Geotechnical Engineering
Site Feasibility Study

Possible Residential Development
N. University Drive
Waukesha, Wisconsin

Prepared for:

Waukesha County
Waukesha, Wisconsin

January 14, 2025
Project No. 1G-2410035

GILES
ENGINEERING ASSOCIATES, INC.

GILES
ENGINEERING ASSOCIATES, INC.
GEOTECHNICAL, ENVIRONMENTAL & CONSTRUCTION MATERIALS CONSULTANTS

- Dallas, TX
- Los Angeles, CA
- Manassas, VA
- Milwaukee, WI

January 14, 2025

Waukesha County
Department of Parks and Land Use
515 W. Moreland Boulevard
Waukesha, WI 53188

Attention: Jim Rose
Senior Conservation Specialist

Subject: Geotechnical Engineering Site Feasibility Study
Possible Residential Development
N. University Drive
Waukesha, Wisconsin
Project No. 1G-2410035

Dear Mr. Rose:

As requested, Giles Engineering Associates, Inc. conducted a *Geotechnical Engineering Site Feasibility Study* regarding the subject property. The accompanying report describes the services that were performed, and it provides geotechnical-related information that was derived from those services.

We sincerely appreciate the opportunity to provide geotechnical consulting services for the proposed project. Please contact the undersigned if there are questions about the report or if we may be of further service.

Very truly yours,

GILES ENGINEERING ASSOCIATES, INC.

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BENJAMIN M.
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MILWAUKEE,
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Distribution: Waukesha County Department of Parks and Land Use
Attn: Jim Rose (email: jrose@waukeshacounty.gov)

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GEOTECHNICAL ENGINEERING SITE FEASIBILITY STUDY

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N. UNIVERSITY DRIVE
WAUKESHA, WISCONSIN
PROJECT NO. 1G-2410035

GEOTECHNICAL ENGINEERING SITE FEASIBILITY STUDY

POSSIBLE RESIDENTIAL DEVELOPMENT
N. UNIVERSITY DRIVE
WAUKESHA, WISCONSIN
PROJECT NO. 1G-2410035

1.0 SCOPE OF SERVICES

As requested, Giles Engineering Associates, Inc. ("Giles") conducted a *Geotechnical Engineering Site Feasibility Study* regarding the subject site, which is being considered as a possible location for a residential development. The study was only performed to evaluate the feasibility and practicality of developing the site; it was not performed to provide design or construction recommendations. Geotechnical-related recommendations for design and construction of a future development should be determined from a comprehensive geotechnical engineering evaluation including additional test borings and groundwater observation wells, which should be monitored over a sufficient timeframe. The comprehensive geotechnical engineering evaluation should be based on the final details of the future development, including the final layout and elevations of future structures, pavement areas, and utilities.

The *Geotechnical Engineering Site Feasibility Study* included a geotechnical subsurface exploration program, geotechnical laboratory services, and geotechnical engineering. The scope of each service area was narrow and limited as directed by our client and because Giles' objective was only to evaluate the feasibility and practicality of developing the subject site. Service areas are briefly described later. Geotechnical information regarding construction of a future residential development at the subject site is provided herein.

2.0 SITE DESCRIPTION

The subject site is south of the University of Wisconsin-Waukesha campus on the east side of N. University Drive in Waukesha, Wisconsin. The site is shown on the *Test Boring Location Plan*, enclosed as Figure 1 in Appendix A. When the test borings (discussed below) were performed, the site was occupied by soccer fields and grassy groundcover. It is understood that wetland areas exist at the north area of the site and that portions of the site are within a floodplain. Ground elevations at the test boring locations varied between ±El. 942.62 and ±El. 948.16; those elevations were provided by the client.

3.0 PROJECT DESCRIPTION

The site is being considered for a possible residential development with single-family and multi-family dwellings. This report assumes that the possible residential buildings will be one- or two-story wood-frame structures, typical of residential construction in southeast Wisconsin. It is understood that the residences will not have basements or below-grade spaces. The possible residential development includes roads, which will assumedly be constructed of asphalt-concrete pavement. This report assumes that relatively minor grade changes will be needed for site development, except that site grades will be raised, as needed, to position roads and structures sufficiently above the seasonal-high water table.

4.0 WEB SOIL SURVEY REVIEW

According to the *Web Soil Survey*, operated by the USDA Natural Resources Conservation Service and accessed online, near-surface soil at the site consists of Kendall silt loam, Pella silt loam, and Hochheim loam. The *Web Soil Survey* states that the depth to water table is about 0 inches for Kendall silt loam, about 12 to 36 inches for Pella silt loam, and more than 80 inches for Hochheim loam.

5.0 GEOTECHNICAL SUBSURFACE EXPLORATION PROGRAM

To evaluate the subsurface conditions, on a preliminary basis, ten geotechnical test borings were conducted at the site using an all-terrain drill rig. Each test boring was advanced to ±21 feet below-ground, except Test Boring 3 which was terminated at 14 feet below-ground due to auger refusal; auger refusal appears to be due to cobbles and boulders. Test boring locations were staked by the client. Approximate locations of the test borings are shown on the *Test Boring Location Plan*.

Soil samples were collected from each test boring, at certain depths, using the Standard Penetration Test (SPT), conducted with the drill rig. A brief description of the SPT is given in Appendix B along with descriptions of other field procedures. Immediately after sampling, select portions of SPT samples were placed in containers that were sealed and labeled at the site. A Standard Penetration Resistance value (N-value) was determined from each SPT. Measured N-values are reported on the *Test Boring Logs* (in Appendix A), which are records of the test borings.

The boreholes were backfilled upon completion; however, backfill material will likely settle or heave, creating a hazard that can injure people and animals. Borehole areas should, therefore, be carefully and routinely monitored by the property owner or by others; settlement and heave of backfill material should be repaired immediately. Giles will not monitor or repair boreholes.

Ground elevations at the test borings were provided by the client. The test boring elevations are noted on the *Test Boring Logs*.

6.0 GEOTECHNICAL LABORATORY SERVICES

Samples that were retained from the test borings were transported to Giles' geotechnical laboratory where the samples were classified using the descriptive terms and particle-size criteria shown on the *General Notes* in Appendix D and by using the Unified Soil Classification System (ASTM D 2488) as a general guide. The classifications are shown on the *Test Boring Logs* along with horizontal lines that show estimated depths of material change. Field-related information pertaining to the test borings is also shown on the *Test Boring Logs*. For simplicity and abbreviation, terms and symbols are used on the *Test Boring Logs;* the terms and symbols are defined on the *General Notes*.

Unconfined compression (without measured strain), torvane (shear vane), penetrometer resistance, and water content tests were performed on select soil samples to evaluate their general engineering properties. Results of the laboratory tests are on the *Test Boring Logs*. Because testing was performed on SPT samples, which are categorized as disturbed samples, results of the unconfined compression and penetrometer resistance tests are approximate. Laboratory procedures are briefly described in Appendix C.

