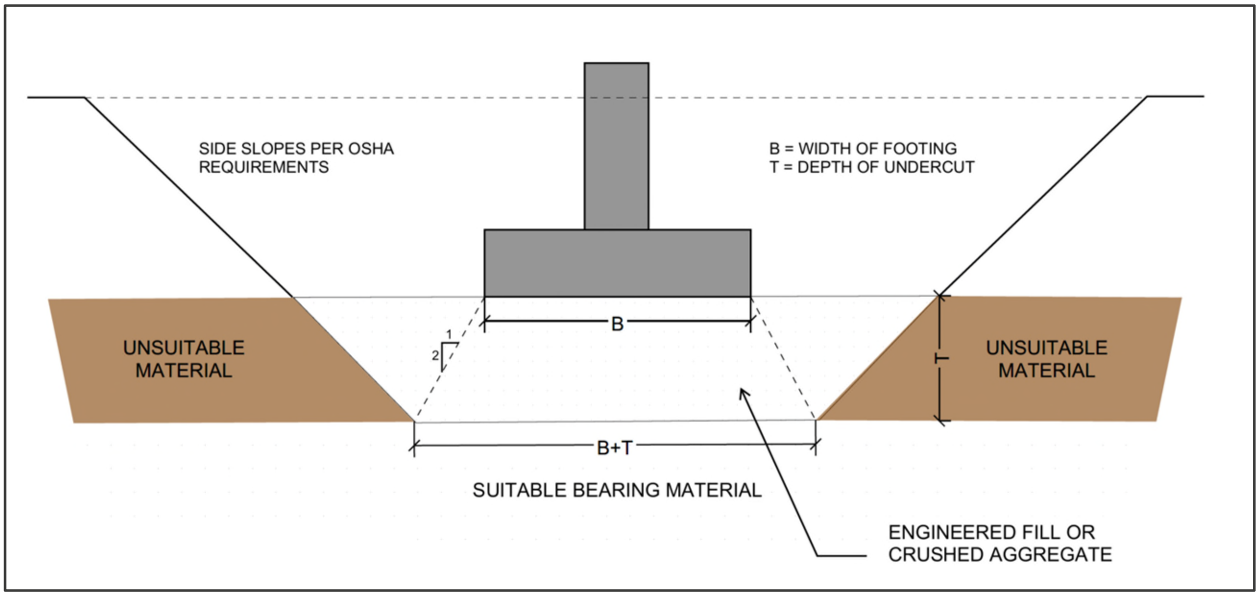


IMAGE NO. 2: Foundation Undercutting Diagram

Exterior foundations must have a minimum embedment of 30 inches below the lowest adjacent exterior grade in unheated areas for protection against frost movement during normal winters. Interior foundations in heated areas can be constructed at shallower levels on suitable soils but deeper undercuts through fill soils may be required. Footing trenches need to be excavated to a level bearing surface, cleaned of mud and loose cuttings, and protected against water accumulation from rainfall, surface drainage, or excavation sidewall seepage prior to placing concrete. Place foundation concrete as soon as foundation excavations have been completed, and the design bearing pressure verified to reduce the potential for disturbance of the foundation subgrade. In cases where the excavation will remain exposed for a longer period of time, protect the subgrade soils with a concrete mud mat and protect the bearing soils from freezing if the work is performed during seasonally cold weather.

For frost heave considerations, vertical excavation sidewalls must be maintained during foundation concrete placement and the side walls must not be allowed to “mushroom out” near the top. If vertical earthen sidewalls cannot be maintained, it will be necessary to slope back the foundation excavations and form foundation sidewalls to maintain vertical faces for foundations and reduce the potentially adverse effects resulting from frost heave.

Use a minimum width of 18 inches to design continuous (wall) foundations and a minimum dimension of 30 inches to design isolated (column) foundations for bearing capacity and settlement considerations. In cases where structural loading is light, the minimum recommended foundation sizes, and not the design bearing pressure, may govern the sizes of the foundations.

Total settlements for shallow spread foundations bearing on suitable clays are estimated to be 1 inch or less, and differential settlements are estimated to be less than one-half of the total settlement. These settlement estimates are based on the boring information, the design maximum net allowable soil bearing pressure, the anticipated design structural loads (outlined in Section 1.2), our experience with similar structures and soil conditions, and field verification of suitable bearing soils by SME.

Based on the proposed construction and estimated design loads, we estimate that uplift loading may control the foundation design, which may require wider footings and/or embedment depths extending below the frost line. Soil backfill placed over top of spread footings can be used to resist uplift loading. A moist soil unit weight of 120 pcf can be used for lean clay backfill placed above footings, provided the backfill is placed, compacted, and tested in accordance with Section 2.1.5 of this report.

2.3 SEISMIC SITE CLASS

Based on the subsurface information obtained from the borings to a maximum depth of 20 feet and our previous experience in the project area, seismic site Class C applies to this site in accordance with the most current version of the Indiana Building Code referencing Table 20.3-1 in ASCE Standard ASCE/SEI 7.

2.4 GENERAL SLOPE CONSIDERATIONS

We understand the south side of the proposed salt barn will be located adjacent to an existing slope, of which the geometry will be modified to address the proposed construction and proposed 3:1 slope. Our scope of work did not include an assessment of slope stability at this location. Construction of a salt barn adjacent to the slope crest will induce a surcharge on the slope, which could initiate slope instability. We recommend that the slope stability be analyzed to evaluate the impact of the proposed loading on the slope. SME can assist with these additional analyses upon request.

3. EXCAVATION AND GROUNDWATER MANAGEMENT

Groundwater seepage into shallow excavations is not anticipated to be a significant factor during construction. However, groundwater infiltration and/or accumulations from precipitation events, surface run-off, or perched groundwater conditions could be encountered. Standard sump pit and pumping procedures are expected to be adequate to control these accumulations on a localized basis. Higher capacity dewatering would be necessary for excavations extending more than 1 foot below the groundwater level at this site. A working surface of either crushed aggregate may be required to protect the exposed subgrade where seepage is encountered.

Based on the site history and boring logs, there is potential for random areas of undocumented fill with variable depths within the proposed buildings and pavement areas. We recommend the bid documents require prospective contractors to include unit prices for excavating disturbed soils and other unsuitable soils and replacing them with engineered fill. Also, establish a contingency in the construction budget for this work. Actual quantities can be verified during construction by measuring excavation volumes, counting truck loads, or a combination of the two methods.

Take care during demolition and earthwork operations to protect adjoining and adjacent utility structures. Do not undermine existing structures. Where necessary, install temporary shoring/bracing to properly shore/brace existing structures and protect them from distress.

The contractor must provide safely sloped excavations or adequately constructed and braced shoring systems in accordance with federal, state and local safety regulations for individuals working in an excavation that may expose them to the danger of moving ground. If material is stored or heavy equipment is operated near an excavation, use appropriate shoring to resist the extra pressure due to the superimposed loads.

Handle, transport and dispose excavated materials and groundwater in accordance with applicable environmental regulatory requirements. Refer to the project environmental consultant for additional information regarding handling and/or disposal of onsite soils and/or groundwater.

4. EVALUATION PROCEDURES

4.1 FIELD EXPLORATION

The proposed number, locations, and depths of the borings were determined jointly by CEI and SME. SME located the borings in field using our hand-held global positioning system (GPS). SME estimated the existing ground surface elevations at the borings to the nearest 1 foot based on the topographic information referenced in Section 1.

The borings were advanced using a rotary drill rig equipped with continuous flight augers. The borings included soil sampling based on Split-Barrel Sampling procedures.

Groundwater level measurements were recorded during and immediately after completion of each boring. After completion of drilling and obtaining groundwater level measurements the boreholes were backfilled with auger cuttings.

The Field Testing Procedures in Appendix B provides more detailed descriptions of field tests typically performed by SME and referenced in this report.

Soil samples recovered from the field exploration were taken to our laboratory for further observations and testing.

