ECS Midwest, LLC

Geotechnical Engineering Report

Proposed Self-Supporting Monopole Tower
Monopole Tower Site (N44.5266° W88.0975°)

909 Packerland Drive
Green Bay, Brown County, Wisconsin

ECS Project No. 59:3567

August 16, 2023

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Ms. Sara Bushie
Mission Support Services, LLC
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ECS Project No. 59:3567

Reference: Report of Subsurface Exploration and Geotechnical Engineering
Proposed Self-Supporting Monopole Tower
909 Packerland Drive
Green Bay, Brown County, Wisconsin

Dear Ms. Bushie:

ECS Midwest, LLC (ECS) has completed the subsurface exploration, laboratory testing, and geotechnical engineering analyses for the above-referenced project. Our services were performed in general accordance with our agreed to scope of services. This report presents our understanding of the geotechnical aspects of the project along with the results of the field exploration and laboratory testing conducted, and our design and construction recommendations.

It has been our pleasure to be of service to you during the design phase of this project. We would appreciate the opportunity to remain involved during the continuation of the design phase, and we would like to provide our services during construction phase operations to verify subsurface conditions assumed for this report. Should you have questions concerning the information contained in this report, or if we can be of further assistance to you, please contact us.

Respectfully submitted,

ECS Midwest, LLC

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EXECUTIVE SUMMARY

The following summarizes the main findings of the exploration, particularly those that may have a cost impact on the planned development. Further, we summarized our principal foundation recommendations. Information gleaned from the Executive Summary should not be utilized in lieu of reading the entire geotechnical report.

- Possible fill soils were observed in both soil borings, which extended to depths of about 2 and 4 feet below the existing grades.
- Given the depth of possible fill soils and no compaction test results data, there is higher than typical risk of total and differential settlement of grade supported structures bearing on these possible fill soils.
- The proposed self-supporting tower can be supported on a mat foundation bearing on competent natural soils at a depth of about 8 feet below the existing grade, which may be designed for a maximum net allowable soil bearing pressure of 6,000 psf (pounds per square foot). Competent natural soils can be identified on the test boring logs as glacial outwash, having Standard Penetration Test (SPT) N-values of at least 25 bpf (blows per foot). Alternatively, the proposed self-supported tower could be supported by deep foundation such as drilled shaft bearing within the medium dense to dense glacial outwash or dense to very dense silt soils (Lacustrine) at a minimum depth of about 20 feet below the existing grade.
- The proposed miscellaneous grade supported small equipment (electrical vault, generator, cabinet etc.) can be supported on structural fill after removal and replacement of the possible fill.
- Two (2) design alternatives for preparation of the subgrade below pavements with varying anticipated cost and level of risk are provided in section 5.1.2 of this report.

1.0 INTRODUCTION

ECS prepared this report for the purpose of providing the results of our subsurface exploration and laboratory testing, site characterization, engineering analysis, and geotechnical recommendations for the design and construction of self-supported monopole tower and equipment building foundations. The recommendations developed for this report are based on project information supplied by Ms. Sara Bushie with Mission Support Services, LLC.

ECS provided geotechnical engineering services in accordance with ECS Proposal No. 59:4608-GP, dated October 17, 2022, and authorized by Mission Support Services, LLC Purchase Order dated June 01, 2023.

This report contains the procedures and results of our subsurface exploration and laboratory testing programs, review of existing site conditions, engineering analyses, and recommendations for the design and construction of the proposed infrastructure. The report includes the following items:

- A brief review and description of our field and laboratory test procedures and results.
- A review of the observed surface topographical features and site conditions.
- A review of area and site geologic conditions.
- A review of subsurface soil stratigraphy with pertinent available physical properties.
- Final test boring logs.
- Shallow (mat/skid/slab) and deep foundation recommendations for the proposed monopole tower and grade supported equipment pads, applicable soil bearing pressures and anticipated settlements.
- Recommendations for uplift resistance, lateral earth pressures, sliding resistance coefficients, drainage, and foundation backfill.
- Seismic Site Classification in accordance with applicable International Building Code based on the SPT 'N' blow counts and provide applicable seismic site coefficients (no liquefactions analysis).
- Pavement section (unsurfaced) recommendations based on anticipated traffic loading information.
- Soil electrical resistivity test results.
- Evaluation and recommendations relative to groundwater control.
- Recommendations for site preparation and construction of compacted fills, including an evaluation of on-site soils for use as compacted fills and identification of potentially unsuitable soils and/or soils exhibiting excessive moisture at the time of sampling.
- Recommendations for additional testing and/or consultation that might be required to complete the geotechnical assessment and related geotechnical engineering for this project.

2.0 PROJECT INFORMATION

2.1 PROJECT LOCATION/CURRENT SITE USE/PAST SITE USE

The project site is located at 909 Packerland Drive in Green Bay, Wisconsin. Specifically, the proposed monopole tower will be located immediately north of the existing building. The site location is shown in the figure below and on the Site Location Diagram in Appendix A of this report:

Site Location (approximately outlined in red)

Based on the historical aerials from the County GIS website dated back 1930's, the site was being used as an agricultural land until late-1960's or early-1970's. During site development in mid-1970's, the site was regraded and developed with the existing building and pavement areas constructed during mid to late-1980's.

Site-specific topographic information was not available at this time. However, based on publicly available information such as *Google Earth and County GIS,* the site grades range from about EL. 699 to EL. 704 feet MSL (above Mean Sea Level) within the proposed development areas.

2.2 PROPOSED CONSTRUCTION

Based on the provided information and Preliminary Site Plan, the information listed in the Table below summarizes our understanding and assumptions of the structures and its loads:

DESIGN INFORMATION		
Subject	**Design Characteristic**	
Self-supported Monopole Tower	The proposed monopole tower is anticipated to be 150 feet in height. The proposed tower is anticipated to be supported on reinforced concrete mat foundation or drilled shaft.	R/B
Tower Loads	Maximum Compression: 75 kips Maximum Uplift: 65 kips Maximum Shear: 25 kips	B
Misc. Grade Supported Structures	Miscellaneous small equipment/structures are anticipated to be supported on slab foundations bearing at a depth of about 6 to 12 inches below adjacent final grades.	R/B
Settlement Tolerance	Estimated to be 1-inch total and ¾-inch differential	B
Grading Operations	Less than 2 feet	B
Pavement	Unsurfaced/crushed aggregate access road	B

R: Reported by client and/or Design Team
B: Based on ECS estimate in the absence of information from the Client and/or Design Team

Where the borings encountered subsurface conditions that might be detrimental to the support of the proposed construction, ECS anticipates the owner will have an acceptable risk level if the detrimental material remains in place. This report anticipates the owner would only be willing to accept a low risk for foundation settlement less than 1 inch and a moderate risk for reduced pavement performance.

If ECS' understanding of the project or the owner's anticipated acceptable risk level are not correct or the design changes, then please contact ECS so that we may review these changes and revise our recommendations, as appropriate.

3.0 FIELD EXPLORATION AND LABORATORY TESTING

Our exploration procedures are explained in greater detail in Appendix B including the insert titled "Subsurface Exploration Procedures." Our scope of services included drilling two (2) Standard Penetration Test (SPT) soil borings, extending to depths of approximately 15 to 30 feet below existing grade. ECS located the soil borings using conventional measuring techniques referenced from existing site features and the approximate locations are shown on the Boring Location Diagram in Appendix A.

The ground surface elevation at the boring locations were not surveyed by a licensed surveyor and ECS estimated the surface elevations from *Google Earth*. Ground surface elevations and boring locations determined without professional survey are approximate and may not be appropriate for final design.

ECS utilized a truck-mounted drilling rig equipped with continuous flight, hollow stem augers. The soil borings were extended to depths of about 15 to 30 feet below the existing grades. Standard penetration tests (SPTs) were conducted in the borings at regular intervals in general accordance with ASTM D1586. The drill crew backfilled the boreholes upon completion of the drilling operations. Settlement of borehole backfill can occur over time resulting in a trip hazard.

Monitoring the boreholes after initial drilling activities is not within our scope but should be done by the client or property owner.

3.1 SUBSURFACE CHARACTERIZATION

The surficial deposits of Brown County, which are up to 30 m in thickness, include seven lithologically distinct till units, silty and clayey offshore lacustrine sediment of several ages, and meltwater and stream sediment of several ages. Except for a small area of till that was deposited by the Lake Michigan Lobe in the southeastern corner of the county, the sediments in Brown County were deposited by the Green Bay Lobe and in the lakes and streams associated with it. Modern stream sediment, deposits of windblown sand, and accumulations of organic sediment are also present. The bottommost formation consisted of Galena Formation bedrock, consisting of fine to medium grained dolostone. Well construction records for wells installed near the site indicate bedrock to be at a depth of between 100 and 130 feet.

According to the Soil Survey from the USDA - Natural Resources Conservation Service (websoilsurvey.nrcs.usda.gov), which provides soil information to a shallow depth (generally less than 5 feet), the near surface soils are mapped as Boyer Loamy fine Sand (BrB). Soil survey mapping of the site vicinity is presented in Appendix A of this report. The parent material for these soils consists of Sandy drift over loamy drift over stratified sandy and gravelly outwash, that are generally well drained, classified as being in Hydrologic Soil Group A, and have a low to moderate potential for frost action.

The encountered subsurface conditions in the soil borings appeared to match published geological mapping except for the possible fill. For subsurface information at a specific test boring location, refer to the boring logs in Appendix B. The following sections provide generalized characterizations of the soil strata:

GENERALIZED SOIL PROFILE CHARACTERISTICS			
Approximate Depth Range(ft)	Stratum	Description	Range of SPT(1) N-values (bpf)
0 to 0.25 Surface cover	N/A	Topsoil: Approximately 3 inches	N/A
0.25 to 4	I²	Possible Fill: loose to medium dense, SILTY SAND (SM)	6 to 14
4 to 18½	II	Glacial Outwash: medium dens to very dense, SILTY SAND (SM), SILTY CLAYEY SAND (SC-SM)	16 to 55
18½ to 30 (End of Boring)	III	Lacustrine: dense to very dense, SILT (ML)	40 to 65

Notes:
(1) Standard Penetration Testing
(2) Encountered in soil borings B-01 and B-02.