7.0 MATERIAL CONDITIONS

Because material sampling at the test borings was discontinuous, it was necessary to estimate conditions between sample intervals. Estimated conditions at the test borings are briefly discussed in this section and are described in more detail on the *Test Boring Logs*. This report is based only on the estimated conditions shown on the *Test Boring Logs.*

7.1. Surface Materials

Topsoil that was between ±4 and ±16 inches thick was at the surface of the test borings and generally consisted of lean clay, silty clay, and sandy silt with trace to little amounts of organic matter. Approximately 4 inches of crushed limestone was beneath the topsoil at Test Boring 3.

7.2. Fill Material

At Test Borings 3, 5, and 6, material classified as fill was below the surface materials and was encountered to about ±4, ±2, and ±5 feet below-ground, respectively. In general, the fill material consisted of lean clay, sandy clay, and gravelly sand. Based on laboratory and field testing, the fill material exhibited relatively low to moderate strength characteristics.

7.3. Native Soil

Native soil was below the materials described above and extended to the termination depth at each test boring. In general, the native soil varied but was predominantly cohesive and included lean clay, silty clay, and sandy clay, but silty fine sand and silt were also encountered. Variable amounts of gravel were encountered throughout the native soil, and cobbles and boulders were encountered at Test Borings 1 and 3. Dark brown lean clay with estimated trace amounts of organic matter was encountered below the topsoil and fill at Test Borings 1, 2, 3 and 4 to depths between 3 and 5 feet below-ground. At a depth of about between ±18, a very dense “hard pan” layer was encountered at Test Borings 1, 6, and 8. This layer consisted of sandy silt and sandy clay with gravel. Based on laboratory testing, native cohesive soil exhibited comparative consistencies primarily between soft and stiff, but some very stiff to hard soils were encountered. Also, based on corrected SPT N-values, native granular soil exhibited firm to very dense relative densities, but at least some of the measured N-values are likely not representative of in-place density because gravel, cobbles, and/or boulders were encountered during sampling/testing.

8.0 GROUNDWATER CONDITIONS

It is estimated that the water table was about 1 to 2 feet below-ground at Test Borings 1 through 9, and about 9 feet below-ground at Test Boring 10. Considering the mottled soil, the moisture conditions and strength characteristics of the retained soil samples, and the *Web Soil Survey* (discussed in Section 3.0), the site appears to be subject to significant perched groundwater conditions, where groundwater occasionally perches above the water table and near or at the ground surface. Groundwater conditions will fluctuate.

Because Giles' estimate of the groundwater conditions is only an approximation, the actual groundwater conditions could differ from the conditions described above. Therefore, the water table could be higher or lower than estimated. If a precise determination of the water table is needed, groundwater observation wells are recommended to be installed and monitored at the site. Giles can install and monitor observation wells.

9.0 PRELIMINARY GEOTECHNICAL INFORMATION

This report provides preliminary geotechnical information to evaluate the feasibility and practicality of developing the subject site. This report is not for design or construction. Geotechnical-related recommendations for design and construction of a future development should be determined from a comprehensive geotechnical engineering evaluation including additional test borings and groundwater observation wells, which should be monitored over a sufficient timeframe. The comprehensive geotechnical engineering evaluation should be based on final details of the future development, including the final layout and elevations of structures, pavement areas, and utilities. Because the following information is preliminary and general it does not cover all geotechnical aspects of developing the site.

9.1. Preliminary Foundation Information

Spread Footing Foundations

Existing fill and low strength native soil were encountered at each test boring to depths ranging between approximately ±9 and ±18 feet below the existing ground surface. Existing fill is not considered suitable for support of foundations. Based on the test borings, it is expected that spread-footing foundations can be used to support residential structures at the proposed site (which are assumed to be relatively light). However, because of the relatively low-strength native soils, an engineered fill layer of compacted aggregate is recommended to completely underlie all foundations. It is recommended that the aggregate layer thickness be determined based on the final details of the proposed structures and based on additional test borings within the proposed structures. In addition, some overexcavation of unsuitable native soils beneath the engineered fill layer should be anticipated during foundation construction, considering the soils encountered at the test borings.

Based on the test boring and assuming that a uniform engineered fill layer is constructed beneath foundations, it is expected that spread-footing foundations could be designed using a maximum, net, allowable soil bearing capacity in the range of about 1,500 to 2,000 pounds per square foot (psf). However, the actual bearing capacity for foundation design should be determined on a per-building basis from a comprehensive geotechnical engineering evaluation. Unsuitable soil beneath the engineered fill layer could be replaced with engineered fill. Due to the variable soil conditions across the site, which included highly variable strength and compressibility characteristics of native soils, it is recommended that additional borings be conducted as part of a comprehensive *Geotechnical Engineering Exploration and Analysis* in order to provide those recommendations. Locations and depths of additional borings would be based on the layout and final details of the proposed development.

Ground Improvement Alternative

As an alternative to a relatively low bearing capacity and construction of an engineered fill layer beneath foundations, the foundation areas could be improved through specialized ground-improvement techniques, such as by installing compacted-aggregate piers or stone columns at predetermined locations within the foundation areas. Compacted aggregate piers and stone columns are proprietary systems installed by specialty ground-improvement contractors. Based on the test borings, it is expected that compacted aggregate piers or stone columns will extend about 3 to 5 feet into suitable native soil. However, the actual lengths and spacing of the ground-improvement elements must be determined by the ground-improvement contractor.

If the foundation areas are properly improved through ground improvement, it is expected that a spread-footing foundation can be used to support the possible residential building. For budgeting purposes, with proper ground improvement it is expected that spread footings can likely be designed using a maximum, net, allowable bearing capacity in the range of about 2,000 to 4,000 psf, but the ground-improvement contractor must provide the actual bearing capacity for foundation design.

9.2. Preliminary At-Grade Floor Slab Information

With proper site preparation, it is expected that existing site soil will be suitable to support at-grade floor slabs. Due to the existing fill, perched groundwater, and lower-strength native soil, subgrade improvement prior to construction of at-grade floors should be expected and budgeted. The need for, and type of, subgrade improvement should be determined during construction with the assistance of a geotechnical engineer. Engineered fill could also support ground-bearing floor slabs, provided the engineered fill is properly compacted and is placed on suitable-bearing existing soil.