4.2 LABORATORY TESTING

The laboratory testing program consisted of visually classifying the recovered samples in accordance with ASTM D-2488. Based on the laboratory testing, we prepared soil descriptions and assigned a group symbol for the various soil strata observed based on the Unified Soil Classification System (USCS). In addition, moisture content and hand penetrometer tests were performed on portions of cohesive samples obtained.

Upon completion of the laboratory testing, boring logs were prepared and include the soil descriptions, penetration resistances, pertinent field observations, and the results of the laboratory testing. Each log also includes the estimated existing ground surface elevation. Explanations of symbols and terms used on the boring logs are provided on the Boring Log Terminology sheet included in Appendix A.

Soil samples are normally retained in our laboratory for 60 days and are then disposed, unless instructed otherwise.

The Laboratory Testing Procedures in Appendix B provides descriptions of the laboratory tests. The results of the laboratory tests are presented on the boring logs and included in Appendix A.

5. SIGNATURES

PREPARED BY:

Austin J Smith

Austin J. Smith, EIT
Staff Engineer

REVIEWED BY:

Wesley J. Hemp, PE, PG (KY, LA), BC.GE, LEED AP
Senior Consultant

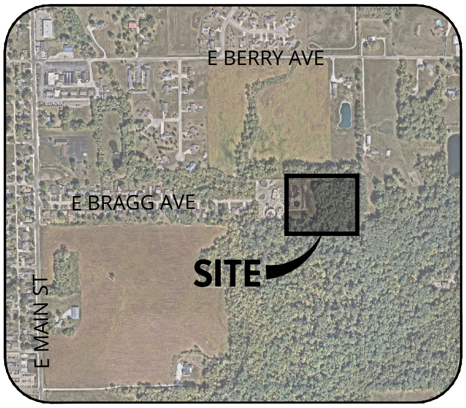
PREPARED BY:

Jamie M. Bates

Jamie M. Bates, PE
Senior Project Manager

APPENDIX A

BORING LOCATION DIAGRAM (FIGURE NO. 1)
BORING LOG TERMINOLOGY
BORING LOGS (B1 THROUGH B6)



LOCATION MAP

NOT TO SCALE

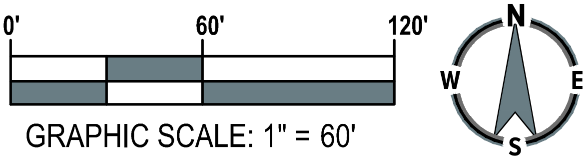
LEGEND

⌖ APPROXIMATE BORING LOCATION

NOTES:

1. BASE DRAWING INFORMATION TAKEN FROM A DRAWING TITLED "PRELIMINARY SITE PLAN" WITH A LATEST REVISION DATE OF 9-26-2025, PREPARED BY HALSTEAD ARCHITECTS.

2. AERIAL IMAGE TAKEN FROM NEARMAP WITH AN IMAGE DATE OF 9-18-2025.



GRAPHIC SCALE: 1" = 60'

AERIAL BACKGROUND OBTAINED FROM: NEARMAP IMAGE DATE 9-18-2025.

No.	Revision Date		
		Date	11-7-2025
		CADD	M. NOWAK-ROCHFORD
		Designer	J. BATES
		Project No.	101022.00

DRAWING NOTE: SCALE DEPICTED IS MEANT FOR 8.5"x11" AND WILL SCALE INCORRECTLY IF PRINTED ON ANY OTHER SIZE MEDIA

BORING LOCATION DIAGRAM
UPLAND WWTP IMPROVEMENTS
UPLAND, INDIANA

NO REPRODUCTION SHALL BE MADE WITHOUT THE PRIOR CONSENT OF SME

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Figure No. **FIGURE NO. 1**

SME

BORING LOG TERMINOLOGY

UNIFIED SOIL CLASSIFICATION AND SYMBOL CHART

COARSE-GRAINED SOIL
(more than 50% of material is larger than No. 200 sieve size.)

GRAVEL More than 50% of coarse fraction larger than No. 4 sieve size	Clean Gravel (Less than 5% fines)		
	▨	GW	Well-graded gravel; gravel-sand mixtures, little or no fines
	▨	GP	Poorly-graded gravel; gravel-sand mixtures, little or no fines
	Gravel with fines (More than 12% fines)		
	▨	GM	Silty gravel; gravel-sand-silt mixtures
	▨	GC	Clayey gravel; gravel-sand-clay mixtures
SAND 50% or more of coarse fraction smaller than No. 4 sieve size	Clean Sand (Less than 5% fines)		
	▨	SW	Well-graded sand; sand-gravel mixtures, little or no fines
	▨	SP	Poorly graded sand; sand-gravel mixtures, little or no fines
	Sand with fines (More than 12% fines)		
	▨	SM	Silty sand; sand-silt-gravel mixtures
	▨	SC	Clayey sand; sand–clay-gravel mixtures

FINE-GRAINED SOIL
(50% or more of material is smaller than No. 200 sieve size)

SILT AND CLAY Liquid limit less than 50%	▨	ML	Inorganic silt; sandy silt or gravelly silt with slight plasticity
	▨	CL	Inorganic clay of low plasticity; lean clay, sandy clay, gravelly clay
	▨	OL	Organic silt and organic clay of low plasticity
SILT AND CLAY Liquid limit 50% or greater	▨	MH	Inorganic silt of high plasticity, elastic silt
	▨	CH	Inorganic clay of high plasticity, fat clay
	▨	OH	Organic silt and organic clay of high plasticity
HIGHLY ORGANIC SOIL	▨	PT	Peat and other highly organic soil

OTHER MATERIAL SYMBOLS

▨ Topsoil	▨ Void	▨ Sandstone
▨ Asphalt Concrete	▨ Glacial Till	▨ Siltstone
▨ Aggregate Base	▨ Coal	▨ Limestone
▨ Portland Cement Concrete	▨ Shale	▨ Fill

LABORATORY CLASSIFICATION CRITERIA

GW	$C_U = \frac{D_{60}}{D_{10}}$ greater than 4; $C_C = \frac{D_{30}^{\ 2}}{D_{10} \times D_{60}}$ between 1 and 3	
GP	Not meeting all gradation requirements for GW	
GM	Atterberg limits below “A” line or PI less than 4	Above “A” line with PI between 4 and 7 are borderline cases requiring use of dual symbols
GC	Atterberg limits above “A” line with PI greater than 7	
SW	$C_U = \frac{D_{60}}{D_{10}}$ greater than 6; $C_C = \frac{D_{30}^{\ 2}}{D_{10} \times D_{60}}$ between 1 and 3	
SP	Not meeting all gradation requirements for SW	
SM	Atterberg limits below “A” line or PI less than 4	Above “A” line with PI between 4 and 7 are borderline cases requiring use of dual symbols
SC	Atterberg limits above “A” line with PI greater than 7	

Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows:

Less than 5 percent……………………….……...GW, GP, SW, SP
More than 12 percent…………………….………GM, GC, SM, SC
5 to 12 percent……………...……..Cases requiring dual symbols

- SP-SM or SW-SM (SAND with Silt or SAND with Silt and Gravel)
- SP-SC or SW-SC (SAND with Clay or SAND with Clay and Gravel)
- GP-GM or GW-GM (GRAVEL with Silt or GRAVEL with Silt and Sand)
- GP-GC or GW-GC (GRAVEL with Clay or GRAVEL with Clay and Sand)

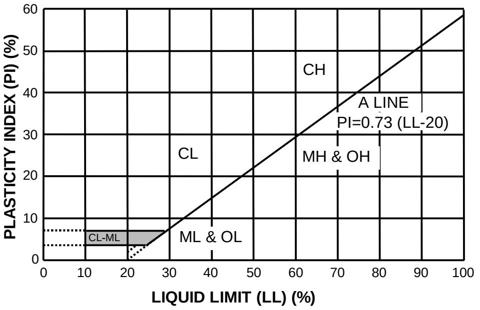
If the fines are CL-ML:

- SC-SM (SILTY CLAYEY SAND or SILTY CLAYEY SAND with Gravel)
- SM-SC (CLAYEY SILTY SAND or CLAYEY SILTY SAND with Gravel)
- GC-GM (SILTY CLAYEY GRAVEL or SILTY CLAYEY GRAVEL with Sand)