Where the drill crew used discontinuous material sampling intervals at the test borings, ECS inferred conditions between sample intervals. The soil stratification shown on the boring logs represents the interpreted soil conditions at the actual boring locations. Variations in the stratification can occur between sample intervals and boring locations. The subsurface conditions at other times and locations on the site may differ from those found at the boring locations. If different site conditions are encountered during construction, ECS should be contacted to review our recommendations relative to the new information.

Because of the limitations of the split-spoon sampler, which has a 1⅜-inch inside diameter, the soil classifications noted on the boring logs may not be representative of the entire soil matrix. Materials larger than the 1⅜-inch inside diameter of the split-spoon sampler cannot be collected and observed directly. Where possible, the drill crew noted the estimated depth of larger diameter materials, such as cobbles and boulders, based on things such as changes in the observed drilling resistance and auger cuttings.

3.2 GROUNDWATER OBSERVATIONS

The drill crew observed the boreholes for a measurable groundwater level during sampling and at the completion of drilling. Measurable groundwater was observed at a depth of approximately 8.8 feet below the existing grade at Boring B-02 during drilling. Groundwater was observed at a depth of 10.6 feet in Boring B-01 immediately upon completion of the soil borings.

Based upon our interpretation of the subsurface data, including water level measurements, the borings likely didn't encounter saturated (water table) aquifer. Perched groundwater is distinguished differently from the water table aquifer as defined below:

> "Perched water is typically of limited quantity, replenished or recharged very slowly. When encountered in an excavation, perched water will typically drain off very quickly, with limited continuous flow or bleeding, unless a source of recharge, such as a leaking utility is present."
>
> From: *Construction Dewatering and Groundwater Control – New Methods and Applications, 3rd Addition*

A water table aquifer is distinguished from a perched groundwater table based on the recharge ability of the water table aquifer, which may be limitless but can be lowered temporarily through adequate dewatering techniques such as deep wells and well points. Perched groundwater is often alleviated in excavations by pumping from sump pits and French drains.

The highest groundwater observations are normally encountered in late winter and early spring and our current groundwater observations likely differ from the seasonal maximum water table. In addition, variations in both groundwater types (perched and groundwater table aquifer) can occur because of seasonal variations in precipitation, evaporation, surface water runoff, lateral drainage conditions, construction activities, and other factors. The time of year and the weather history during the advancement of the borings should be considered when estimating groundwater levels at other points in time.

3.3 IN-SITU ELECTRICAL RESISTIVITY

Electrical resistivity (ER) testing was performed using a Wenner four-electrode array and followed *ASTM G57 Standard Test Method for Measurement of Soil Resistivity Using the Wenner Four-Electrode Method* guidelines. The ER testing was performed using electrode spacings of 2½± feet, 5± feet, 10± feet, and 20± feet orientated in both east-west and north-south directions at the proposed tower location at the project site. The electrodes were installed by hammering approximately 18-inch-long stainless-steel stakes approximately 12 inches into the ground at each electrode location. The ER survey was conducted using a Fluke 1625 GEO Earth Ground Tester with 4 electrodes using accepted geophysical industry procedures. The resistivity data was recorded in the field and post-processed in our office. Results are included on the data sheets enclosed in Appendix B.

3.4 LABORATORY SERVICES

The laboratory services performed by ECS for this project included select tests performed on samples retained from the field exploration operations. Classification and index property tests were performed on representative soil samples obtained from the test borings to aid classification of the soils and to estimate engineering properties. The following paragraphs briefly describe the completed laboratory services program.

- A geotechnical engineered classified each soil sample retained from the test borings on the basis of texture and plasticity using the Unified Soil Classification System (USCS) and ASTM D-2488 (Description and Identification of Soils-Visual/Manual Procedures) as a guide. After classification, the geotechnical engineer grouped the various soil types into the major zones noted on the boring logs in Appendix B. The group symbols for each soil type are indicated in parentheses along with the soil descriptions on the boring logs. The stratification lines designating the interfaces between earth materials on the boring logs are approximate; in situ, the transitions may be gradual.
- Percent passing US Standard Sieve No. 200 on select soil samples in general accordance with ASTM D 1140.

The soil samples will be retained in our laboratory for a period of 60 days, after which, they will be discarded, unless other instructions are received as to their disposal.

4.0 DESIGN RECOMMENDATIONS

4.1 FOUNDATIONS

4.1.1 Mat Foundations

The foundation analysis was conducted using the information noted in the *Proposed Construction* Section, the boring information, and anticipating that the self-supported tower is supported by a mat foundation bearing at a depth of about 8 feet below the finished grade. Based on our experience with similar projects, ECS anticipates a mat foundation size on the order of 20 feet by 20 feet for the foundation analysis. The following parameters are recommended for proposed self-supported tower mat foundation design:

<table>
<tr><th colspan="3">MAT FOUNDATION DESIGN - SELF SUPPORTED TOWER [1]</th></tr>
<tr><th colspan="2">Design Parameter</th><th>Value</th></tr>
<tr><td colspan="2">Maximum Net Allowable Soil Bearing Pressure [2]</td><td>6,000 psf</td></tr>
<tr><td colspan="2">Soil Subgrade Modulus (k_v)</td><td>25 to 50 psi/in</td></tr>
<tr><td colspan="2">Acceptable Bearing Soil Material</td><td>Stratum II (glacial outwash)</td></tr>
<tr><td colspan="2">Competent Soils Designated Adequate for the Allowable Bearing Pressure</td><td>N ≥ 25 bpf</td></tr>
<tr><td rowspan="2">Post-Construction Estimated Settlement</td><td>Total</td><td>Approximately 1 inch</td></tr>
<tr><td>Differential [3]</td><td>Approximately ½ inch</td></tr>
</table>

Notes:

1. We recommend a structural engineer provide specific foundation details including footing dimensions, reinforcing, and other details.
2. The applied pressure in excess of the surrounding overburden soils above the base of the foundation and includes a factor of safety of 3.

The modulus of subgrade reaction can be used for design and analysis of a mat foundation. The subgrade modulus, kv, is not a fundamental soil property and depends on many factors, which include the width, shape, and depth below the ground surface of the loaded area, position under the foundation, and time. The subgrade modulus is the ratio of pressure (*p*) per unit area of the surface of contact between a loaded mat and the subgrade, and the settlement (δ) produced by this load application. Based on the non-rigid methods of analyzing the deformations within the mat foundation and underling subgrade soils, a single value of subgrade modulus (k) should not be considered for design, as suggested in the rigid analysis method. In reality, the uniformly loaded mat foundation (underlain by uniform subgrade soils) is expected to settle more at the center as compared to the edges, which will form a dish shape deformation of mat and subgrade soils. Model studies (Pseudo-Coupled) indicate that reasonable results are obtained when “kv” value along the perimeter of the mat are about twice those in the center (ACI, 1993). It should be noted that the project structural engineer would be responsible for the structural design of the concrete mat. The structural stiffness of the mat will determine the distribution of the loading from each temporary/permanent structure added to the mat. Loading the mat from the middle of the mat to the outside edges in a symmetric pattern would produce a more uniform distribution to the overall distortion of the mat during operation. Conversely, loading an extreme corner or edge in an asymmetric pattern would likely produce a larger distortion to the mat during its operation.

Potential Undercuts: The soils at the foundation bearing elevation(s) are anticipated to be competent for support of the proposed tower structure. Where undocumented fill, organic soils, soft/very loose or otherwise inadequate soils are observed at the footing bearing elevations, the poor soils should be undercut and removed, and replaced with a lean concrete. We recommended ECS be retained to observe and test the foundation bearing grade as recommended in the *Foundation Observations* Section. It is also recommended backfill of foundation undercuts be done as recommended in the *Earthwork Operations* Section of this report.

Foundation Lateral Loading: Lateral load resistance will be developed by friction acting at the base of foundations, and the passive earth pressure developed by the footings below-grade. Passive pressure and sliding resistance (friction) may be used in combination, without reduction, in determining the total resistance to lateral loads. The parameters in the following table are recommended for lateral foundation loading:

FOUNDATION LATERAL LOADING RESISTANCE PARAMETERS	
Soil Parameter	**Estimated value**
Coefficient of Passive Earth Pressure (K_p)	2.75
Soil Moist Unit Weight (γ) - backfill sand	120 pcf
Interface Friction Angle [*Poured concrete on sandy soil*] (ϕ_f)	19°
Sliding Friction Coefficient [*Poured concrete on granular soil*] (μ)	0.36
Passive Equivalent Fluid Pressure [(1)]	330H (psf)

Notes:

1. Neglect the passive earth pressure on the low side of the wall within the frost zone because of loss of strength seasonally and strain required to mobilize.

The final design of the foundation for lateral loads should be based on a minimum factor of safety against sliding of 1.5 and overturning of 2. Also, if the resultant force of the maximum vertical force does not act within the middle one-third (kern) of the footing, a smaller effective bearing area is expected to occur and thereby result in a higher effective bearing pressure that should be accounted for in the design.

Where utility trenches or other excavations are located adjacent to foundations, the bottom of the footing should be located below an imaginary 1:1 (horizontal to vertical) plane projected upward from the nearest bottom edge of the utility trench.