9.3. Preliminary Pavement Information

The use of hot-mix asphalt (HMA) pavement and Portland cement concrete (PCC) pavement, both with an aggregate base, are suitable for the site. Pavement and base thicknesses will depend on the subgrade materials and expected traffic conditions. Due to the existing fill, perched groundwater, and lower-strength native soil, the subgrade within certain pavement areas will likely need to be improved. There are various methods of subgrade improvement, including over-excavation, coarse-aggregate modification, and soil stabilization with hydrated lime or Portland cement. The need for, and type of, subgrade improvement should be determined during construction with the assistance of a geotechnical engineer. Geogrid or geotextile could likely also be used to improve pavement support and might need to be below the base course or sub-base of pavement within certain areas, depending on the conditions that are encountered during construction. Also, depending on the pavement elevations, an under-pavement drain system might be necessary/beneficial, considering the shallow groundwater conditions.

9.4. Preliminary Construction Information

Drain tile

Because the site has been used for agriculture and considering the perched groundwater, drain-tile likely exists at the site. Drain tile that is encountered during construction should be rerouted around development areas and be discharged to suitable locations on a permanent basis. Drain tile should not be plugged, since it may drain large areas. Drain tile that is damaged during construction should be repaired. It is recommended that a geotechnical engineer observe drain tile prior to its repair or rerouting.

Use of Site Soil as Engineered Fill

It is expected that site soil that does not contain adverse organic content or other deleterious materials can be used as engineered fill. However, site soil will likely need to be moisture conditioned (uniformly moistened or dried) before being used as engineered fill. If construction is during adverse weather, drying site soil will likely not be feasible. In this case, fill material might need to be imported to the site.

Adverse Weather

Site soil is moisture sensitive and will likely become unstable when exposed to adverse weather, such as rain, snow, and freezing temperatures. Therefore, it might be necessary to remove or stabilize the upper 6 to 12 inches (or more) of soil due to adverse weather, which commonly occurs during late fall, winter, and early spring. At least some over-excavation or stabilization of unstable soil should be expected if construction is during or after adverse weather. Because site preparation is weather dependent, bids for site preparation and other earthwork activities should consider the time of year that construction will be conducted.

Dewatering

Construction dewatering might be necessary, considering the groundwater conditions. Filtered sump pumps, drawing water from sump pits excavated in the bottom of construction trenches, are expected to be adequate to remove water that collects in excavations. However, multiple sump pumps might be necessary, depending on the conditions that are encountered. Excavated sump pits should be lined with geotextile and filled with free-draining aggregate, such as crushed stone that meets the gradation requirements of ASTM No. 57 aggregate.

Excavation Stability

Excavations should be made in accordance with current OSHA excavation and trench safety standards and other applicable requirements. Sides of excavations will need to be benched, sloped, or braced to develop and maintain a safe work environment. Temporary shoring must be designed according to applicable regulatory requirements. Contractors will be responsible for excavation safety.

10.0 BASIS OF REPORT

This report is strictly based on the project description given in Section 3.0. Giles must be notified if the project description or our assumptions are not accurate so that this report can be amended, if needed. This report assumes that a future development will be designed and constructed according to the codes that govern construction at the site.

This report is only based on the estimated subsurface conditions shown on the *Test Boring Logs*. Giles must be notified if the subsurface conditions at the site are known to differ from those shown on the *Test Boring Logs*; this report will likely need to be revised. General comments and limitations of this report are given in the appendix.

This report has been promulgated in accordance with generally accepted professional engineering practices in the field of geotechnical engineering. No other warranty is either expressed or implied.

APPENDIX A

FIGURES AND TEST BORING LOGS

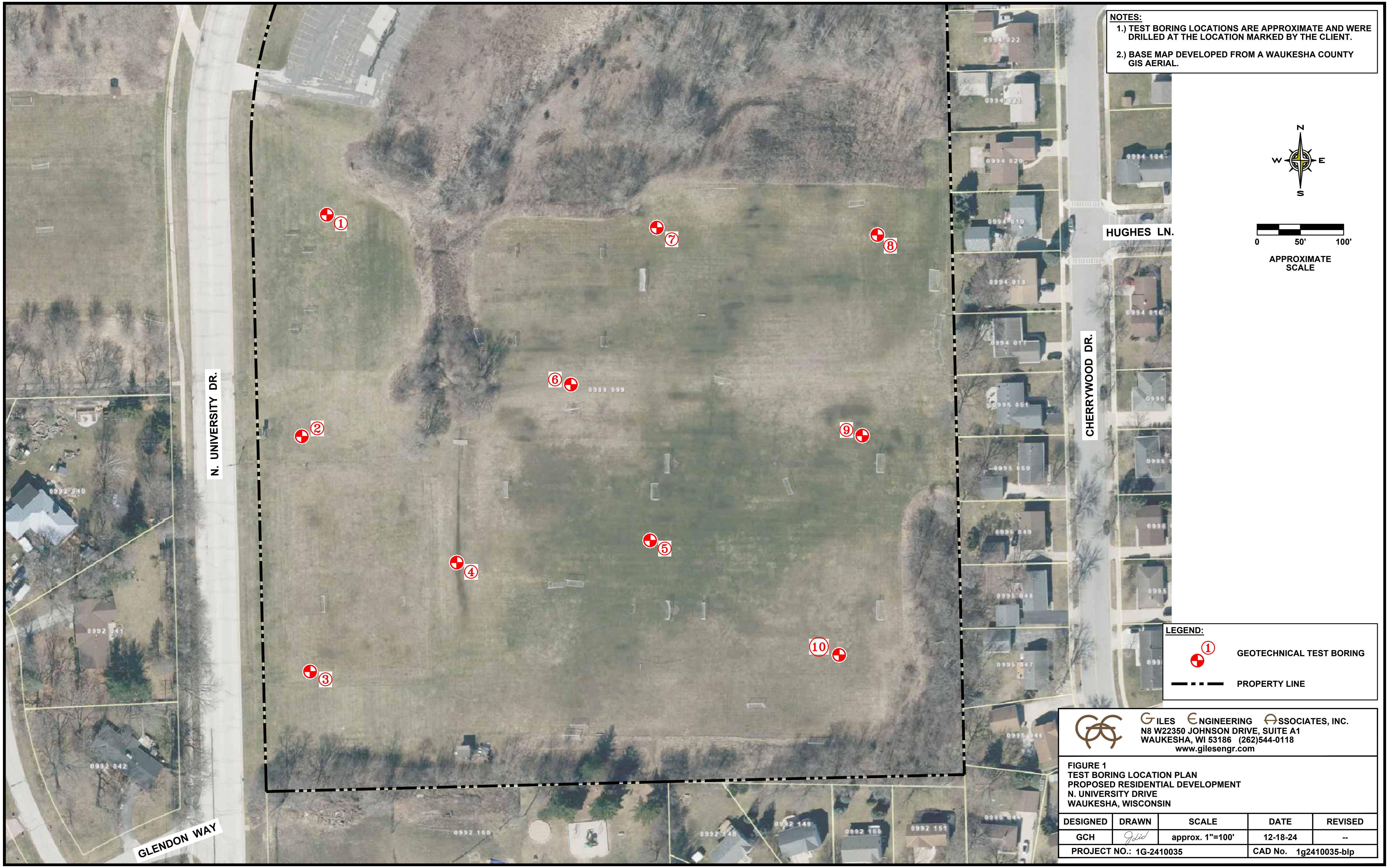
The Test Boring Location Plan contained herein was prepared based upon information supplied by *Giles'* client, or others, along with *Giles'* field measurements and observations. The diagram is presented for conceptual purposes only and is intended to assist the reader in report interpretation.