PARTICLE SIZES

Boulders - Greater than 12 inches
Cobbles - 3 inches to 12 inches
Gravel- Coarse - 3/4 inches to 3 inches
Fine - No. 4 to 3/4 inches
Sand- Coarse - No. 10 to No. 4
Medium - No. 40 to No. 10
Fine - No. 200 to No. 40
Silt and Clay - Less than (0.074 mm)

PLASTICITY CHART



VISUAL MANUAL PROCEDURE

When laboratory tests are not performed to confirm the classification of soils exhibiting borderline classifications, the two possible classifications would be separated with a slash, as follows:

For soils where it is difficult to distinguish if it is a coarse or fine-grained soil:

- SC/CL (CLAYEY SAND to Sandy LEAN CLAY)
- SM/ML (SILTY SAND to SANDY SILT)
- GC/CL (CLAYEY GRAVEL to Gravelly LEAN CLAY)
- GM/ML (SILTY GRAVEL to Gravelly SILT)

For soils where it is difficult to distinguish if it is sand or gravel, poorly or well-graded sand or gravel; silt or clay; or plastic or non-plastic silt or clay:

- SP/GP or SW/GW (SAND with Gravel to GRAVEL with Sand)
- SC/GC (CLAYEY SAND with Gravel to CLAYEY GRAVEL with Sand)
- SM/GM (SILTY SAND with Gravel to SILTY GRAVEL with Sand)
- SW/SP (SAND or SAND with Gravel)
- GP/GW (GRAVEL or GRAVEL with Sand)
- SC/SM (CLAYEY to SILTY SAND)
- GM/GC (SILTY to CLAYEY GRAVEL)
- CL/ML (SILTY CLAY)
- ML/CL (CLAYEY SILT)
- CH/MH (FAT CLAY to ELASTIC SILT)
- CL/CH (LEAN to FAT CLAY)
- MH/ML (ELASTIC SILT to SILT)

DRILLING AND SAMPLING ABBREVIATIONS

2ST – Shelby Tube – 2” O.D.
3ST – Shelby Tube – 3” O.D.
AS – Auger Sample
GS – Grab Sample
LS – Liner Sample
NR – No Recovery
PM – Pressuremeter
RC – Rock Core diamond bit. NX size, except where noted
SB – Split Barrel Sample 1-3/8” I.D., 2” O.D., except where noted
VS – Vane Shear
WS – Wash Sample

OTHER ABBREVIATIONS

WOH – Weight of Hammer
WOR – Weight of Rods
SP – Soil Probe
PID – Photo Ionization Device
FID – Flame Ionization Device

DEPOSITIONAL FEATURES

Parting – as much as 1/16 inch thick
Seam – 1/16 inch to 1/2 inch thick
Layer – 1/2 inch to 12 inches thick
Stratum – greater than 12 inches thick
Pocket – deposit of limited lateral extent
Lens – lenticular deposit
Hardpan/Till – an unstratified, consolidated or cemented mixture of clay, silt, sand and/or gravel, the size/shape of the constituents vary widely
Lacustrine – soil deposited by lake water
Mottled – soil irregularly marked with spots of different colors that vary in number and size
Varved – alternating partings or seams of silt and/or clay
Occasional – one or less per foot of thickness
Frequent – more than one per foot of thickness
Interbedded – strata of soil or beds of rock lying between or alternating with other strata of a different nature

DESCRIPTION OF RELATIVE QUANTITIES

The visual-manual procedure uses the following terms to describe the relative quantities of notable foreign materials, gravel, sand or fines:

Trace – particles are present but estimated to be less than 5%
Few – 5 to 10%
Little – 15 to 25%
Some – 30 to 45%
Mostly – 50 to 100%

CLASSIFICATION TERMINOLOGY AND CORRELATIONS

Cohesionless Soils

Relative Density	N_{60} (N-Value) (Blows per foot)
Very Loose	0 to 4
Loose	5 to 10
Medium Dense	11 to 30
Dense	31 to 50
Very Dense	51 to 80
Extremely Dense	Over 81

Cohesive Soils

Consistency	N_{60} (N-Value) (Blows per foot)	Undrained Shear Strength (kips/ft^2)
Very Soft	<2	0.25 or less
Soft	2 - 4	> 0.25 to 0.50
Medium	5 - 8	> 0.50 to 1.0
Stiff	9 - 15	> 1.0 to 2.0
Very Stiff	16 - 30	> 2.0 to 4.0
Hard	> 30	> 4.0 or greater

Standard Penetration ‘N-Value’ = Blows per foot of a 140-pound hammer falling 30 inches on a 2-inch O.D. split barrel sampler, except where noted. N60 values as reported on boring logs represent raw N-values corrected for hammer efficiency only.

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BORING B 1

PAGE 1 OF 1

BORING DEPTH: 20 FEET

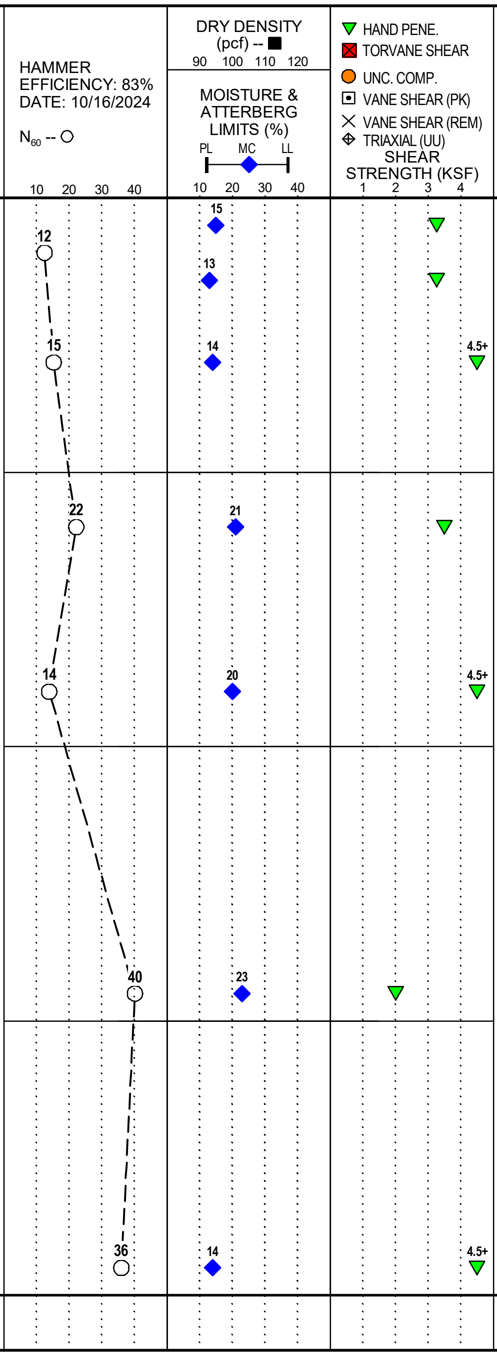
PROJECT NAME: Upland WWTP Improvements
PROJECT NUMBER: 101022.00
CLIENT: Commonwealth Engineers, Inc.
PROJECT LOCATION: Upland, Indiana
DATE STARTED: 10/24/25
COMPLETED: 10/24/25
BORING METHOD: Hollow Stem Augers
DRILLER: CS/MC
RIG NO.: 558 Mobile B-29 - Truck
LOGGED BY: AJS
CHECKED BY: JMB

ELEVATION: 916± FT

HAMMER EFFICIENCY: 83%
DATE: 10/16/2024

DEPTH (FEET)	PROFILE DESCRIPTION	ELEVATION (FEET)	SAMPLE TYPE/NO. INTERVAL	RECOVERY LENGTH (INCHES)	SPT BLOWS PER SIX INCHES	N60	MOISTURE (%)	SHEAR STRENGTH (KSF)	REMARKS
0.3	3 inches of TOPSOIL	915.7							
2.0	FILL- LEAN CLAY- Occasional Roots- Brown- Stiff to Very Stiff (CL)	914.0	SB1	18	4 4 5 4	12	15 13		
5.0	LEAN CLAY- Few Gravel- Brown- Stiff to Hard (CL)	911.0	SB2	6	4 5 6 9	15	14	4.5+	
13.5	LEAN CLAY- Occasional Silt Seams- Brown- Stiff to Hard (CL)	902.5	SB3 SB4	18 18	7 10 6 5 7 5 5 6	22 14	21 20	4.5+	
18.5	LEAN CLAY- Few Gravel- Brown- Very Stiff to Hard (CL)	897.5	SB5	8	9 12 17	40	23		
20.0	LEAN CLAY- Occasional Silt Seams- Few Gravel- Hard (CL)	896.0	SB6	10	6 10 16	36	14	4.5+	

END OF BORING AT 20.0 FEET.