4.1.2 Deep Foundations (Self-supported Tower)

Alternatively, the proposed self-supported tower could be supported by a system of deep foundations such as drilled shafts. The allowable axial load carrying capacity of a drilled shaft can be computed using the static method of analysis. According to this method, allowable axial capacity, $\mathbf{Q_a}$, at a given penetration is taken as the sum of the skin friction on the side of the shaft, $\mathbf{Q_{as}}$, and the end or point bearing at the shaft tip, $\mathbf{Q_{ap}}$, so that:

$$\mathbf{Q_a = Q_{as} + Q_{ap} = f\ A_s + q\ A_p}$$

where A_s and A_p represent, respectively, the embedded perimeter surface area and the end area of the shaft; f and q represent, respectively, the allowable unit skin friction and the allowable unit end or point bearing. The total allowable axial capacity in compression will be the summation of the allowable frictional capacity and the allowable end bearing capacity. The total allowable axial capacity in tension (uplift) will be the allowable frictional capacity of the shaft neglecting the end bearing component. Based on the project characteristics, the horizontal (shear) force and overturning moment will govern the design (depth and size) of the shaft. Steel reinforcing steel must be designed by the structural engineer to resist the referenced loads. The Tables below summarize the soil design parameters which should be used for uplift and lateral stability analyses based on the LPILE computer programs. Due to the disturbance during shaft installation, frost depth and wetting/drying cycles, ECS recommends that the upper 66 inches be ignored for the design of the foundation system.1

ALLOWABLE BEARING CAPACITIES IN COMPRESSION AND UPLIFT			
Soil Type	**Approximate Depth below the Existing Grade (ft)**	**Allowable Unit End Bearing (psf)**	**Allowable Unit Skin Friction (psf)**
Possible Fill/Natural: loose to medium dense, silty sand	0 to 5½	NA	400
Medium dense to very dense, silty sand, silty clayey sand	5½ to 18½	9,000	800
Dense to very dense, silt	18½ to 30	16,000	1,200

**Allowable End bearing and Skin friction values are estimated based on AASHTO LRFD Bridge Design*

LPILE SOIL PARAMETERS FOR LATERAL LOAD ANALYSIS						
Soil Type	**Depth Below the Existing Ground Surface (ft)**	**"p-y" criteria**	**Effective Unit Weight (pcf)**	**Friction Angle ϕ**	**Lateral Modulus K (pci)**	**Strain Factor $\varepsilon_{50/Krm}$**
Possible Fill/Natural: Silty sand	0 to 5½	Neglect[1]				
Silty Sand/Silty Clayey Sand	5½ to 18½	Medium Dense	125/63[2]	32	K=90/60	NA
Silt	18½ to 30	Dense	68[2]	34	K=125	NA

Note: ϕ-Friction Angle; K-modulus of subgrade reaction (pci) for sand, and $\varepsilon_{50/Krm}$ - strain corresponding 50% of the maximum stress.
[1]Neglect upper 5½ feet of soil for lateral load analyses.
[2]Buoyant Unit Weight of soil. Free ground water is estimated to be at a depth of about 8 feet below the existing grades

**Friction Angles are estimated based on AASHTO LRFD Bridge*

4.1.3 Drilled Shaft Excavation/Installation Considerations

Based on the information yielded by the borings, sandy soils are noted from surface to a depth of about 18½ feet and underlain by silt soils to the termination depth of soil boring B-01. Relatively shallow groundwater was observed in both soil borings. The procurement of a highly-qualified specialty contractor experienced in the dewatering of wet granular soils and the installation of drilled shaft foundations for this project is mandatory in order to reduce the potential for construction problems relative to cave-ins and 'running ground' conditions (ACI 304R, Chapter 8, Concrete Placed Under Water). A temporary steel casing is highly recommended to control these adverse ground issues. It is also recommended that the geotechnical engineer who has knowledge of the design, and the geologic and subsurface conditions observe the drilled shaft excavation/construction and backfilling of concrete.

Completed drilled shaft excavations should have a planar bottom for axial bearing considerations. Loose material, disturbed material and water should be removed from the drilled shaft excavation prior to concrete backfilling. Drilled shaft excavations should not be allowed to stand open for a significant length of time because this may allow water to accumulate. Based on the observed groundwater levels and expected drilled shaft depths, water is expected to be encountered during shaft excavation. The maximum depth of water at the bottom of the shaft excavation should not exceed 3 inches prior to concrete pour. Use of drilling mud, downhole dewatering, bentonite or polymer slurry and/or tremie concrete construction methods are expected because of relatively shallow groundwater was encountered at the boring locations and cannot be managed using temporary casing only. Another alternative may be to use a larger diameter shaft to help reduce drilled shaft embedment depths needed. Sidewalls are recommended to be visually free of cuttings. A clean-out bucket should be provided to permit manual removal of all loose or disturbed soils within the drilled shaft excavation.

The potential for significant caving of shaft sidewalls in granular soils exists and temporary steel casing and/or slurry will be required during drilled shaft excavation. Care must be taken while removing liners or temporary casing during concrete placement so the head of concrete inside the casing is at all times greater than the earth pressure and hydrostatic head outside the casing. The concrete mix for the drilled shaft should be designed and placed to reduce the potential for arching of the concrete during removal of the casing. The temporary steel casing should be extended a minimum 3½ feet above the ground surface for safety and to reduce the potential risk of accidental fall-in of foreign materials and personnel into the excavation hole.

Concrete should consist of a Portland cement mixture properly air-entrained with an appropriate water to cement ratio for proper strength and durability. Because of the presence of water bearing granular soils, placement of concrete by tremie method is expected to be required. ECS recommends a slump of 5 to 7 inches if tremie placement is required. Care should be exercised so the concrete will not contact the reinforcing cages during placement otherwise segregation of the concrete will occur. If the mixture is too stiff, then reinforcing cages may be pulled up when liners are withdrawn.

The drilled shaft should have a minimum shaft diameter of 30-inches to help reduce arching and the development of possible voids in placement of the in-situ concrete. The shaft should also contain reinforcing steel (designed by the structural engineer) to resist the combinations of loadings and to satisfy the applicable building codes.

4.1.4 Slab/Skid Foundations (Misc. Equipment Pads)

Possible fill soils were observed in both soil borings below the surficial soil cover. ECS anticipates that the possible fill can remain in place below lightly loaded miscellaneous equipment provided observations made during construction indicate the material is natural and not fill, it contains less than 5 percent organic content, and it meets strength requirements. . Given the available information, lightly loaded miscellaneous equipment will be supported on slab-on-grade/skid foundations that are bearing at depths ranging from approximately 6 to 12 inches below the final grades. Based on the subsurface exploration, the soils at this site are considered low to moderately frost susceptible. Slab/skid foundations bearing at a depth of approximately 6 to 12 inches below the final grades on compacted in situ granular soils or compacted structural fill soil (extending through the existing fill) can be designed for maximum net allowable soil bearing pressure of 1,000 psf. An estimated coefficient of subgrade modulus of 7 to 9 pci can be used for preliminary slab foundation design thickness and rigidity. However, if the bearing contact pressure on the supporting soil mass is greater than 1,000 psf or footings are larger than 6 feet by 6 feet, ECS anticipates excessive total and differential settlement to be in excess of 1 inch and ½ inch, respectively.

4.3 SEISMIC DESIGN CONSIDERATIONS

Seismic Site Classification: The International Building Code (IBC) 2018 requires site classification for seismic design based on the upper 100 feet of a soil profile. There are three methods to estimate the Seismic Site Class, namely Standard Penetration Resistance (N-value), undrained shear strength (Su) and shear wave velocity (v_s) methods.

SEISMIC SITE CLASSIFICATION				
Site Class	Soil Profile Name	Shear Wave Velocity, Vs, (ft./s)	N value (bpf)	Undrained Shear Strength, Su, (psf)
A	Hard Rock	Vs > 5,000 fps	N/A	NA
B	Rock	2,500 < Vs ≤ 5,000 fps	N/A	NA
C	Very dense soil and soft rock	1,200 < Vs ≤ 2,500 fps	>50	> 2,000
D	Stiff Soil Profile	600 ≤ Vs ≤ 1,200 fps	15 to 50	1,000 to 2,000
E	Soft Soil Profile	Vs < 600 fps	<15	< 1,000

ASCE 7-16 Table 20.3-1 Site Classification

The maximum explored depth in the present subsurface exploration was 30 feet below current site grades. As such, based upon our interpretation of the subsurface conditions and our experience with similar geological settings, the recommended Seismic Site Class is "D" as shown in the preceding Table.

Ground Motion Parameters: In addition to the seismic site classification, ECS has determined the design spectral response acceleration parameters following the IBC methodology (ASCE 7-16). The Mapped Reponses were estimated from the *'ATC Hazards by Location'* website (https://hazards.atcouncil.org/) for the address of the project. The design responses for the short period (0.2 sec, S_{DS}) and 1-second period (S_{D1}) are noted in bold at the far-right end of the following Table.

GROUND MOTION PARAMETERS									
Period (sec)	Mapped Spectral Response Accelerations (g)		Values of Site Coefficient for Site Class		Maximum Spectral Response Acceleration Adjusted for Site Class (g)		Design Spectral Response Acceleration (g)		
0.2	S_S	0.050	F_a	1.6	$S_{MS}=F_aS_S$	0.080	$S_{DS}=2/3\ S_{MS}$	**0.054**
1.0	S_1	0.036	F_v	2.4	$S_{M1}=F_vS_1$	0.086	$S_{D1}=2/3\ S_{M1}$	**0.057**

4.4 UNSURFACED PAVEMENT DESIGN CONSIDERATIONS

Given the available information, ECS understands that unsurfaced (crushed aggregate) access road will be constructed during the construction phase of this project, and later will be utilized for occasional maintenance purposes. Traffic data was not available at the time of this report submittal. However, based on ECS experience with similar facilities, ECS anticipates 6 to 8 (H20-44) trucks per day during construction over a period of 6 months and less than 5 to 7 trucks per month and auto traffic. Based on the estimated traffic data and CBR value of 6, the computed thickness for unpaved crushed aggregate pavement is provided in the following Table.