The Test Boring Logs and related information enclosed herein depict the subsurface (soil and water) conditions encountered at the specific boring locations on the date that the exploration was performed. Subsurface conditions may differ between boring locations and within areas of the site that were not explored with test borings. The subsurface conditions may also change at the boring locations over the passage of time.

NOTES:

1.) TEST BORING LOCATIONS ARE APPROXIMATE AND WERE DRILLED AT THE LOCATION MARKED BY THE CLIENT.

2.) BASE MAP DEVELOPED FROM A WAUKESHA COUNTY GIS AERIAL.



LEGEND:

①	GEOTECHNICAL TEST BORING
— · · —	PROPERTY LINE

GILES ENGINEERING ASSOCIATES, INC.
N8 W22350 JOHNSON DRIVE, SUITE A1
WAUKESHA, WI 53186 (262)544-0118
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FIGURE 1
TEST BORING LOCATION PLAN
PROPOSED RESIDENTIAL DEVELOPMENT
N. UNIVERSITY DRIVE
WAUKESHA, WISCONSIN

DESIGNED	DRAWN	SCALE	DATE	REVISED
GCH		approx. 1"=100'	12-18-24	--
PROJECT NO.: 1G-2410035			CAD No. 1g2410035-blp	

BORING NO. & LOCATION:
1

SURFACE ELEVATION:
948.16 feet

COMPLETION DATE:
12/04/24

FIELD REP:
COLLIN BUCKO

TEST BORING LOG

PROPOSED RESIDENTIAL DEVELOPMENT

N UNIVERSITY DRIVE
WAUKESHA, WISCONSIN

PROJECT NO: 1G-2410035

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
±12" Topsoil: Dark Brown Silty Clay, little Organic Matter-Moist			1-SS	7				16		
Dark Brown lean Clay, trace Organic Matter-Moist			2-SS	13				24		
Gray mottled with Brown Silty Clay, little Sand-Very Moist		945								
	5		3-SS	7	0.6	0.3		28		
			4-SS	10				14		
		940								
Brown Sandy Clay, little Gravel-Moist										
	10		5-SS	13		0.3		10		
		935								
	15		6-SS	24				12		(a)
		930								
Gray Sandy Clay, trace Gravel (Includes Cobbles and Boulders)-Moist										
	20		7-SS	50/3"				9		

Boring Terminated at about 21 feet (EL. 927.16')

	Water Observation Data	Remarks:
∇	Water Encountered During Drilling:	(a) No SPT Sample Recovery, Auger Sample Obtained
▼	Water Level At End of Drilling:	
░	Cave Depth At End of Drilling:	
▼	Water Level After Drilling:	
▓	Cave Depth After Drilling:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION:
2

SURFACE ELEVATION:
946.23 feet

COMPLETION DATE:
12/04/24

FIELD REP:
COLLIN BUCKO

TEST BORING LOG

PROPOSED RESIDENTIAL DEVELOPMENT

N UNIVERSITY DRIVE
WAUKESHA, WISCONSIN

PROJECT NO: 1G-2410035

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
±10" Topsoil: Dark Brown Silty Clay, little Sand, trace Organic Matter-Moist		945	1-SS	11		2.5		24		
Dark Brown lean Clay, trace Organic Matter, Sand and Gravel-Moist			2-SS	14		2.0		32		
Gray mottled with Brown lean Clay-Moist	5		3-SS	6	1.2	1.5		24		
	▽	940								
Gray Silty Clay-Wet			4-SS	8	0.1		0.1	26		
	10		5-SS	6	0.5	0.3		29		
		935								
Brown Silty fine Sand-Moist	15		6-SS	38				14		
		930								
	20		7-SS	23				27		(a)

Boring Terminated at about 21 feet (EL. 925.23')

	Water Observation Data
▽	Water Encountered During Drilling: 6.5 ft.
▼	Water Level At End of Drilling:
░	Cave Depth At End of Drilling:
▼	Water Level After Drilling:
▓	Cave Depth After Drilling:

Remarks:

(a) Poor Sample Recovery

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION:	3
SURFACE ELEVATION:	946.32 feet
COMPLETION DATE:	11/26/24
FIELD REP:	COLLIN BUCKO

TEST BORING LOG

PROPOSED RESIDENTIAL DEVELOPMENT

N UNIVERSITY DRIVE
WAUKESHA, WISCONSIN

PROJECT NO: 1G-2410035

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
±8" Topsoil: Dark Brown Silty Clay, little Sand, trace Organic Matter-Moist		945	1-SS	9						
±4" Crushed Limestone										
Fill: Gray lean Clay, trace Sand-Moist			2-SS	8	1.0	1.8		26		
Dark Brown lean Clay, trace Organic Matter-Moist	5									
Gray mottled with Brown lean Clay-Moist			3-SS	7	0.8	1.0		27		
		940								
Gray Silty fine Sand-Moist			4-SS	11				9		
Brown Sandy Clay, trace Gravel (Includes Cobbles and Boulders)-Moist	10		5-SS	14				13		(a)
		935								

Auger Refusal
Boring Terminated at about 14 feet (EL. 932.32')

	Water Observation Data
∇	Water Encountered During Drilling:
▼	Water Level At End of Drilling:
░	Cave Depth At End of Drilling:
▼	Water Level After Drilling:
▓	Cave Depth After Drilling:

Remarks:

(a) No SPT Sample Recovery, Auger Sample Obtained

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION:
4

SURFACE ELEVATION:
943.04 feet

COMPLETION DATE:
11/26/24

FIELD REP:
COLLIN BUCKO

TEST BORING LOG

PROPOSED RESIDENTIAL DEVELOPMENT

N UNIVERSITY DRIVE
WAUKESHA, WISCONSIN

PROJECT NO: 1G-2410035

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
±16" Topsoil: Dark Brown Silty Clay, trace Organic Matter-Moist			1-SS	5		2.0		26		
Dark Gray lean Clay, trace Organic Matter-Moist		940	2-SS	6		1.5		27		
Gray lean Clay-Moist	5		3-SS	5	1.8	1.5		24		
		935	4-SS	8	0.9	0.5		32		
	∇									
Gray Silty Clay (Includes fine Sand layers)-Moist	10		5-SS	5	0.8	0.5		26		
		930	6-SS	13	2.8	3.0		16		
	15		7-SS	20	2.6	2.5		20		
		925								
Gray Sandy Clay, little Gravel-Wet	20		8-SS	15		0.5		9		