GROUNDWATER & BACKFILL INFORMATION

GROUNDWATER WAS NOT ENCOUNTERED

	DEPTH (FT)	ELEV (FT)
CAVE-IN OF BOREHOLE AT:	9.0	907.0

BACKFILL METHOD: Auger Cuttings

NOTES:
1. The indicated stratification lines are approximate. The in-situ transitions between materials may be gradual.
2. The colors depicted on the symbolic profile are solely for visualization purposes and do not necessarily represent the in-situ colors encountered.

BORING B 2

PAGE 1 OF 1

BORING DEPTH: 20 FEET

PROJECT NAME: Upland WWTP Improvements
PROJECT NUMBER: 101022.00
CLIENT: Commonwealth Engineers, Inc.
PROJECT LOCATION: Upland, Indiana

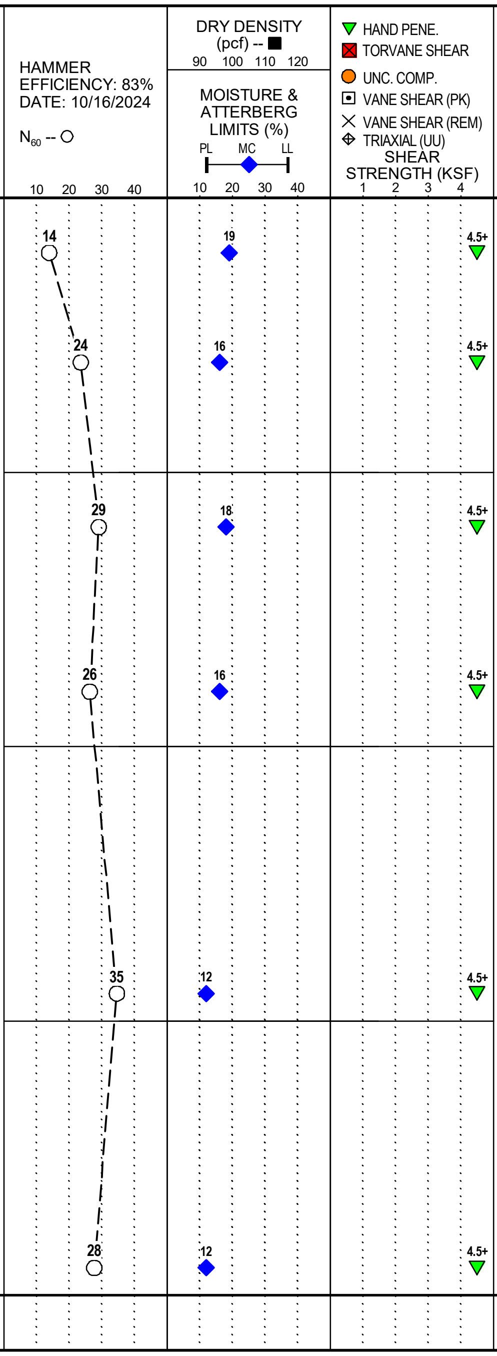
DATE STARTED: 10/24/25
COMPLETED: 10/24/25
BORING METHOD: Hollow Stem Augers
DRILLER: CS/MC
RIG NO.: 558 Mobile B-29 - Truck
LOGGED BY: AJS
CHECKED BY: JMB

ELEVATION: 915± FT

HAMMER EFFICIENCY: 83%
DATE: 10/16/2024

ELEVATION (FEET)	DEPTH (FEET)	PROFILE DESCRIPTION	SAMPLE TYPE/NO. INTERVAL	RECOVERY LENGTH (INCHES)	SPT BLOWS PER SIX INCHES	N60	MC (%)	HAND PENE. (KSF)	REMARKS
914.7	0.3	3 inches of TOPSOIL							
913.0	2.0	LEAN CLAY- Frequent Roots- Brown- Stiff to Hard (CL)	SB1	14	2 5 5 6	14	19	4.5+	
907.0	8.0	LEAN CLAY- Brown and Gray- Very Stiff to Hard (CL)	SB2	16	7 8 9 10	24	16	4.5+	
			SB3	16	12 11 10 12	29	18	4.5+	
901.5	13.5	LEAN CLAY- Dark Brown- Very Stiff to Hard (CL)	SB4	17	9 10 9 10	26	16	4.5+	
895.0	20.0	Sandy LEAN CLAY- Brown- Very Stiff to Hard (CL)	SB5	14	7 9 16	35	12	4.5+	
			SB6	16	8 8 12	28	12	4.5+	

END OF BORING AT 20.0 FEET.



GROUNDWATER & BACKFILL INFORMATION

GROUNDWATER WAS NOT ENCOUNTERED

	DEPTH (FT)	ELEV (FT)
CAVE-IN OF BOREHOLE AT:	6.0	909.0

BACKFILL METHOD: Auger Cuttings

NOTES:
1. The indicated stratification lines are approximate. The in-situ transitions between materials may be gradual.
2. The colors depicted on the symbolic profile are solely for visualization purposes and do not necessarily represent the in-situ colors encountered.

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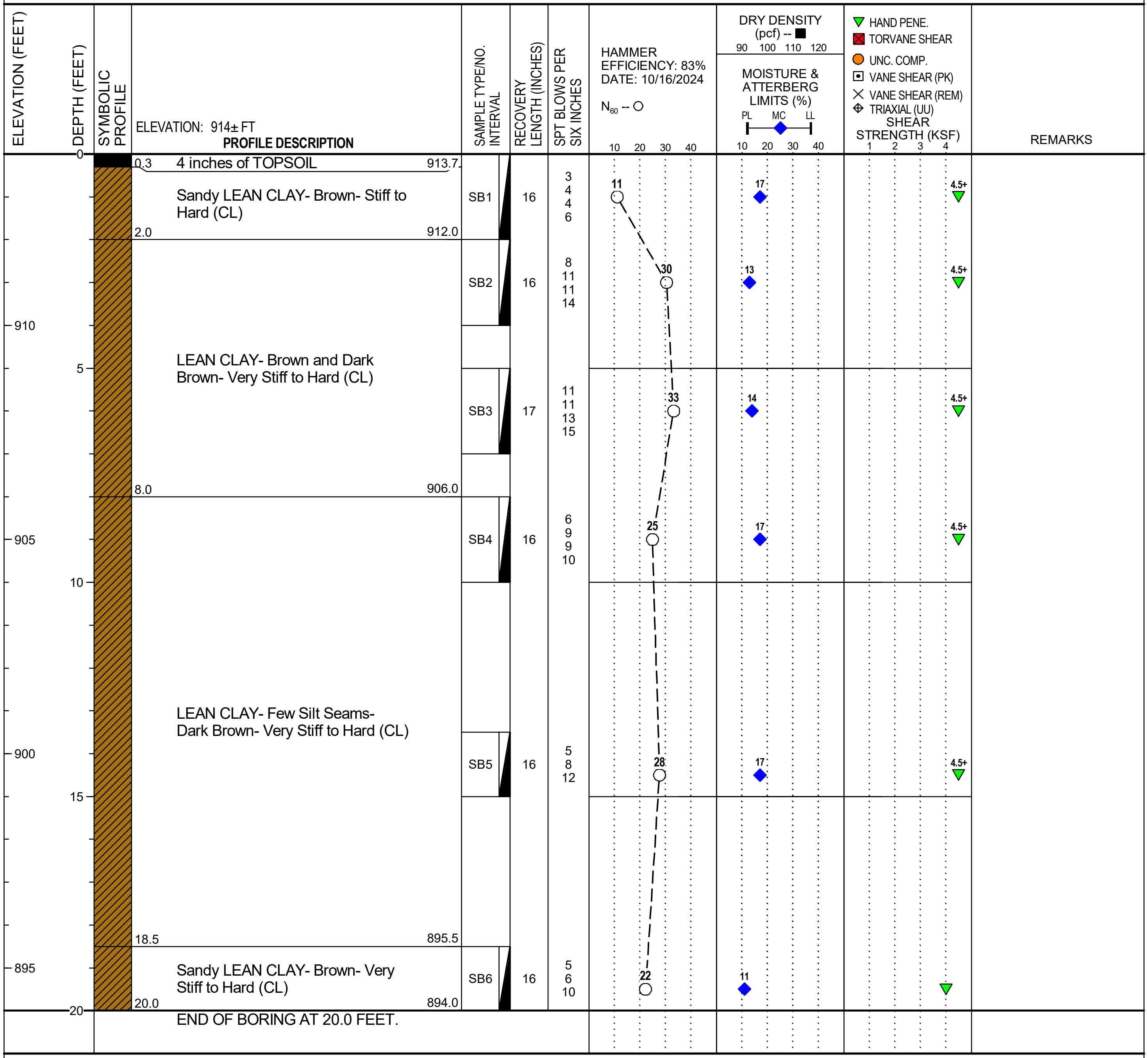
BORING B 3