RECOMMENDED MINIMUM URSURFACED PAVEMENT SECTION	
Pavement Material	Compacted Material Thickness (inches)
Surface Aggregate (Dense Graded ¾-inch)	3
Crushed Aggregate (Dense Graded 1¼ -inch) without Geogrid	8
Crushed Aggregate with TX5 Geogrid or lime treatment within upper 12 inches	6

Pavement Drainage: An important consideration with the design and construction of pavements is surface and subsurface drainage. Where standing water develops, either on the pavement surface or within the base course layer, softening of the subgrade and other problems related to the deterioration of the pavement can be expected. Based on our estimated groundwater level, ECS considers surface water infiltration to be the main source of water to be considered for pavement design on this project.

The final pavement surface is recommended to be shaped or crowned to properly direct surface water to adequate on or off-site stormwater drainage infrastructure. The pavement subgrade should be properly sloped to avoid dips or pockets where water may become trapped. Dips in the subgrade can result in a “bathtub” effect, which may trap water. This trapped water can soften the subgrade and potentially heave the pavement during freezing weather. The subgrade in areas requiring undercut and backfill with granular soils are recommended to be graded to drain toward a drain tile. The drain tile should be sloped a minimum of ½ to 1 percent to discharge to nearby storm sewers, drainage ditches or other appropriate drainage facilities. Edge drains should be installed where site grades slope toward the pavement edge to reduce the potential for water to enter the base course layer. Edge drains should be sloped to the nearest appropriate drainage facility. Water that ponds on the subgrade surface can lead to deterioration of the subgrade soils, reduction of the base course support characteristics, and result in pavement heave during freezing conditions. Good drainage should help reduce the possibility of the subgrade materials being wet over a long period of time. To reduce the potential for shallow perched water to develop in areas of the site, install "stub" or "finger" drains around catch basins and in other low-lying areas of the parking lot to reduce the accumulation of water above and within the subgrade soils and aggregate base.

Pavement Maintenance: A sound maintenance program should be implemented to help maintain and enhance the performance of pavements and help attain the design service life. A preventative maintenance program should be started early in the pavement life to be effective. The "standard in the industry" supported by research indicates that preventative maintenance should typically begin within 2 to 5 years of the placement of pavement. In addition, routine maintenance should be performed, particularly in turning areas with repeated load applications. At a minimum, these areas will require regrading periodically to maintain effective drainage and to reshape after rutting and shoving. Observe pavements for distresses, such as depressions and poor drainage, typically once in the spring and once in the fall.

4.5 ELECTRICAL RESISTIVITY

The results of the in-situ (Wenner Four-Probe) resistivity testing can be found in the *Soil Resistivity Testing Report* included in Appendix B. The results indicate resistance generally decreases with depth. Based on the test results, we recommend using an average value of 11,000 Ω-cm for grounding installed within 20 feet of the existing grade. However, we recommend testing of the completed grounding system for compliance with required ground specification prior to the startup of the location.

A major factor in the determination of soil corrosivity is electrical resistivity. Electrical resistivity is a measure of resistance to the flow of electrical current. Buried metal corrosion is an electrochemical process in which the amount of metal loss due to corrosion is directly proportional to the flow of electrical current from the metal into the soil. Corrosion currents, following Ohm's Law, are inversely proportional to soil resistivity. Lower electrical resistivities typically occur in soil with higher moisture and higher soluble salt contents, which can be an indication of corrosive soil. A severity rating of corrosivity toward ferrous metals based on resistivity is listed below:

CORROSION SEVERITY RATING BASED ON SOIL RESISTIVITY (1)	
Soil Resistivity (ohm-cm)	**Corrosivity Rating**
>20,000	Essentially noncorrosive
10,000 – 20,000	Mildly corrosive
5,000 – 10,000	Moderately corrosive
3,000 – 5,000	Corrosive
1,000 – 3,000	Highly corrosive
< 1,000	Extremely corrosive

Notes:
1. Poursaee, Rangaraju, and Ding, 2019, Evaluation of H-pile Corrosion Rates for WI Bridges Located in Aggressive Subsurface Environments, Report No. WHRP 0092-16-03.

The results of the electrical resistivity testing suggest the tested soil is mildly corrosive. Note soil resistivity is not the only parameter affecting the risk of corrosion damage and a high soil resistivity does not guarantee the absence of corrosion potential. If corrosion potential is a concern, then we recommend additional testing and analysis be conducted for the soils within the area of the boring. The additional testing should include soil pH, Redox Potential, and sulfide content. We then recommend rating the soil using the American Water Works Association (AWWA) 10-Point system for corrosion potential.

5.0 SITE CONSTRUCTION RECOMMENDATIONS

5.1 SUBGRADE PREPARATION

5.1.1 Stripping and Initial Site Preparation

The subgrade preparation should consist of stripping vegetation, rootmat, topsoil, undocumented fill, and other soft/very loose or inadequate materials from the 10-foot expanded foundation limits, 5-foot expanded pavement limits, and 5 feet beyond the toe of structural fills, where feasible. ECS should be retained to observe and document that topsoil and other poor surficial materials have been removed prior to the placement of structural fill or construction of structures.

5.1.2 Possible Fill

Shallow possible fill soils were encountered in both soil borings, which extended to depths of about 2 to 4 feet below the existing site grades. For the possible fill, it can be difficult to distinguish “clean” fill from the natural soils in the absence of deleterious materials in the boring samples. With undocumented fill there is an inherent risk for the owner that generally stems from the potential for deleterious material to exist within the fill or to be buried by the fill that was not found by the boring exploration, which could go undetected. This risk of unforeseen conditions cannot be eliminated without complete removal of the existing fill but can be reduced by performing the recommended observation and testing. Using undocumented material for support of grade supported structures would require acceptance of risk by the owner. It is not the geotechnical engineer’s responsibility to accept the risk, but rather to inform others of the risk so that they may make a judgment as to what level of cost versus performance risk is acceptable to them.

<u>Test pits at this site may also be prudent to better check the composition of the possible fill and to better evaluate its ability for support of the structure and the associated risks.</u>

Fill in Pavement Areas: If undocumented fill is present, then below are two alternatives for subgrade preparation in pavement areas with varying anticipated cost and level of risk associated.

- **Alternative 1 (Complete Removal and Replacement)**: Completely remove the existing fill materials from pavement areas and replace with structural fill. Note that removal and replacement of the undocumented existing fill material would require excavations of approximately an average depth of at least 4 feet below existing site grades. <u>However, deeper fill may be present in areas not explored by the borings</u>. This option carries a low risk of poor pavement performance but is anticipated to have the greatest construction costs of the alternatives presented.
- **Alternative 2 (Proofroll and Replace as Needed):** After the site has been stripped and the subgrade has been exposed, the subgrade should be proofrolled as recommended in the <u>*Proofrolling*</u> Section below. This alternative will only identify near surface soils that are

inadequate for pavement support, and unidentified deeper pockets of inadequate fill could lead to poor performance of the pavements. This alternative is expected to have the lowest construction costs of the alternatives presented, but also has a moderate risk of poor pavement performance.

5.1.3 Frost Susceptible Soils

The frost susceptible silty and clayey soils encountered in the borings provide a concern for the pavement system. A risk for reduced pavement performance exists with the construction of pavements on frost susceptible soil. The reduced pavement performance may occur because of potential detrimental frost heaving and spring thaw weakening. The risk associated with frost susceptible soils can be reduced by removal of frost susceptible soils from within 3 feet of the finished pavement grade. The risk at this site related to the frost susceptible soils is generally moderate. However, the risk is expected to be high in areas where highly frost susceptible soil such as SILT (ML) or SILTY CLAY (CL-ML) is present within 3 feet of the finished pavement grade.

ECS anticipates the moderately frost susceptible soil will remain in place below pavements provided the soil meets strength requirements and contains less than 5 percent organic content. However, ECS recommends removing highly frost susceptible soils from within 3 feet of the finished pavement grade where it is encountered during construction. The ends of over-excavated areas should be sloped across a minimum length of 10 feet to reduce the potential abrupt changes in the pavement support characteristics that could lead to future pavement distress. The removed material should then be replaced with a properly compacted engineered fill.

5.1.4 Proofrolling

After the removal of inadequate surface materials, cutting to the proposed subgrade, and prior to the placement of structural fill or other construction materials, the exposed pavement subgrade should be observed by ECS. The exposed subgrade should be proofrolled with construction equipment having a minimum axle load of 10 tons (e.g., fully loaded tandem-axle dump truck in clayey soils or large smooth drum roller in sandy soils). Proofrolling should be traversed with overlapping passes of the vehicle under the observation of ECS. This procedure is intended to assist in identifying localized yielding materials.

Unstable or pumping subgrade areas identified during the proofroll should be repaired prior to the placement of subsequent structural fill or other construction materials. Unstable subgrade repair methods, such as undercutting, or moisture conditioning and recompaction, or chemical stabilization, should be discussed with ECS to determine the appropriate procedures regarding the existing conditions causing the instability. Test pits may be excavated in unstable areas to explore the shallow subsurface materials and to help determine the appropriate remedial action to stabilize the subgrade.

Seasonal reduction of the near surface soil strength can occur during wet times of the year (such as during the spring and fall months, and as evident during drilling operations) or immediately following extended periods of rain. This may result in additional unstable or pumping subgrade areas. Some undercutting or repair of unstable subgrade soils should be anticipated during pavement subgrade preparation**.** The method of subgrade repair or improvement chosen may be influenced by several factors such as weather and schedule, as well as the area, depth and nature of the unstable subgrade soils. Depending on these and other factors, potential subgrade repair methods are described below, but the actual depth of subgrade undercut and/or stabilization method should be determined at the time of construction. Some common subgrade repair methods include:

Scarification and Compaction: Soils can be scarified, moisture conditioned (i.e., dried or wetted) to within a narrow range of the material's optimum moisture content and compacted. Scarification and compaction is generally most applicable where very shallow unstable conditions are encountered and at times when the soil can be properly dried or wetted to within a narrow range of the materials optimum moisture content.