Boring Terminated at about 21 feet (EL. 922.04')

	Water Observation Data	Remarks:
∇	Water Encountered During Drilling: 9 ft.	
▼	Water Level At End of Drilling:	
░	Cave Depth At End of Drilling:	
▼	Water Level After Drilling:	
▓	Cave Depth After Drilling:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: 5	
SURFACE ELEVATION: 944.38 feet	
COMPLETION DATE: 11/26/24	
FIELD REP: COLLIN BUCKO	

TEST BORING LOG

PROPOSED RESIDENTIAL DEVELOPMENT

N UNIVERSITY DRIVE
WAUKESHA, WISCONSIN

PROJECT NO: 1G-2410035

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q$_u$ (tsf)	Q$_p$ (tsf)	Q$_s$ (tsf)	W (%)	PID	NOTES
±4" Topsoil: Dark Brown Silty Clay, little Sand, trace Organic Matter-Moist			1-SS	5						
Fill: Brown Sandy Clay, trace Gravel-Moist										
Gray Silty Clay, trace Sand-Moist			2-SS	6	2.2	2.3		24		
	5	940	3-SS	5	1.7	1.3		22		
Dark Gray Silty Clay-Wet			4-SS	6		0.3		27		
Gray Silty Clay-Moist	10	935	5-SS	4	1.2	0.8		25		
Gray Sandy Clay, little Gravel-Moist	15	930	6-SS	18						(a)
			7-SS	19		0.5		8		
	20	925	8-SS	17	2.9	2.5		9		

Boring Terminated at about 21 feet (EL. 923.38')

	Water Observation Data	Remarks:
⊽	Water Encountered During Drilling:	(a) Poor Sample Recovery
▼	Water Level At End of Drilling:	
░	Cave Depth At End of Drilling:	
▼	Water Level After Drilling:	
▬	Cave Depth After Drilling:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION:
6

SURFACE ELEVATION:
942.62 feet

COMPLETION DATE:
12/04/24

FIELD REP:
COLLIN BUCKO

TEST BORING LOG

PROPOSED RESIDENTIAL DEVELOPMENT

N UNIVERSITY DRIVE
WAUKESHA, WISCONSIN

PROJECT NO: 1G-2410035

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
±10" Topsoil: Dark Brown Silty Clay, little Sand and Organic Matter-Moist **Fill:** Gray mottled with Brown lean Clay, little Sand-Moist			1-SS	7		4.0		20		
Fill: Brown Gravelly fine to medium Sand, little Clay-Moist		940	2-SS	11						
Brown lean Clay-Wet	▽ 5		3-SS	6				28		
		935	4-SS	38				25		(a)
Gray Silty Clay with fine Sand-Wet	10		5-SS	24		0.3		17		
		930								
	15		6-SS	40						(b)
		925								
Gray Sandy Silt with Gravel-Moist	20		7-SS	50/5						

Boring Terminated at about 21 feet (EL. 921.62')

	Water Observation Data
▽	Water Encountered During Drilling: 5 ft.
▼	Water Level At End of Drilling:
░	Cave Depth At End of Drilling:
▼	Water Level After Drilling:
▬	Cave Depth After Drilling:

Remarks:

(a) No SPT Sample Recovery, Auger Sample Obtained
(b) Poor Sample Recovery

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION:
7

SURFACE ELEVATION:
944.05 feet

COMPLETION DATE:
12/04/24

FIELD REP:
COLLIN BUCKO

TEST BORING LOG

PROPOSED RESIDENTIAL DEVELOPMENT

N UNIVERSITY DRIVE
WAUKESHA, WISCONSIN

PROJECT NO: 1G-2410035

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
±12" Topsoil Dark Brown Sandy Silt, little Organic Matter-Moist			1-SS	9		3.3		25		
Gray mottled with Brown lean Clay-Moist			2-SS	6		1.3		27		
		940								
	5		3-SS	9		1.0		27		
			4-SS	12	1.5	1.0		27		
	∇	935								
Gray Sandy Clay, little Gravel-Moist	10		5-SS	24		2.5		9		
Brown Silty Clay-Wet		930								
	15		6-SS	11			0.13	28		
Gray Sandy Clay, little Gravel-Moist		925								
	20		7-SS	26		3.0		8		

Boring Terminated at about 21 feet (EL. 923.05')

	Water Observation Data
∇	Water Encountered During Drilling: 9 ft.
▼	Water Level At End of Drilling:
░	Cave Depth At End of Drilling:
▼	Water Level After Drilling:
▓	Cave Depth After Drilling:

Remarks:

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

TEST BORING LOG

BORING NO. & LOCATION:
8

SURFACE ELEVATION:
944.55 feet

COMPLETION DATE:
11/26/24

FIELD REP:
COLLIN BUCKO

PROPOSED RESIDENTIAL DEVELOPMENT

N UNIVERSITY DRIVE
WAUKESHA, WISCONSIN

PROJECT NO: 1G-2410035

GILES ENGINEERING ASSOCIATES, INC.

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
±12" Topsoil: Dark Brown Silty Clay, little Organic Matter-Moist			1-SS	8		2.5		22		
Gray slightly mottled with Brown lean Clay, trace Sand-Moist			2-SS	11	2.6	1.5		22		
	5	940	3-SS	10	2.2	1.8		28		
Gray Silty Clay, trace Sand-Wet	∇		4-SS	9	1.0	1.3		36		
Gray Silt-Moist	10	935	5-SS	30				29		
Gray lean Clay, little Gravel (Included Cobbles and Boulders)-Moist	15	930	6-SS	62				20		(a)
Gray Sandy Clay with Gravel-Damp	20	925	7-SS	50/5"						

Boring Terminated at about 21 feet (EL. 923.55')

	Water Observation Data
∇	Water Encountered During Drilling: 6.5 ft.
▼	Water Level At End of Drilling:
░	Cave Depth At End of Drilling:
▼	Water Level After Drilling:
▓	Cave Depth After Drilling:

Remarks:

(a) No SPT Sample Recovery, Auger Sample Obtained

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: 9	# TEST BORING LOG	**GILES ENGINEERING ASSOCIATES, INC.**
SURFACE ELEVATION: 944.41 feet	PROPOSED RESIDENTIAL DEVELOPMENT N UNIVERSITY DRIVE WAUKESHA, WISCONSIN PROJECT NO: 1G-2410035	
COMPLETION DATE: 11/26/24		
FIELD REP: COLLIN BUCKO		