PAGE 1 OF 1

BORING DEPTH: 20 FEET

PROJECT NAME: Upland WWTP Improvements
PROJECT NUMBER: 101022.00
CLIENT: Commonwealth Engineers, Inc.
PROJECT LOCATION: Upland, Indiana

DATE STARTED: 10/24/25
COMPLETED: 10/24/25
BORING METHOD: Hollow Stem Augers
DRILLER: CS/MC
RIG NO.: 558 Mobile B-29 - Truck
LOGGED BY: AJS
CHECKED BY: JMB



ELEVATION: 914± FT

DEPTH (FEET)	PROFILE DESCRIPTION	ELEVATION (FEET)
0.3	4 inches of TOPSOIL	913.7
2.0	Sandy LEAN CLAY- Brown- Stiff to Hard (CL)	912.0
8.0	LEAN CLAY- Brown and Dark Brown- Very Stiff to Hard (CL)	906.0
18.5	LEAN CLAY- Few Silt Seams- Dark Brown- Very Stiff to Hard (CL)	895.5
20.0	Sandy LEAN CLAY- Brown- Very Stiff to Hard (CL)	894.0

END OF BORING AT 20.0 FEET.

SAMPLE TYPE/NO. INTERVAL	RECOVERY LENGTH (INCHES)	SPT BLOWS PER SIX INCHES	N60	MC (%)	HAND PENE. (KSF)
SB1	16	3 4 4 6	11	17	4.5+
SB2	16	8 11 11 14	30	13	4.5+
SB3	17	11 11 13 15	33	14	4.5+
SB4	16	6 9 9 10	25	17	4.5+
SB5	16	5 8 12	28	17	4.5+
SB6	16	5 6 10	22	11	

GROUNDWATER & BACKFILL INFORMATION

GROUNDWATER WAS NOT ENCOUNTERED

	DEPTH (FT)	ELEV (FT)
CAVE-IN OF BOREHOLE AT:	9.0	905.0

BACKFILL METHOD: Auger Cuttings

NOTES:
1. The indicated stratification lines are approximate. The in-situ transitions between materials may be gradual.
2. The colors depicted on the symbolic profile are solely for visualization purposes and do not necessarily represent the in-situ colors encountered.

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BORING B 4

PAGE 1 OF 1

BORING DEPTH: 20 FEET

PROJECT NAME: Upland WWTP Improvements
PROJECT NUMBER: 101022.00
CLIENT: Commonwealth Engineers, Inc.
PROJECT LOCATION: Upland, Indiana

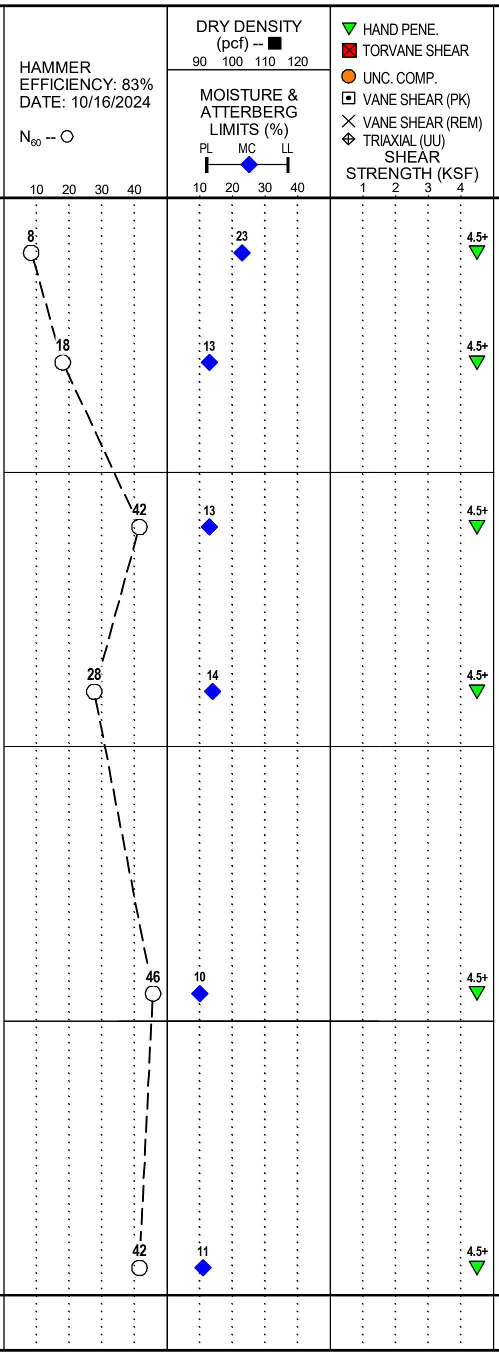
DATE STARTED: 10/24/25
COMPLETED: 10/24/25
BORING METHOD: Hollow Stem Augers
DRILLER: CS/MC
RIG NO.: 558 Mobile B-29 - Truck
LOGGED BY: AJS
CHECKED BY: JMB

ELEVATION: 914± FT

DEPTH (FEET)	PROFILE DESCRIPTION	ELEVATION (FEET)
0.5	6 inches of TOPSOIL	913.5
2.0	Sandy LEAN CLAY- Few Gravel- Brown- Medium to Hard (CL)	912.0
8.0	LEAN CLAY- Brown and Gray- Very Stiff to Hard (CL)	906.0
18.5	LEAN CLAY- Brown- Very Stiff to Hard (CL)	895.5
20.0	Sandy LEAN CLAY- Few Gravel- Brown- Hard (CL)	894.0

END OF BORING AT 20.0 FEET.

SAMPLE TYPE/NO. INTERVAL	RECOVERY LENGTH (INCHES)	SPT BLOWS PER SIX INCHES	N_{60}	MOISTURE (%) MC	HAND PENE. SHEAR STRENGTH (KSF)	REMARKS
SB1	14	1 3 3 5	8	23	4.5+	
SB2	14	3 6 7 10	18	13	4.5+	
SB3	18	10 15 15 17	42	13	4.5+	
SB4	16	9 11 9 11	28	14	4.5+	
SB5	14	13 13 20	46	10	4.5+	
SB6	16	16 12 18	42	11	4.5+	



GROUNDWATER & BACKFILL INFORMATION

GROUNDWATER WAS NOT ENCOUNTERED

	DEPTH (FT)	ELEV (FT)
CAVE-IN OF BOREHOLE AT:	8.5	905.5

BACKFILL METHOD: Auger Cuttings

NOTES:
1. The indicated stratification lines are approximate. The in-situ transitions between materials may be gradual.
2. The colors depicted on the symbolic profile are solely for visualization purposes and do not necessarily represent the in-situ colors encountered.