Undercut and Replacement: ECS recommends soft or yielding soils be evaluated in approximately 6 to 12-inch intervals to help limit the volume of undercuts. If soft or yielding soils are identified, the contractor should remove only 6 to 12 inches of material at a time in the subject area and then proofroll/evaluate the undercut subgrade to determine if additional undercut is needed. This may take more time but could potentially reduce the removal of more soil than necessary. Use of a geogrid could also be considered to locally reduce undercut depths. A geogrid, if used, should be placed after underground work, such as utility construction, is complete. Do not operate equipment on the geogrid until after 1 foot of structural fill is placed above it. Depending on the conditions at the time of repair, use of an aggregate structural fill, such as crushed stone, crushed concrete or gravel, may be needed.

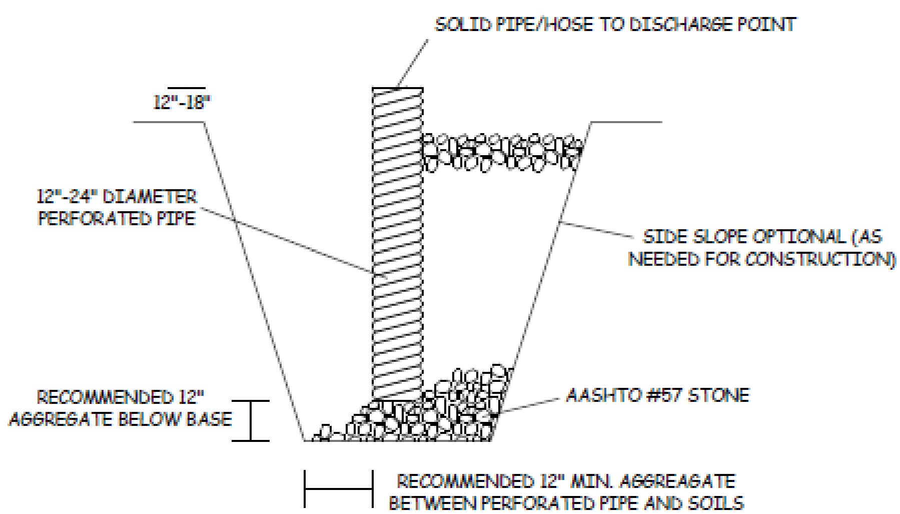
Chemical Modification: Alternatively, if these soils cannot be stabilized by conventional methods, chemical modification of the subgrade soils, such as with lime, lime kiln dust, cement, cement kiln dust, or other materials, may be utilized to reduce the moisture content and/or provide additional stabilization. An experienced pre-qualified contractor that has successfully chemically modified similar-sized projects with similar soil conditions is recommended to be used. The soil modification procedure, such as determination of the type and quantity of additive, and mixing and curing procedures, should be evaluated before implementation. This evaluation may include testing the soil to check if an adverse chemical reaction could occur. Chemical modification agents can have caustic effects to humans and property. The contractor should be required to minimize dusting or implement dust control measures. For preliminary estimating purposes, the approximate incorporation rate (based on dry weight of soil) is typically in the range of 6 to 7 percent, by dry weight, for hydrated lime or lime by-products, and 5 to 7 percent, by dry weight, for Portland cement. Typically, the percentage needed is less for hydrated lime than other lime by-products because the available calcium oxide content of lime by-products tends to be lower. Note insufficient mellowing of modified soils could lead to heaving after placement. Subgrade modification can result in the creation of an 'aquiclude' layer which will allow water to pond above the stabilized surface within the base course. Such water, if not drained properly, can freeze in cold weather potentially resulting in significant heave of the pavement. Alterations to the pavement sections to include additional drainage, such as an open-graded drainage aggregate layer, may be needed if a chemically modified subgrade is used.

5.1.5 Site Temporary Dewatering

The contractor shall make their own assessment of temporary dewatering needs based upon the limited subsurface groundwater information presented in this report. Soil sampling is not continuous, and thus soil and groundwater conditions may vary between sampling intervals (typically 5 feet). If the contractor believes additional subsurface information is needed to assess dewatering needs, they should obtain such information at their own expense. ECS makes no warranties or guarantees regarding the adequacy of the provided information to determine dewatering requirements; such recommendations are beyond our scope of services.

Dewatering systems are a critical component of many construction projects. Dewatering systems should be selected, designed, and maintained by a qualified and experienced (specialty or other) contractor familiar with the geotechnical and other aspects of the project. The failure to properly design and maintain a dewatering system for a given project can result in delayed construction, unnecessary foundation subgrade undercuts, detrimental phenomena such as ‘running sand’ conditions, internal erosion (i.e., ‘piping’), the migration of ‘fines’ down-gradient towards the dewatering system, localized settlement of nearby infrastructure, foundations, slabs, and pavements, etc. Water discharged from a site dewatering system shall be discharged in accordance with local, state, and federal requirements.

Strategies for Addressing Perched Groundwater: The typical primary strategy for addressing perched groundwater seeping into excavations is pumping from a trench (or French drain) and sump pits with sump pumps. The inlet of the sump pump is placed at the bottom of the corrugated pipe and the discharge end of the sump is directed to an appropriate stormwater drain. A typical sump pump drain (found in a sump pit or along a French drain) is depicted below:



Sump Pit/Pump Diagram

A typical French drain consists of an 18 to 24-inches wide by 18 to 24-inches deep bed of AASHTO No. 57 (or similar open graded aggregate) aggregate wrapped in a medium duty, non-woven geotextile and (sometimes) containing a 6-inch diameter, Schedule 40 PVC perforated or slotted pipe. Actual dimensions should be as determined necessary by ECS during construction. After the installation has been completed, the geotextile should be wrapped over the top of the aggregate and pipe followed by placement of backfill. The top of the drain should be positioned at least 18 inches below the design subgrade elevations. Drains should not be routed within the expanded building limits.

Pumping wells or a vacuum system could also be used to address perched groundwater. These techniques often are only effective during the initial depletion of the perched water quantity and may quickly be ineffective at addressing accumulation of water from rain, snow, etc.

Surface Drainage: The surface soils may be erodible. Therefore, the contractor should provide and maintain good site surface drainage during earthwork operations to maintain the integrity of the surface soils. Erosion and sedimentation controls should be in accordance with sound engineering practices and local requirements. Surface water should be directed away from the construction area, and the work area should be sloped away from the construction area at a gradient of 1 percent or steeper to reduce the potential of ponding water and the subsequent saturation of the surface soils. At the end of each workday, the subgrade soils should be sealed by rolling the surface with a smooth drum roller to reduce infiltration of surface water.

5.2 EARTHWORK OPERATIONS

5.2.1 Structural Fill

Prior to placement of structural fill, representative bulk samples (about 50 pounds) of on-site and off-site borrow should be submitted to ECS for laboratory testing, which will typically include natural moisture content, Atterberg limits, grain-size distribution, and moisture-density relationships (i.e., Proctors) for compaction. Imported materials should be tested prior to being hauled to the site to determine if they meet project specifications. Alternatively, Proctor data from other accredited laboratories can be submitted if the test results are within the last 90 days.

Satisfactory Structural Fill Materials: Structural fill is defined as inorganic soils with the following engineering properties and compaction requirements:

STRUCTURAL FILL INDEX PROPERTIES	
Subject	**Property**
Liquid Limit (LL) and Plasticity Index (PI)	LL < 40, PI < 20
Maximum Particle Size	3 inches
Maximum Fines Content Passing #200 Sieve	25% by dry weight
Maximum Organic Content	5% by dry weight

STRUCTURAL FILL COMPACTION REQUIREMENTS	
Subject	**Requirement**
Compaction Standard	Modified Proctor, ASTM D1557

STRUCTURAL FILL COMPACTION REQUIREMENTS	
Subject	**Requirement**
Minimum Required Compaction	95% of Max. Dry Density
Moisture Content	-2 to +3% points of the soil's optimum value
Maximum Loose Thickness	8 inches prior to compaction

On-Site Borrow: The moderately SILTY SAND (SM) soils (P200 ≤ 25 percent) would likely meet our recommendations for use as structural fill but should be further evaluated and tested by ECS prior to use. On-site soil used as structural fill should be free of frozen matter, deleterious materials, or chemicals that may result in the material being classified as "contaminated." Some conditions at the time of construction, such as wet or freezing weather, may preclude the use of on-site soil, and it may be necessary to use an imported less moisture sensitive or less frost susceptible granular material. Some of the soil samples appeared to have a relatively high moisture content, so the contractor should expect some drying of on-site soil prior to reuse as structural fill.

Fill Placement: Fill materials should not be placed on frozen soils, on frost-heaved soils, and/or on excessively wet soils. Borrow fill materials should not contain frozen materials at the time of placement, and frozen or frost-heaved soils should be removed prior to placement of structural fill or other fill soils and aggregates. Excessively wet soils or aggregates should be scarified, aerated, and moisture conditioned.

Structural fill placed below foundations and within the foundation influence zone should extend 1 foot beyond the outside edges of the footings and from that point, outward laterally 1 foot for every 2 feet of fill thickness below the footing. Use of lean mix concrete to limit lateral over-excavation may not be effective at this site because of caving of excavation sidewalls within the granular soils. In addition, we ***strongly*** recommend ECS document the material exposed in the excavations does not exhibit obvious characteristics that would adversely affect the performance of the foundation system.

Compaction Equipment: Compaction equipment appropriate to the soil type being compacted should be used to compact the subgrades and fill materials. Sheepsfoot compaction equipment should be used for the compaction of fine-grained soils (Clays and Silts). A vibratory steel drum roller should be used for compaction of coarse-grained soils (Sands and Gravels) as well as to help seal compacted surfaces.

5.3 FOUNDATION OBSERVATIONS

Protection of Foundation Excavations: Exposure to the environment may weaken the soils at the footing bearing level if the foundation excavations remain open for too long a time. Foundation concrete should be placed the same day that excavations are made. If the bearing soils are softened by surface water intrusion or exposure, then the softened soils should be removed from the foundation excavation bottom immediately prior to placement of concrete. If the excavation must remain open overnight, or if rainfall becomes imminent while the bearing soils are exposed, then a 1 to 3-inch thick "mud mat" of "lean" concrete should be placed on the bearing soils before the placement of reinforcing steel.