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
±8" Topsoil: Dark Brown lean Clay, little Organic Matter, trace Sand-Moist			1-SS	6		1.8		25		
Gray mottled with Brown lean Clay-Moist			2-SS	8	2.3	2.3		23		
	5	940	3-SS	6	1.0	0.8		27		
Dark Gray Silty Clay-Wet	∇		4-SS	7		0.8		26		
	10	935	5-SS	9		0.5		18		
Gray Sandy Clay, little Gravel-Moist										
	15	930	6-SS	9	0.5	1.5		10		
Dark Gray lean Clay, trace Sand and Gravel-Moist										
	20	925	7-SS	41				18		(a)

Boring Terminated at about 21 feet (EL. 923.41')

	Water Observation Data	**Remarks:**
∇	Water Encountered During Drilling: 6.5 ft.	(a) No SPT Sample Recovery, Auger Sample Obtained
▼	Water Level At End of Drilling:	
░	Cave Depth At End of Drilling:	
▼	Water Level After Drilling:	
▓	Cave Depth After Drilling:	

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

BORING NO. & LOCATION: 10	# TEST BORING LOG	**GILES ENGINEERING ASSOCIATES, INC.**
SURFACE ELEVATION: 946.31 feet	PROPOSED RESIDENTIAL DEVELOPMENT N UNIVERSITY DRIVE WAUKESHA, WISCONSIN PROJECT NO: 1G-2410035	
COMPLETION DATE: 11/26/24		
FIELD REP: COLLIN BUCKO		

MATERIAL DESCRIPTION	**Depth (ft)**	**Elevation**	**Sample No. & Type**	**N**	**Q_u (tsf)**	**Q_p (tsf)**	**Q_s (tsf)**	**W (%)**	**PID**	**NOTES**
±8" Topsoil: Dark Brown lean Clay, little Organic Matter, trace Sand-Moist		945	1-SS	8		1.8		9		
Brown Sandy Clay, little Gravel-Moist			2-SS	11	0.8	1.3		10		
	5	940	3-SS	31				10		(a)
			4-SS	17	0.6	0.5		12		
	∇									
Gray lean Clay, trace Sand and Gravel-Moist	10	935	5-SS	11	3.7	4.5+		14		
Gray Sandy Clay, little Gravel-Moist	15	930	6-SS	33				11		(a)
	20		7-SS	8				13		

Boring Terminated at about 21 feet (EL. 925.31')

	Water Observation Data
∇	Water Encountered During Drilling: 9 ft.
▼	Water Level At End of Drilling:
░	Cave Depth At End of Drilling:
▼	Water Level After Drilling:
▓	Cave Depth After Drilling:

Remarks:

(a) No SPT Sample Recovery, Auger Sample Obtained

Changes in strata indicated by the lines are approximate boundary between soil types. The actual transition may be gradual and may vary considerably between test borings. Location of test boring is shown on the Boring Location Plan.

APPENDIX B

FIELD PROCEDURES

The field operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) designation D

420 entitled “Standard Guide for Sampling Rock and Rock” and/or other relevant specifications. Soil samples were preserved and transported to *Giles’* laboratory in general accordance with the procedures recommended by ASTM designation D 4220 entitled “Standard Practice for Preserving and Transporting Soil Samples.” Brief descriptions of the sampling, testing and field procedures commonly performed by *Giles* are provided herein.

GENERAL FIELD PROCEDURES

Test Boring Elevations

The ground surface elevations reported on the Test Boring Logs are referenced to the assumed benchmark shown on the Boring Location Plan (Figure 1). Unless otherwise noted, the elevations were determined with a conventional hand-level and are accurate to within about 1 foot.

Test Boring Locations

The test borings were located on-site based on the existing site features and/or apparent property lines. Dimensions illustrating the approximate boring locations are reported on the Boring Location Plan (Figure 1).

Water Level Measurement

The water levels reported on the Test Boring Logs represent the depth of “free” water encountered during drilling and/or after the drilling tools were removed from the borehole. Water levels measured within a granular (sand and gravel) soil profile are typically indicative of the water table elevation. It is usually not possible to accurately identify the water table elevation with cohesive (clayey) soils, since the rate of seepage is slow. The water table elevation within cohesive soils must therefore be determined over a period of time with groundwater observation wells.

It must be recognized that the water table may fluctuate seasonally and during periods of heavy precipitation. Depending on the subsurface conditions, water may also become perched above the water table, especially during wet periods.

Borehole Backfilling Procedures

Each borehole was backfilled upon completion of the field operations. If potential contamination was encountered, and/or if required by state or local regulations, boreholes were backfilled with an “impervious” material (such as bentonite slurry). Borings that penetrated pavements, sidewalks, etc. were “capped” with Portland Cement concrete, asphaltic concrete, or a similar surface material. It must, however, be recognized that the backfill material may settle, and the surface cap may subside, over a period of time. Further backfilling and/or re-surfacing by *Giles’* client or the property owner may be required.

FIELD SAMPLING AND TESTING PROCEDURES

Auger Sampling (AU)

Soil samples are removed from the auger flights as an auger is withdrawn above the ground surface. Such samples are used to determine general soil types and identify approximate soil stratifications. Auger samples are highly disturbed and are therefore not typically used for geotechnical strength testing.

Split-Barrel Sampling (SS) – (ASTM D-1586)

A split-barrel sampler with a 2-inch outside diameter is driven into the subsoil with a 140-pound hammer free-falling a vertical distance of 30 inches. The summation of hammer-blows required to drive the sampler the final 12-inches of an 18-inch sample interval is defined as the “Standard Penetration Resistance” or N-value is an index of the relative density of granular soils and the comparative consistency of cohesive soils. A soil sample is collected from each SPT interval.

Shelby Tube Sampling (ST) – (ASTM D-1587)

A relatively undisturbed soil sample is collected by hydraulically advancing a thin-walled Shelby Tube sampler into a soil mass. Shelby Tubes have a sharp cutting edge and are commonly 2 to 5 inches in diameter.

Bulk Sample (BS)

A relatively large volume of soils is collected with a shovel or other manually-operated tool. The sample is typically transported to *Giles’* materials laboratory in a sealed bag or bucket.

Dynamic Cone Penetration Test (DC) – (ASTM STP 399)

This test is conducted by driving a 1.5-inch-diameter cone into the subsoil using a 15-pound steel ring (hammer), free-falling a vertical distance of 20 inches. The number of hammer-blows required to drive the cone 1¾ inches is an indication of the soil strength and density, and is defined as “N”. The Dynamic Cone Penetration test is commonly conducted in hand auger borings, test pits and within excavated trenches.

- Continued -

Ring-Lined Barrel Sampling – (ASTM D 3550)

In this procedure, a ring-lined barrel sampler is used to collect soil samples for classification and laboratory testing. This method provides samples that fit directly into laboratory test instruments without additional handling/disturbance.

Sampling and Testing Procedures

The field testing and sampling operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) and/or other relevant specifications. Results of the field testing (i.e. N-values) are reported on the Test Boring Logs. Explanations of the terms and symbols shown on the logs are provided on the appendix enclosure entitled “General Notes”.