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BORING B 5

PAGE 1 OF 1

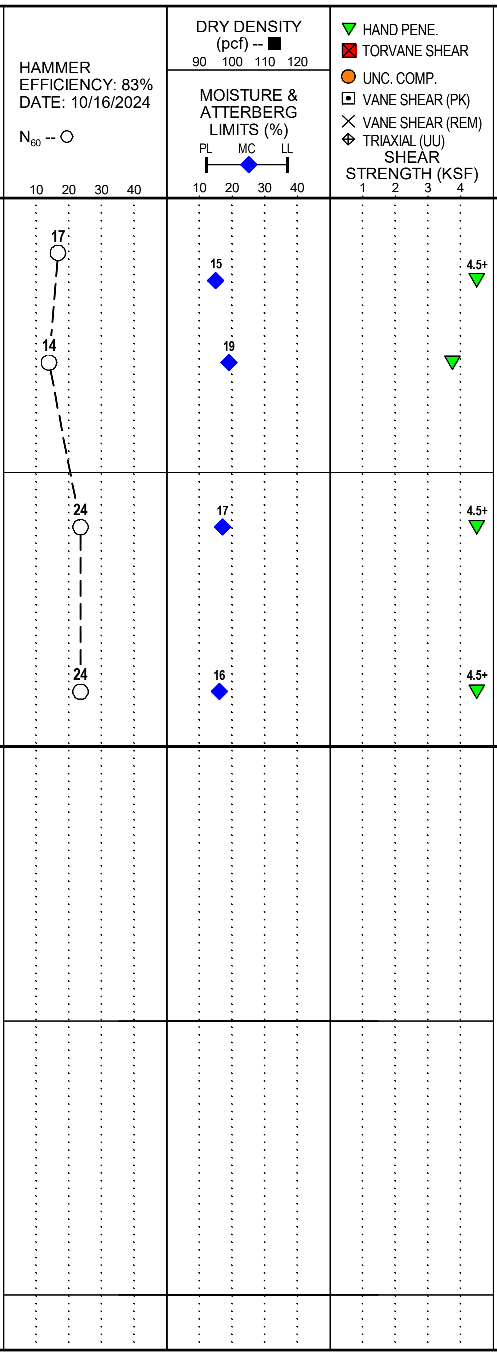
BORING DEPTH: 10 FEET

PROJECT NAME: Upland WWTP Improvements
PROJECT NUMBER: 101022.00
CLIENT: Commonwealth Engineers, Inc.
PROJECT LOCATION: Upland, Indiana
DATE STARTED: 10/24/25
COMPLETED: 10/24/25
BORING METHOD: Hollow Stem Augers
DRILLER: CS/MC
RIG NO.: 558 Mobile B-29 - Truck
LOGGED BY: AJS
CHECKED BY: JMB

ELEVATION: 915± FT

ELEVATION (FEET)	DEPTH (FEET)	PROFILE DESCRIPTION		
915	0	12 inches of TOPSOIL	1.0	914.0
		Sandy LEAN CLAY- Occasional Roots- Brown- Stiff to Hard (CL)	5.0	910.0
910	5	LEAN CLAY- Brown and Gray- Very Stiff to Hard (CL)	10.0	905.0
905	10	END OF BORING AT 10.0 FEET.		
900	15			
895	20			

SAMPLE TYPE/NO. INTERVAL	RECOVERY LENGTH (INCHES)	SPT BLOWS PER SIX INCHES	N_{60}	MC (%)	HAND PENE. (KSF)
SB1	16	4 6 6 7	17	15	4.5+
SB2	14	4 5 5 6	14	19	
SB3	16	6 8 9 10	24	17	4.5+
SB4	16	8 9 8 10	24	16	4.5+



REMARKS

GROUNDWATER & BACKFILL INFORMATION

GROUNDWATER WAS NOT ENCOUNTERED

	DEPTH (FT)	ELEV (FT)
CAVE-IN OF BOREHOLE AT:	5.0	910.0

BACKFILL METHOD: Auger Cuttings

NOTES:
1. The indicated stratification lines are approximate. The in-situ transitions between materials may be gradual.
2. The colors depicted on the symbolic profile are solely for visualization purposes and do not necessarily represent the in-situ colors encountered.

11/17/25 10:19:25 AM

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BORING B 6

PAGE 1 OF 1

BORING DEPTH: 10 FEET

PROJECT NAME: Upland WWTP Improvements
PROJECT NUMBER: 101022.00
CLIENT: Commonwealth Engineers, Inc.
PROJECT LOCATION: Upland, Indiana
DATE STARTED: 10/24/25
COMPLETED: 10/24/25
BORING METHOD: Hollow Stem Augers
DRILLER: CS/MC
RIG NO.: 558 Mobile B-29 - Truck
LOGGED BY: AJS
CHECKED BY: JMB

ELEVATION: 910± FT

HAMMER EFFICIENCY: 83%
DATE: 10/16/2024

DEPTH (FEET)	ELEVATION (FEET)	PROFILE DESCRIPTION	SAMPLE TYPE/NO. INTERVAL	RECOVERY LENGTH (INCHES)	SPT BLOWS PER SIX INCHES	N_{60} -- ○	MOISTURE & ATTERBERG LIMITS (%) MC	SHEAR STRENGTH (KSF) HAND PENE.	REMARKS
0.7	909.3	8 inches of TOPSOIL							
2.0	908.0	Sandy LEAN CLAY- Occasional Roots- Brown- Stiff to Hard (CL)	SB1	14	2 3 4 4	10	21	4.5+	
		LEAN CLAY- Trace Roots- Brown- Stiff to Hard (CL)	SB2	16	4 5 6 6	15	15	4.5+	
8.0	902.0		SB3	16	4 6 5 8	15	17	4.5+	
10.0	900.0	LEAN CLAY- Occasional Silt Seams- Brown- Very Stiff to Hard (CL)	SB4	16	3 6 8 11	19	20	4.5+	

END OF BORING AT 10.0 FEET.

DRY DENSITY (pcf) -- ■ 90 100 110 120

PL MC LL

▼ HAND PENE.
☒ TORVANE SHEAR
● UNC. COMP.
⊡ VANE SHEAR (PK)
× VANE SHEAR (REM)
⊕ TRIAXIAL (UU)

GROUNDWATER & BACKFILL INFORMATION

GROUNDWATER WAS NOT ENCOUNTERED

	DEPTH (FT)	ELEV (FT)
CAVE-IN OF BOREHOLE AT:	3.5	906.5

BACKFILL METHOD: Auger Cuttings

NOTES: 1. The indicated stratification lines are approximate. The in-situ transitions between materials may be gradual.
2. The colors depicted on the symbolic profile are solely for visualization purposes and do not necessarily represent the in-situ colors encountered.

APPENDIX B

FIELD TESTING PROCEDURES
LABORATORY TESTING PROCEDURES
LIMITATIONS PERTAINING TO SUBSURFACE CONDITIONS

FIELD TESTING PROCEDURES

STANDARD PENETRATION TESTS

The Standard Penetration Tests (SPT) generally follow the American Standard Test Method (ASTM) D-1586 “Standard Test Method for SPT and Split-Barrel Sampling of Soils”. It is typically performed using a 2-inch O.D. split-spoon sampler, which is driven to obtain samples at selected intervals. The number of blows of a 140-pound hammer dropping 30 inches is recorded for each of three or four, 6-inch penetration intervals for an 18 or 24-inch drive at each sample location. The sum of blow counts for the second and third 6-inch penetration intervals equals the raw (uncorrected) N-value for a given sample interval (i.e. 5-4-2, N = 6). We periodically calibrate SME’s drill rig auto-hammers. The hammer efficiency determined from this calibration is used to calculate the corrected N-values (N_{60}), as reported on the logs. When sampling in rock or hard soil, where a penetration of 6 inches or less was obtained for 50 hammer blows, the actual blow count and depth of penetration in inches for that interval is recorded (i.e. 50/2”). When sampling in very loose or very soft soil, where a penetration of more than 6 inches is obtained for a single hammer blow, the actual depth of penetration for that hammer blow is recorded (i.e. 1-0-0). Where the sampling equipment advanced under its own weight, “WOH” (weight of hammer) and the corresponding penetration depth are shown on the boring logs.