Footing Subgrade Observations: Because the boring encountered possible fill soils, it will be important to have ECS observe the foundation subgrade prior to placing foundation concrete, to confirm and document the anticipated bearing soils. Where undocumented fill, organic soils, soft/very loose soil, or inadequate soils are observed in the foundation influence zone, we recommend the removal of the poor soils. Undercuts should be backfilled with structural fill up to the original design bottom of footing elevation. The original footing is then recommended to be constructed on top of the structural fill.

5.4 UTILITY INSTALLATIONS

Utility construction should be in accordance with *The Standard Specifications for Sewer and Water Line Construction in Wisconsin*.

Utility Subgrades: ECS expects the soils encountered in our exploration to be generally adequate for support of utility pipes at typical utility depths except for the existing fill and soft/very loose soil mentioned above. The pipe subgrade should be observed and probed for stability by ECS to confirm the encountered materials meet our recommendations. Existing fill, soft/very loose, organic, or otherwise substandard materials encountered at the utility pipe subgrade elevation should be removed and replaced with properly compacted structural fill or pipe bedding material.

Utility Backfilling: The granular bedding material should be at least 4 inches thick, but not less than that specified by the project drawings and specifications. ECS recommends granular bedding consist of crushed stone chips in accordance with Table 32 and Chapter 8.43.0 of The Standard Specifications for Sewer and Water Line Construction in Wisconsin. Fill placed for support of the utilities, as well as backfill over the utilities, should satisfy the recommendations for engineered fill given in this report. We recommend cover material consist of material in accordance with Table 36 and Chapter 8.43.3 of The Standard Specifications for Sewer and Water Line Construction in Wisconsin. Granular backfill material should consist of material in accordance with Table 37 and Chapter 8.43.4 of The Standard Specifications for Sewer and Water Line Construction in Wisconsin. Excavated material in accordance with Chapter 8.43.5 of The Standard Specifications for Sewer and Water Line Construction in Wisconsin, and as recommended in the Earthwork Operations Section of this report could also be used as backfill.

We do not recommend flood compaction of the backfill, especially within a cohesive soil excavation, where cohesive soils are used as backfill, and/or where a shallow water table exists. Mechanical compaction is recommended and preferred since it generally provides more uniform compaction than flood compaction.

Excavation Safety: The contractor should make and maintain excavations and slopes in accordance with OSHA excavation safety standards. The contractor is solely responsible for designing and constructing stable, excavations and slopes and should shore, slope, or bench the sides of the excavations and slopes as required to maintain stability of both the excavation sides and bottom.
The contractor's responsible person, as defined in OSHA 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, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal safety regulations. ECS is providing this information solely as a service to our client. ECS is not assuming responsibility for construction site safety or the

contractor's activities; ECS does not imply such responsibility, and the contractor, design team and owner should not infer it.

6.0 CLOSING

ECS has prepared this report to guide the geotechnical-related design and construction aspects of the project. We performed these services in accordance with the standard of care expected of professionals in the industry performing similar services on projects of like size and complexity at this time in the region. No other representation expressed or implied, and no warranty or guarantee is included or intended in this report.

The description of the proposed project is based on information provided to ECS by Mission Support Services, LLC. If this information is inaccurate or changes, either because of our interpretation of the documents provided or site or design changes that may occur later, then ECS should be contacted so we can review our recommendations and provide additional or alternate recommendations that reflect the proposed construction.

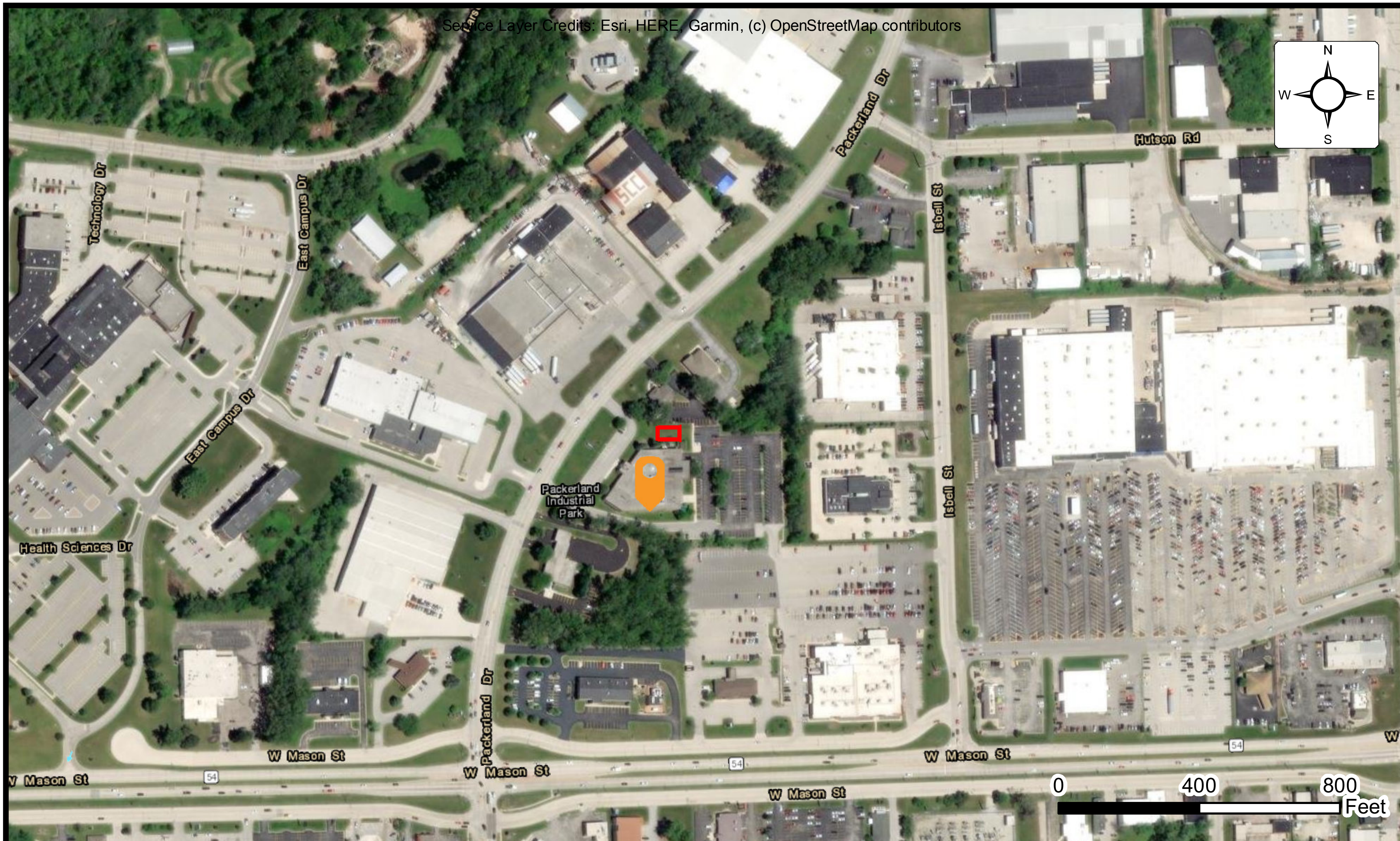
We recommend that ECS review the project plans and specifications so we can confirm that those plans/specifications are in accordance with the recommendations of this geotechnical report.

Field observations, and quality assurance testing during earthwork and foundation installation are an extension of, and integral to, the geotechnical design. We recommend that ECS be retained to apply our expertise throughout the geotechnical phases of construction, and to provide consultation and recommendation should issues arise.

ECS is not responsible for the conclusions, opinions, or recommendations of others based on the data in this report.

APPENDIX A – Diagrams and Reports

Site Location Diagram
Boring Location Diagram
Subsurface Cross-Section
Soil Survey Map