APPENDIX C

LABORATORY TESTING AND CLASSIFICATION

The laboratory testing was conducted under the supervision of a geotechnical engineer in accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) and/or other relevant specifications. Brief descriptions of laboratory tests commonly performed by *Giles* are provided herein.

LABORATORY TESTING AND CLASSIFICATION

Photoionization Detector (PID)

In this procedure, soil samples are “scanned” in *Giles’* analytical laboratory using a Photoionization Detector (PID). The instrument is equipped with an 11.7 eV lamp calibrated to a Benzene Standard and is capable of detecting a minute concentration of **certain** Volatile Organic Compound (VOC) vapors, such as those commonly associated with petroleum products and some solvents. Results of the PID analysis are expressed in HNu (manufacturer’s) units rather than actual concentration.

Moisture Content (w) (ASTM D 2216)

Moisture content is defined as the ratio of the weight of water contained within a soil sample to the weight of the dry solids within the sample. Moisture content is expressed as a percentage.

Unconfined Compressive Strength (qu) (ASTM D 2166)

An axial load is applied at a uniform rate to a cylindrical soil sample. The unconfined compressive strength is the maximum stress obtained or the stress when 15% axial strain is reached, whichever occurs first.

Calibrated Penetrometer Resistance (qp)

The small, cylindrical tip of a hand-held penetrometer is pressed into a soil sample to a prescribed depth to measure the soils capacity to resist penetration. This test is used to evaluate unconfined compressive strength.

Vane-Shear Strength (qs)

The blades of a vane are inserted into the flat surface of a soil sample and the vane is rotated until failure occurs. The maximum shear resistance measured immediately prior to failure is taken as the vane-shear strength.

Loss-on-Ignition (ASTM D 2974; Method C)

The Loss-on-Ignition (L.O.I.) test is used to determine the organic content of a soil sample. The procedure is conducted by heating a dry soil sample to 440°C in order to burn-off or “ash” organic matter present within the sample. The L.O.I. value is the ratio of the weight loss due to ignition compared to the initial weight of the dry sample. L.O.I. is expressed as a percentage.

Particle Size Distribution (ASTB D 421, D 422, and D 1140)

This test is performed to determine the distribution of specific particle sizes (diameters) within a soil sample. The distribution of coarse-grained soil particles (sand and gravel) is determined from a “sieve analysis,” which is conducted by passing the sample through a series of nested sieves. The distribution of fine-grained soil particles (silt and clay) is determined from a “hydrometer analysis” which is based on the sedimentation of particles suspended in water.

Consolidation Test (ASTM D 2435)

In this procedure, a series of cumulative vertical loads are applied to a small, laterally confined soil sample. During each load increment, vertical compression (consolidation) of the sample is measured over a period of time. Results of this test are used to estimate settlement and time rate of settlement.

Classification of Samples

Each soil sample was visually-manually classified, based on texture and plasticity, in general accordance with the Unified Soil Classification System (ASTM D-2488-75). The classifications are reported on the Test Boring Logs.

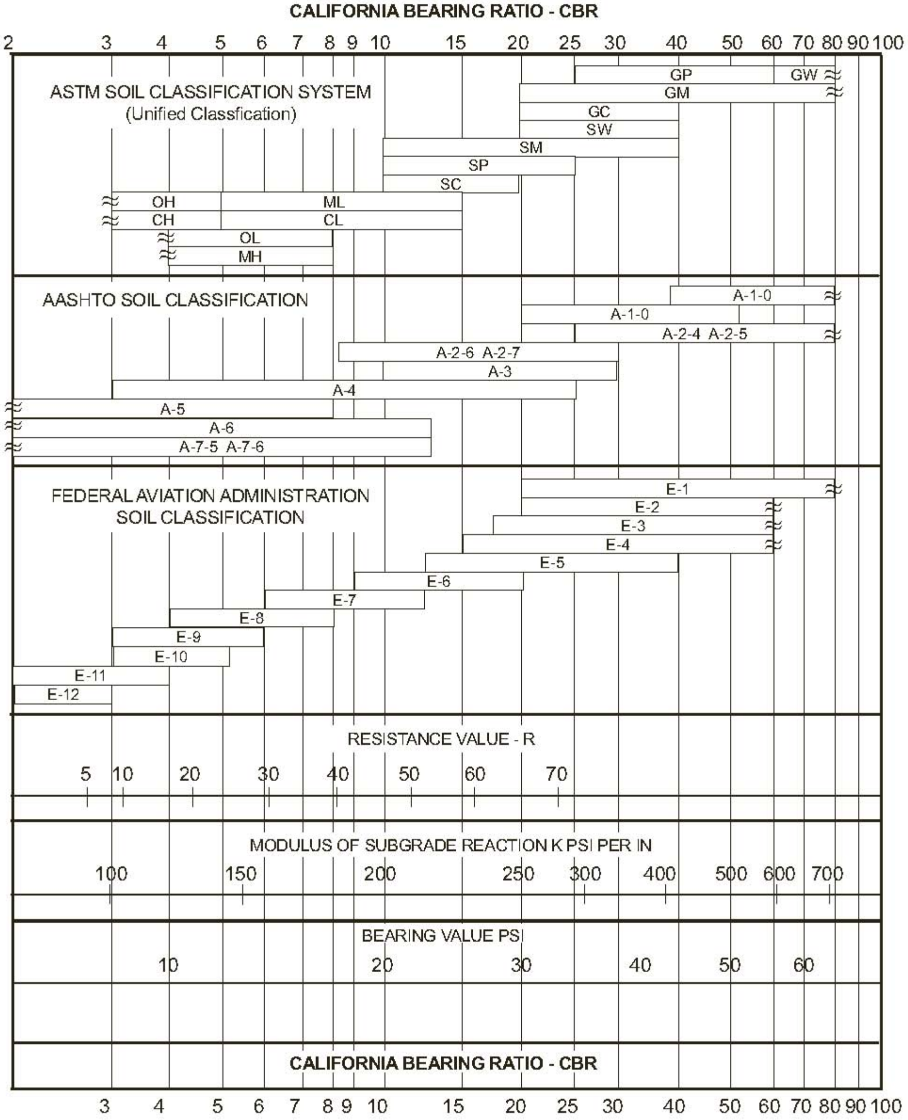
Laboratory Testing

The laboratory testing operations were conducted in general accordance with the procedures recommended by the American Society for Testing and Materials (ASTM) and/or other relevant specifications. Results of the laboratory tests are provided on the Test Boring Logs or other appendix enclosures. Explanation of the terms and symbols used on the logs is provided on the appendix enclosure entitled “General Notes.”