INFILTRATION TESTS

In-situ infiltration tests generally follow the double-ring infiltrometer field test procedures outlined in Appendix E in the Low Impact Development (LID) Manual for Michigan (dated 2008) prepared by the Southeast Michigan Council of Governments (SEMCOG). This procedure is also referenced in the States of Indiana, Ohio, and Kentucky applicable local manuals. The double-ring infiltrometer field test set-up consists of performing a boring or test pit to the test depth, installing an outer 6-inch-diameter standpipe and an inner 4-inch-diameter standpipe, and then driving the standpipes a suitable distance, per the referenced manual, below the bottom of the test depth. Soil is pre-soaked with approximately 12 inches of water for approximately one hour. The water drop rate per the last 30 minutes of the soaking period determines the subsequent interval for infiltration readings (i.e., 10- or 30-minute intervals). After completing the soaking period, standpipes are filled with water to a height of approximately 12 inches above the test depth, and water level changes in the standpipes are measured with a water level measuring tape with markings every 0.01 feet and recorded after the time intervals. This procedure is repeated until a minimum of four consecutive height changes within 1/4-inch of one another are measured. The height drop that occurred during the final time interval or the average stabilized rate is used to calculate the infiltration rate. After completion of the double-ring infiltrometer field test, the standpipes are removed, and the test hole is backfilled.

DYNAMIC CONE PENETROMETER (DCP)

USACE TESTS

Dynamic Cone Penetrometer (DCP) testing designed by the U.S. Army Corps of Engineers (USACE) is conducted to estimate the California Bearing Ratio (CBR) of the subgrade materials and existing pavement sub-layers. The USACE DCP consists of a 5/8-inch-diameter steel rod with a steel cone attached to one end driven by a sliding dual mass hammer. The rate of penetration per blow is measured at selected penetration, or hammer drop, intervals. CBR is an index commonly used in pavement design that provides an indication of subgrade support characteristics. The Corps of Engineers developed relationships to estimate the CBR value from the results of the USACE DCP test.

SME DCP TESTS

SME Dynamic Cone Penetrometer (DCP) test consists of dropping a 10-pound slide hammer that falls 24 inches and drives a rod with a 1-1/8-inch conical tip into the subgrade. The number of hammer drops required to drive the cone penetrometer are recorded for each six-inch increment and are used to estimate the relative density of the granular soils. The DCP blow counts were used to estimate Standard Penetration Test resistances (N-values) commonly used in geotechnical evaluations, based on empirical correlations developed by SME.

PRESSUREMETER TESTING

Pressuremeter testing in the field models the static loading characteristics of the soil and the resulting analyses based on pressuremeter test results are considered to be a more accurate indicator of the ultimate foundation bearing pressure, and associated settlement, than analyses using empirical correlations based on dynamic test methods, such as the Standard Penetration Test (SPT) and/or dynamic cone penetrometer (DCP) tests. Results of the SPT and/or DCP were recorded, at the pressuremeter test locations and depths to provide additional information on the relative density of the in-place subgrade, and to correlate the pressuremeter test results with data obtained at other site locations. The pressuremeter test depths were selected to provide representative information corresponding to the bearing soils anticipated within the stress influence zone of the proposed footings at the design bearing levels.

In the pressuremeter test, a radial expandable cylindrical probe is inserted into a prepared borehole at the selected testing depth. After obtaining N-values from driving a standard 2-inch O.D. diameter split-spoon sampler, borehole preparation consisted of then driving a 3-inch O.D. split-spoon sampler (or using a roller bit with wash rotary methods) to develop the appropriately sized borehole diameter for the pressuremeter probe. The cylindrical probe was inserted into the borehole to the sampling depth and then expanded against the sides of the borehole by pressurizing fluid within the system using a hydraulic screw-jack console positioned at the ground surface.

Simultaneous measurements of pressure and volume change within the probe were observed at the pressuremeter console and recorded. The pressure was incrementally increased until the maximum probe volume was reached, or until significant creep deformation (soil failure) was observed.

MUCK PROBE

The muck probe consists of a smooth rod about 1/2-inch in diameter manually pushed into the subgrade until encountering significant resistance (determined “by feel”), presumably indicating the bottom of organic soil stratum.

VANE SHEAR TESTING

In-situ vane shear testing generally follows the American Standard Test Method (ASTM) D-2573 “Standard Test Method for Field Vane Shear Test in Cohesive Soil”. Per the ASTM, the field vane shear test consists of placing a four-bladed vane (sized based on the expected cohesive soil strength) in the undisturbed soil and rotating it from the surface at a constant rate to determine the torque required to shear a cylindrical surface with the vane. This torque, or moment, is then converted to the unit shearing resistance of the failure surface by limit equilibrium analysis. Friction of the vane rod and instrument is either minimized during readings by an open hole, casing, or accounted for and subtracted from the total torque to determine the torque applied to the vane. After initially shearing the soil to determine the peak “undisturbed” ultimate shear strength, the test can be repeated to determine the remolded “residual” ultimate shear strength. The ratio of the peak shear strength divided by the remolded shear strength equals the degree of sensitivity.

DEGREE OF SENSITIVITY

DEGREE OF SENSITIVITY	DESCRIPTION
2	Insensitive
4	Moderately Sensitive
8	Extra Sensitive

ELECTRICAL RESISTIVITY TESTING

Field or laboratory resistivity testing generally follows the American Standard Test Method (ASTM) G-57 “Standard Test Method for Measurement of Soil Resistivity Using the Four-Electrode Method”.

FIELD TESTING

Per the ASTM, the field Wenner four-electrode method requires four metal electrode probes placed with equal separation at various distances in a straight line. The probes are inserted in the surface of the soil to a depth not exceeding 5 percent of the minimum separation of the electrodes (or 12 inches maximum, whichever is less). The electrode

separation is selected with consideration of the soil strata location and depth of interest. A voltage is impressed between the outer electrodes and the voltage drop between the inner electrodes is measured. The resulting resistivity measurement represents the average resistivity of a hemisphere of soil of a radius equal to the electrode separation.

LABORATORY TESTING

Soil is tamped into a soil box to resemble the compaction where the soil sample was taken until the soil is level with the top of the box. Two brass pins are inserted at the premanufactured distances into the soil sample with two endpins connected to the box. The four test leads are connected to the soil box. A voltage is impressed between the two endpins to measure the resistance, and soil resistivity is calculated based on the product-specified conversion.

CONE PENETRATION TESTING

Cone Penetration Tests (CPT) measures the soil resistance to the penetration of a standard 10 square centimeter (cm) projected area. The cone is hydraulically pushed into the soil at approximately a 2 cm per second rate. Soil resistance is recorded in kilograms per square cm at 20 cm depth intervals. Soil friction values are measured by a friction sleeve at each test interval.

LABORATORY TESTING PROCEDURES

VISUAL ENGINEERING CLASSIFICATION

Visual classification was performed on recovered samples. The appended General Notes and Unified Soil Classification System (USCS) sheets include a brief summary of the general method used visually classify the soil and assign an appropriate USCS group symbol. The estimated group symbol, according to the USCS, is shown in parentheses following the textural description of the various strata on the boring logs appended to this report. The soil descriptions developed from visual classifications are sometimes modified to reflect the results of laboratory testing.

MOISTURE CONTENT

Moisture content tests were performed by weighing samples from the field at their in-situ moisture condition. These samples were then dried at a constant temperature (approximately 110° C) overnight in an oven. After drying, the samples were weighed to determine the dry weight of the sample and the weight of the water that was expelled during drying. The moisture content of the specimen is expressed as a percent and is the weight of the water compared to the dry weight of the specimen.