SITE LOCATION DIAGRAM
PROPOSED SKENANDOAH MONOPOLE TOWER

909 PACKERLAND DRIVE, GREEN BAY, WISCONSIN
MISSION SUPPORT SERVICES

ENGINEER	MEK1
SCALE	AS NOTED
PROJECT NO.	59:3567
FIGURE	1 OF 1
DATE	8/14/2023

BORING LOCATION DIAGRAM
PROPOSED SKENANDOAH MONOPOLE TOWER

909 PACKERLAND DRIVE, GREEN BAY, WISCONSIN
MISSION SUPPORT SERVICES

ENGINEER	MEK1
SCALE	AS NOTED
PROJECT NO.	59:3567
FIGURE	1 OF 1
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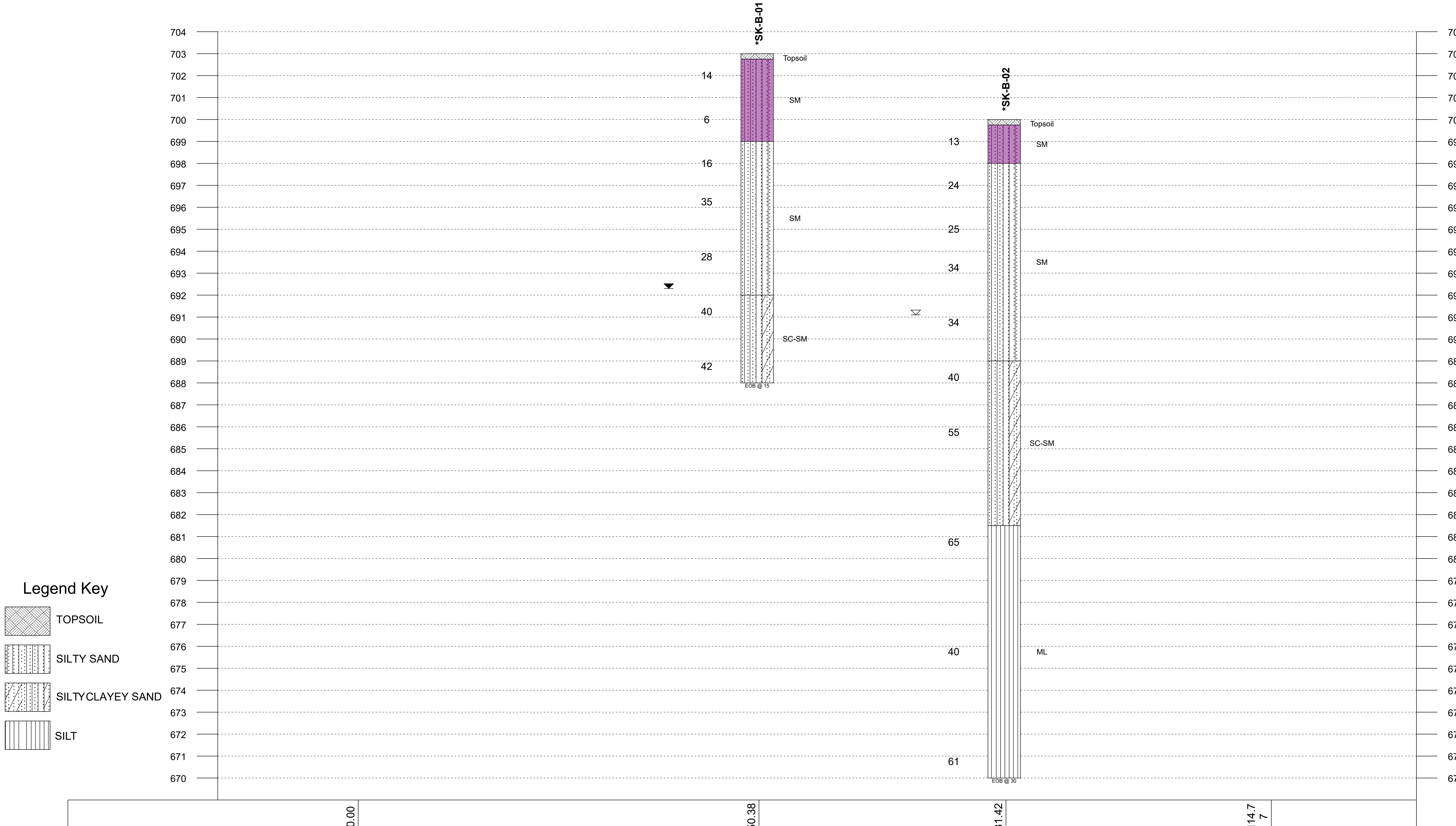


Notes:

1- EOB: END OF BORING AR: AUGER REFUSAL SR: SAMPLER REFUSAL.
2- THE NUMBER BELOW THE STRIPS IS THE DISTANCE ALONG THE BASELINE.
3- SEE INDIVIDUAL BORING LOG AND GEOTECHNICAL INFORMATION.
4- STANDARD PENETRATION TEST RESISTANCE (LEFT OF BORING) IN BLOWS PER FOOT (ASTM D1586).

Plastic Limit Water Content Liquid Limit
[FINES CONTENT %]
BOTTOM OF CASING
LOSS OF CIRCULATION
CALIBRATED PENETROMETER

WL (First Encountered)
WL (Completion)
WL (Estimated Seasonal High Water)
WL (Stabilized)

Fill
Possible Fill
Probable Fill
Rock

GENERALIZED SUBSURFACE SOIL PROFILE

Section line 1

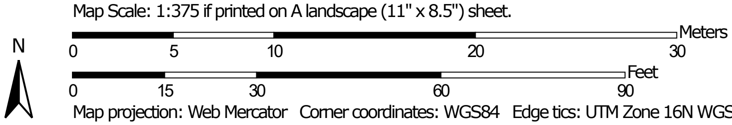
Proposed Skenandoah Monopole Tower

Mission Support Services

909 Packerland Drive, Green Bay, Wisconsin, 54303

Project No: 59:3567 Date: 08/14/2023

Map Scale: 1:375 if printed on A landscape (11" x 8.5") sheet.



Map projection: Web Mercator Corner coordinates: WGS84 Edge tics: UTM Zone 16N WGS84

APPENDIX B – Field Operations

Subsurface Exploration Procedure: Standard Penetration Testing (SPT)
Reference Notes for Boring Logs
Boring Logs B-01 and B-02
In-Situ Electrical Resistivity Test Results

SUBSURFACE EXPLORATION PROCEDURE:
STANDARD PENETRATION TESTING (SPT)
ASTM D 1586
Split-Barrel (Split-Spoon) Sampling

Standard Penetration Testing, or **SPT**, is the most frequently used subsurface exploration test performed worldwide. This test provides samples for identification purposes as well as a measure of penetration resistance, or N-Value. The N-Value, or blow counts, when corrected and correlated, can approximate engineering properties of soils used for geotechnical design and engineering purposes.

SPT Procedure:

- Involves driving a 2-inch outside diameter hollow tube (split-spoon) into the ground by dropping a 140-lb hammer a height of 30 inches at desired depth
- Recording the number of hammer blows required to drive the split-spoon a distance of 12 inches (in 3 or 4 Increments of 6 inches each)

- Auger is advanced* and an additional SPT is performed
- One SPT test is typically performed every 2½ to 5 feet.
- Obtain a 1⅜-inch diameter soil sample

**Drilling Methods May Vary – The predominate drilling methods used for SPT are open hole fluid rotary drilling and hollow-stem auger drilling.*

REFERENCE NOTES FOR BORING LOGS

MATERIAL[1,2]		
	ASPHALT	
	CONCRETE	
	GRAVEL	
	TOPSOIL	
	VOID	
	BRICK	
	AGGREGATE BASE COURSE	
GW	WELL-GRADED GRAVEL	gravel-sand mixtures, little or no fines
GP	POORLY-GRADED GRAVEL	gravel-sand mixtures, little or no fines
GM	SILTY GRAVEL	gravel-sand-silt mixtures
GC	CLAYEY GRAVEL	gravel-sand-clay mixtures
SW	WELL-GRADED SAND	gravelly sand, little or no fines
SP	POORLY-GRADED SAND	gravelly sand, little or no fines
SM	SILTY SAND	sand-silt mixtures
SC	CLAYEY SAND	sand-clay mixtures
ML	SILT	non-plastic to medium plasticity
MH	ELASTIC SILT	high plasticity
CL	LEAN CLAY	low to medium plasticity
CH	FAT CLAY	high plasticity
OL	ORGANIC SILT or CLAY	non-plastic to low plasticity
OH	ORGANIC SILT or CLAY	high plasticity
PT	PEAT	highly organic soils

DRILLING SAMPLING SYMBOLS & ABBREVIATIONS

SS	Split Spoon Sampler	PM	Pressuremeter Test
ST	Shelby Tube Sampler	RD	Rock Bit Drilling
WS	Wash Sample	RC	Rock Core, NX, BX, AX
BS	Bulk Sample of Cuttings	REC	Rock Sample Recovery %
PA	Power Auger (no sample)	RQD	Rock Quality Designation %
HSA	Hollow Stem Auger		

PARTICLE SIZE IDENTIFICATION

DESIGNATION		PARTICLE SIZES
Boulders		12 inches (300 mm) or larger
Cobbles		3 inches to 12 inches (75 mm to 300 mm)
Gravel:	Coarse	¾ inch to 3 inches (19 mm to 75 mm)
	Fine	4.75 mm to 19 mm (No. 4 sieve to ¾ inch)
Sand:	Coarse	2.00 mm to 4.75 mm (No. 10 to No. 4 sieve)
	Medium	0.425 mm to 2.00 mm (No. 40 to No. 10 sieve)
	Fine	0.074 mm to 0.425 mm (No. 200 to No. 40 sieve)
Silt & Clay ("Fines")		<0.074 mm (smaller than a No. 200 sieve)

COHESIVE SILTS & CLAYS

UNCONFINED COMPRESSIVE STRENGTH, QP[4]	SPT[5] (BPF)	CONSISTENCY[7] (COHESIVE)
<0.25	<2	Very Soft
0.25 - <0.50	2 - 4	Soft
0.50 - <1.00	5 - 8	Medium Stiff
1.00 - <2.00	9 - 15	Stiff
2.00 - <4.00	16 - 30	Very Stiff
4.00 - 8.00	31 - 50	Hard
>8.00	>50	Very Hard

RELATIVE AMOUNT[7]	COARSE GRAINED (%)[8]	FINE GRAINED (%)[8]
Trace	≤5	≤5
With	10 - 20	10 - 25
Adjective *(ex: "Silty")*	25 - 45	30 - 45

GRAVELS, SANDS & NON-COHESIVE SILTS

SPT[5]	DENSITY
<5	Very Loose
5 - 10	Loose
11 - 30	Medium Dense
31 - 50	Dense
>50	Very Dense

WATER LEVELS[6]

- ▽ WL (First Encountered)
- ▼ WL (Completion)
- WL (Seasonal High Water)
- WL (Stabilized)

FILL AND ROCK

FILL | POSSIBLE FILL | PROBABLE FILL | ROCK

[1]Classifications and symbols per ASTM D 2488-17 (Visual-Manual Procedure) unless noted otherwise.

[2]To be consistent with general practice, "POORLY GRADED" has been removed from GP, GP-GM, GP-GC, SP, SP-SM, SP-SC soil types on the boring logs.

[3]Non-ASTM designations are included in soil descriptions and symbols along with ASTM symbol [Ex: (SM-FILL)].

[4]Typically estimated via pocket penetrometer or Torvane shear test and expressed in tons per square foot (tsf).

[5]Standard Penetration Test (SPT) refers to the number of hammer blows (blow count) of a 140 lb. hammer falling 30 inches on a 2 inch OD split spoon sampler required to drive the sampler 12 inches (ASTM D 1586). "N-value" is another term for "blow count" and is expressed in blows per foot (bpf). SPT correlations per 7.4.2 Method B and need to be corrected if using an auto hammer.