HAND PENETROMETER TESTS

In the hand penetrometer test, the unconfined compressive strength of a cohesive soil sample is estimated by measuring the resistance of the sample to the penetration of a small calibrated, spring-loaded cylinder. The maximum capacity of the penetrometer is 4.5 tons per square-foot (tsf). Theoretically, the undrained shear strength of the cohesive sample is one-half the unconfined compressive strength. The undrained shear strength (based on the hand penetrometer test) presented on the boring logs is reported in units of kips per square-foot (ksf).

TORVANE SHEAR TESTS

In the Torvane test, the shear strength of a low strength, cohesive soil sample is estimated by measuring the resistance of the sample to a torque applied through vanes inserted into the sample. The undrained shear strength of the samples is measured from the maximum torque required to shear the sample and is reported in units of kips per square-foot (ksf).

LOSS-ON-IGNITION (ORGANIC CONTENT) TESTS

Loss-on-ignition (LOI) tests are conducted by first weighing the sample and then heating the sample to dry the moisture from the sample (in the same manner as determining the moisture content of the soil). The sample is then re-weighed to determine the dry weight and then heated for four hours in a muffle furnace at a high temperature (approximately 440° C). After cooling, the sample is re-weighed to calculate the amount of ash remaining, which in turn is used to determine the amount of organic matter burned from the original dry sample. The organic matter content of the specimen is expressed as a percent compared to the dry weight of the sample.

ATTERBERG LIMITS TESTS

Atterberg limits tests consist of two components. The plastic limit of a cohesive sample is determined by rolling the sample into a thread and the plastic limit is the moisture content where a 1/8-inch thread begins to crumble. The liquid limit is determined by placing a 1/2-inch-thick soil pat into the liquid limits cup and using a grooving tool to divide the soil pat in half. The cup is then tapped on the base of the liquid limits device using a crank handle. The number of drops of the cup to close the gap formed by the grooving tool 1/2 inch is recorded along with the corresponding moisture content of the sample. This procedure is repeated several times at different moisture contents and a graph of moisture content, and the corresponding number of blows is plotted. The liquid limit is defined as the moisture content at a nominal 25 drops of the cup. From this test, the plasticity index can be determined by subtracting the plastic limit from the liquid limit.

LIMITATIONS PERTAINING TO SUBSURFACE CONDITIONS

EXISTING FILL

It is sometimes difficult to distinguish between existing fill present at a site and natural soils based on samples and cuttings from small-diameter boreholes, especially if portions of the fill do not contain man-made materials, debris, topsoil, or organic layers, and when the fill appears similar in composition to the local natural soils. Therefore, consider the delineation of fill described on the logs, if encountered, to be approximate.

The composition of existing site fill, if encountered, may change abruptly over short distances and will vary from what is reported on the logs. The descriptions of debris within fill may not accurately indicate the quantity, composition, or size of the debris, and may not fully include the types of debris existing within the site fills. Perform test pits if existing fill is encountered to further evaluate the condition of the fill, particularly if the fill will be utilized for support of overlying structures and/or other improvements.

GROUNDWATER

Hydrostatic groundwater levels, and perched groundwater conditions, will fluctuate throughout the year, based on variations in precipitation, evaporation, run-off, and other factors. The groundwater information reported on the logs represent conditions at the time the readings were taken and may vary from the groundwater conditions encountered at other times.

SUBSURFACE PROFILE

The profile described in this report and included on the logs is a generalized description of the conditions observed. The stratification depths described in this report and shown on the logs indicate a zone of transition from one soil type to another. They are not meant to delineate exact depths of change between soil types. Soil conditions may vary between or away from the exploration locations. Refer to the logs for the soil descriptions, rock descriptions (when applicable), and results of the field and laboratory tests at the specific exploration locations.

If only borings with hollow-stem or solid-stem augers are performed, consider thickness measurements of surficial materials reported on the logs (e.g., gravel, asphalt, concrete, aggregate base) to be approximate since mixing of the surface materials with the underlying subgrade can occur while advancing the augers, and it is difficult to measure the thickness of surface materials in small-diameter boreholes. Perform additional evaluations for more accurate measurements of surface materials, such as test pits for topsoil and gravel thicknesses, coring for pavement thicknesses, and hand sampling for aggregate thickness.

RADON

The need for radon control systems for this project was not evaluated as part of our current scope of services. Contact the local building authority to verify whether radon control systems are necessary to meet the applicable local building codes or other requirements. If radon control methods are required, incorporate the recommendations regarding the below-slab leveling course materials and vapor retarders presented in this report with the specific materials and measures necessary to meet the applicable radon control methods. Contact SME for further recommendations.

APPENDIX C

IMPORTANT INFORMATION ABOUT THIS GEOTECHNICAL-ENGINEERING REPORT

GENERAL COMMENTS

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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GENERAL COMMENTS

BASIS OF GEOTECHNICAL REPORT

This report has been prepared in accordance with generally accepted geotechnical engineering practices to assist in the design and/or evaluation of this project. If the project plans, design criteria, and/or other project information referenced in this report and utilized by SME to prepare our recommendations are changed, the conclusions and recommendations contained in this report are not considered valid unless the changes are reviewed, and the conclusions and recommendations of this report are modified or approved in writing.

The discussions and recommendations contained in this report are based on the available project information, described in this report, and the geotechnical data obtained from the field exploration at the locations indicated in the report. Variations in soil and groundwater conditions commonly occur between or away from sampling locations. The nature and extent of the variations may not become evident until the time of construction. If significant variations are observed during construction, SME must be contacted to reevaluate the recommendations of this report.

In the process of obtaining and testing samples and preparing this report, procedures are followed that represent reasonable and accepted practice in the field of geotechnical engineering. Specifically, field logs are prepared during the field exploration that describe field occurrences, sampling locations, and other information. Samples obtained in the field are frequently subjected to additional testing and reclassification in the laboratory and differences may exist between the field logs and the report logs.

The engineer preparing the report reviews the field logs, laboratory classifications, and test data and then prepares the report logs. Our recommendations are based on the contents of the report logs and the information contained therein.

REVIEW OF DESIGN DETAILS, PLANS, AND SPECIFICATIONS

Retain SME to review the design details, project plans, and specifications to verify those documents are consistent with the recommendations contained in this report.

REVIEW OF REPORT INFORMATION WITH PROJECT TEAM

Implementation of our recommendations may affect the design, construction, and performance of the proposed improvements, along with the potential inherent risks involved with the proposed construction. The client and key members of the design team, including SME, should discuss the issues covered in this report so the issues are understood and applied in a manner consistent with the owner’s budget, tolerance of risk, and expectations for performance and maintenance.

FIELD VERIFICATION OF GEOTECHNICAL CONDITIONS

SME needs to be retained to continue our services through construction so we may observe and evaluate the actual subsurface conditions relative to the recommendations made in this report, and so we can verify the recommendations of this report are properly implemented during construction. This may avoid misinterpretation of our recommendations by other parties and will allow us to review and modify our recommendations if variations in the site subsurface conditions are encountered.

PROJECT INFORMATION FOR CONTRACTOR

This report and any future addenda or other reports regarding this site needs to be made available to prospective contractors prior to submitting their proposals for their information only and to supply them with facts relative to the subsurface evaluation and laboratory test results. If the selected contractor encounters subsurface conditions during construction, which differ from those presented in this report, the contractor needs to promptly describe the nature and extent of the differing conditions in writing and SME needs to be notified so we can verify those conditions. The construction contract needs to include provisions for dealing with differing conditions, and contingency funds for potential problems during earthwork and foundation construction. We would be pleased to assist with the development of contract provisions based on our experience.

The contractor needs to be prepared to handle environmental conditions encountered at this site, which may affect the excavation, removal, or disposal of soil; dewatering of excavations; and health and safety of workers. Any Environmental Assessment reports prepared for this site need to be made available for review by bidders and the successful contractor.

THIRD PARTY RELIANCE/REUSE OF THIS REPORT

This report has been prepared solely for the use of our Client for the project specifically described in this report. This report cannot be relied upon by other parties not involved in the project, unless specifically allowed by SME in writing. SME also is not responsible for the interpretation by other parties of the geotechnical data and the recommendations provided herein.

Passionate People Building
and Revitalizing our World

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