[6]The water levels are those levels actually measured in the borehole at the times indicated by the symbol. The measurements are relatively reliable when augering, without adding fluids, in granular soils. In clay and cohesive silts, the determination of water levels may require several days for the water level to stabilize. In such cases, additional methods of measurement are generally employed.

[7]Minor deviation from ASTM D 2488-17 Note 14.

[8]Percentages are estimated to the nearest 5% per ASTM D 2488-17.

CLIENT:	PROJECT NO.:	BORING NO.:	SHEET:
Mission Support Services	59:3567	SK-B-01	1 of 1

PROJECT NAME:	DRILLER/CONTRACTOR:
Proposed Skenandoah Monopole Tower	ECS59 - Crew 2

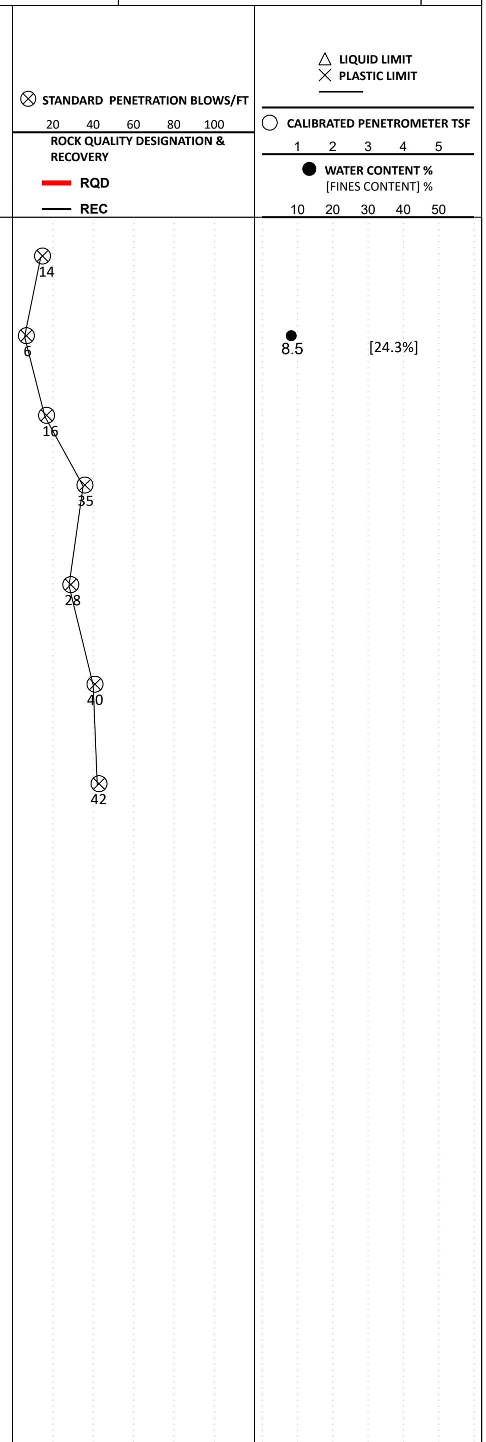
SITE LOCATION: **909 Packerland Drive, Green Bay, Wisconsin, 54303**

LOSS OF CIRCULATION ⟩100%⟩

BOTTOM OF CASING

NORTHING:	EASTING:	STATION:	SURFACE ELEVATION:
-227460.5	2464818.1		703

DEPTH (FT)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE DIST. (IN)	RECOVERY (IN)	DESCRIPTION OF MATERIAL	WATER LEVELS	ELEVATION (FT)	BLOWS/6"
					Topsoil Thickness[3"]			
	S-1	SS	24	20	(SM) POSSIBLE FILL, SILTY SAND WITH CLAY SEAMS, dark brown, moist, medium dense to loose			8-7-7-6 (14)
	S-2	SS	24	18				5-4-2-2 (6)
5	S-3	SS	24	23	(SM) Glacial Outwash, SILTY SAND, trace clay, light brown, moist, medium dense to dense		698	5-6-10-14 (16)
	S-4	SS	18	17				11-15-20 (35)
	S-5	SS	18	18				10-13-15 (28)
10						▼	693	
	S-6	SS	18	17	(SC-SM) Glacial outwash, SILTY CLAYEY SAND, light brown, wet, dense			15-20-20 (40)
	S-7	SS	18	18				11-17-25 (42)
15					**END OF BORING AT 15 FT**		688	
20							683	
25							678	
30							673	



THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL

WL (First Encountered)	None	BORING STARTED:	Jul 06 2023	CAVE IN DEPTH:	
WL (Completion)	10.60	BORING COMPLETED:	Jul 06 2023	HAMMER TYPE:	Auto
WL (Seasonal High Water)		EQUIPMENT: Truck	LOGGED BY: YP	DRILLING METHOD:	3-1/4" Hollow stem auger
WL (Stabilized)					

GEOTECHNICAL BOREHOLE LOG

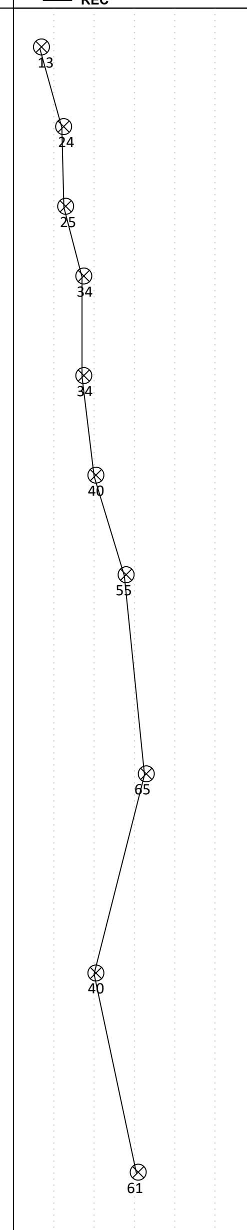
CLIENT:	PROJECT NO.:	BORING NO.:	SHEET:
Mission Support Services	**59:3567**	**SK-B-02**	**1 of 1**
PROJECT NAME: **Proposed Skenandoah Monopole Tower**	DRILLER/CONTRACTOR: **ECS59 - Crew 2**		
SITE LOCATION: **909 Packerland Drive, Green Bay, Wisconsin, 54303**			

NORTHING:	EASTING:	STATION:	SURFACE ELEVATION:
-227459.4	**2464849.1**		**700**

LOSS OF CIRCULATION

BOTTOM OF CASING

DEPTH (FT)	SAMPLE NUMBER	SAMPLE TYPE	SAMPLE DIST. (IN)	RECOVERY (IN)	DESCRIPTION OF MATERIAL	WATER LEVELS	ELEVATION (FT)	BLOWS/6"
	S-1	SS	24	17	Topsoil Thickness[3"] (SM) POSSIBLE FILL, SILTY SAND, brown, moist, medium dense			4-5-8-9 (13)
	S-2	SS	24	15	(SM) Glacial outwash, SILTY SAND, brown to light brown, moist to wet, medium dense to dense			8-10-14-18 (24)
5	S-3	SS	24	17			695	11-15-10-24 (25)
	S-4	SS	18	16				11-16-18 (34)
	S-5	SS	18	17		▽		11-15-19 (34)
10							690	
	S-6	SS	18	18	(SC-SM) Glacial outwash, SILTY CLAYEY SAND, brown, wet, dense to very dense			10-17-23 (40)
	S-7	SS	18	15				21-23-32 (55)
15							685	
	S-8	SS	18	12	(ML) Lacustrine, SILT, grayish brown, wet, dense to very dense			13-27-38 (65)
20							680	
	S-9	SS	18	15				15-12-28 (40)
25							675	
	S-10	SS	18	17				21-27-34 (61)
30					**END OF BORING AT 30 FT**		670	



LIQUID LIMIT
PLASTIC LIMIT
CALIBRATED PENETROMETER TSF
1 2 3 4 5
WATER CONTENT %
[FINES CONTENT] %
10 20 30 40 50

[12.2%]

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES. IN-SITU THE TRANSITION MAY BE GRADUAL

WL (First Encountered)	**8.80**	BORING STARTED:	**Jul 06 2023**	CAVE IN DEPTH:	
WL (Completion)	**None**	BORING COMPLETED:	**Jul 06 2023**	HAMMER TYPE:	**Auto**
WL (Seasonal High Water)		EQUIPMENT: **Truck**	LOGGED BY: **YP**	DRILLING METHOD:	**3-1/4" Hollow stem auger**
WL (Stabilized)					

GEOTECHNICAL BOREHOLE LOG

ELECTRICAL RESISTIVITY SOUNDING DATASHEET

Project Name:	Oneida Broadband Skenandoah Monopole
ECS Project No.	59:3567
Project Location:	909 Packerland Drive, Green Bay, Brown County, WI
Prepared for:	Mission Support Services
Instrument Used:	Fluke 1625 GEO Earth Ground Tester
Array Type:	Wenner 4-Electrode (ASTM G-57)

Test Location	Spacing (feet) A	Resistance (ohms) R	Apparent Resistivity (ohm-cm)
SK-B-01	2.5	137.40	65,780
Line Bearing: N/S	5	73.00	69,898
Tested 6-28-2023	10	14.40	27,576
	20	2.89	11,069
SK-B-01	2.5	126.80	60,706
Line Bearing: E/W	5	63.30	60,610
Tested 6-28-2023	10	17.46	33,436
	20	2.89	11,069

APPENDIX C – Laboratory Test Results

Laboratory Procedures

LABORATORY PROCEDURES:

Moisture content determination was performed on select fine-grained soil samples in accordance with ASTM D 2216.

Percentage Passing US#200 Standard Sieve, refers to amount of material finer than US#200 Standard Sieve (75-µm). This is used to help classify soils for engineering purposes. Tests were performed on select soil samples in accordance with ASTM D 1140.

APPENDIX D – Supplemental Report Documents

Important Information about This Geotechnical-Engineering Report

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 <u>not</u> 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 <u>Not</u> 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 <u>not</u> 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 <u>not</u> building-envelope or mold specialists.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

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