California Bearing Ratio (CBR) Test ASTM D-1833

The CBR test is used for evaluation of a soil subgrade for pavement design. The test consists of measuring the force required for a 3-square-inch cylindrical piston to penetrate 0.1 or 0.2 inch into a compacted soil sample. The result is expressed as a percent of force required to penetrate a standard compacted crushed stone.

Unless a CBR test has been specifically requested by the client, the CBR is estimated from published charts, based on soil classification and strength characteristics. A typical correlation chart is below.



APPENDIX D

GENERAL INFORMATION

AND
IMPORTANT INFORMATION ABOUT THIS GEOTECHNICAL REPORT

GENERAL COMMENTS

The soil samples obtained during the subsurface exploration will be retained for a period of thirty days. If no instructions are received, they will be disposed of at that time.

This report has been prepared exclusively for the client in order to aid in the evaluation of this property and to assist the architects and engineers in the design and preparation of the project plans and specifications. Copies of this report may be provided to contractor(s), with contract documents, to disclose information relative to this project. The report, however, has not been prepared to serve as the plans and specifications for actual construction without the appropriate interpretation by the project architect, structural engineer, and/or civil engineer. Reproduction and distribution of this report must be authorized by the client and *Giles*.

This report has been based on assumed conditions/characteristics of the proposed development where specific information was not available. It is recommended that the architect, civil engineer and structural engineer along with any other design professionals involved in this project carefully review these assumptions to ensure they are consistent with the actual planned development. When discrepancies exist, they should be brought to our attention to ensure they do not affect the conclusions and recommendations provided herein. The project plans and specifications may also be submitted to *Giles* for review to ensure that the geotechnical related conclusions and recommendations provided herein have been correctly interpreted.

The analysis of this site was based on a subsoil profile interpolated from a limited subsurface exploration. If the actual conditions encountered during construction vary from those indicated by the borings, *Giles* must be contacted immediately to determine if the conditions alter the recommendations contained herein.

The conclusions and recommendations presented in this report have been promulgated in accordance with generally accepted professional engineering practices in the field of geotechnical engineering. No other warranty is either expressed or implied.

GUIDE SPECIFICATIONS FOR SUBGRADE AND GRADE PREPARATION FOR FILL, FOUNDATION, FLOOR SLAB AND PAVEMENT SUPPORT; AND SELECTION, PLACEMENT AND COMPACTION OF FILL SOILS USING STANDARD PROCTOR PROCEDURES

1. Construction monitoring and testing of subgrades and grades for fill, foundation, floor slab and pavement; and fill selection, placement and compaction shall be performed by an experienced soils engineer and/or his representatives.

2. All compaction fill, subgrades and grades shall be (a) underlain by suitable bearing material; (b) free of all organic, frozen, or other deleterious material, and (c) observed, tested and approved by qualified engineering personnel representing an experienced soils engineer. Preparation of subgrades after stripping vegetation, organic or other unsuitable materials shall consist of (a) proof-rolling to detect soil, wet yielding soils or other unstable materials that must be undercut, (b) scarifying top 6 to 8 inches, (c) moisture conditioning the soils as required, and (d) recompaction to same minimum in-situ density required for similar materials indicated under Item 5. Note: compaction requirements for pavement subgrade are higher than other areas. Weather and construction equipment may damage compacted fill surface and reworking and retesting may be necessary to assure proper performance.

3. In overexcavation and fill areas, the compacted fill must extend (a) a minimum 1 foot lateral distance beyond the exterior edge of the foundation at bearing grade or pavement subgrade and down to compacted fill subgrade on a maximum 0.5(H):1(V) slope, (b) 1 foot above footing grade outside the building, and (c) to floor subgrade inside the building. Fill shall be placed and compacted on a 5(H):1(V) slope or must be stepped or benched as required to flatten if not specifically approved by qualified personnel under the direction of an experienced soil engineer.

4. The compacted fill materials shall be free of deleterious, organic, or frozen matter, shall contain no chemicals that may result in the material being classified as “contaminated”, and shall be low-expansive with a maximum Liquid Limit (ASTM D-423) and Plasticity Index (ASTM D-424) of 30 and 15, respectively, unless specifically tested and found to have low expansive properties and approved by an experienced soils engineer. The top 12 inches of compacted fill should have a maximum 3-inch-particle diameter and all underlying compacted fill a maximum 6-inch-diameter unless specifically approved by an experienced soils engineer. All fill materials must be tested and approved under the direction of an experienced soils engineer prior to placement. If the fill is to provide non-frost susceptible characteristics, it must be classified as a clean GW, GP, SW or SP per the Unified Soil Classification System (ASTM D-2487).

5. For structural fill depths less than 20 feet, the density of the structural compacted fill and scarified subgrade and grades shall not be less than 95 percent of the maximum dry density as determined by Standard Proctor (ASTM-698) with the exception of the top 12 inches of pavement subgrade which shall have a minimum in-situ density of 100 percent of maximum dry density, or 5 percent higher than underlying fill materials. Where the structural fill depth is greater than 20 feet, the portions below 20 feet should have a minimum in-place density of 100 percent of its maximum dry density of 5 percent greater than the top 20 feet. The moisture content of cohesive soil shall not vary by more than -1 to +3 percent and granular soil ±3 percent of the optimum when placed and compacted or recompacted, unless specifically recommended/approved by the soils engineer monitoring the placement and compaction. Cohesive soils with moderate to high expansion potentials (PI>15) should, however, be placed, compacted and maintained prior to construction at a moisture content 3±1 percent above optimum moisture content to limit further heave. The fill shall be placed in layers with a maximum loose thickness of 8 inches for foundations and 10 inches for floor slabs and pavement, unless specifically approved by the soils engineer taking into consideration the type of materials and compaction equipment being used. The compaction equipment should consist of suitable mechanical equipment specifically designed for soil compaction. Bulldozers or similar tracked vehicles are typically not suitable for compaction.

6. Excavation, filling, subgrade and grade preparation shall be performed in a manner and sequence that will provide drainage at all times and proper control of erosion. Precipitation, springs and seepage water encountered shall be pumped or drained to provide a suitable working platform. Springs or water seepage encountered during grading/foundation construction must be called to the soil engineer’s attention immediately for possible construction procedure revision or inclusion of an underdrain system.

7. Non-structural fill adjacent to structural fill should typically be placed in unison to provide lateral support. Backfill along walls must be placed and compacted with care to ensure excessive unbalanced lateral pressures do not develop. The type of fill material placed adjacent to below-grade walls (i.e. basement walls and retaining walls) must be properly tested and approved by an experienced soils engineer with consideration for the lateral pressure used in the wall design.

8. Whenever, in the opinion of the soils engineer or the Owner’s Representatives, an unstable condition is being created either by cutting or filling, the work shall not proceed into that area until an appropriate geotechnical exploration and analysis has been performed and the grading plan revised, if found necessary.

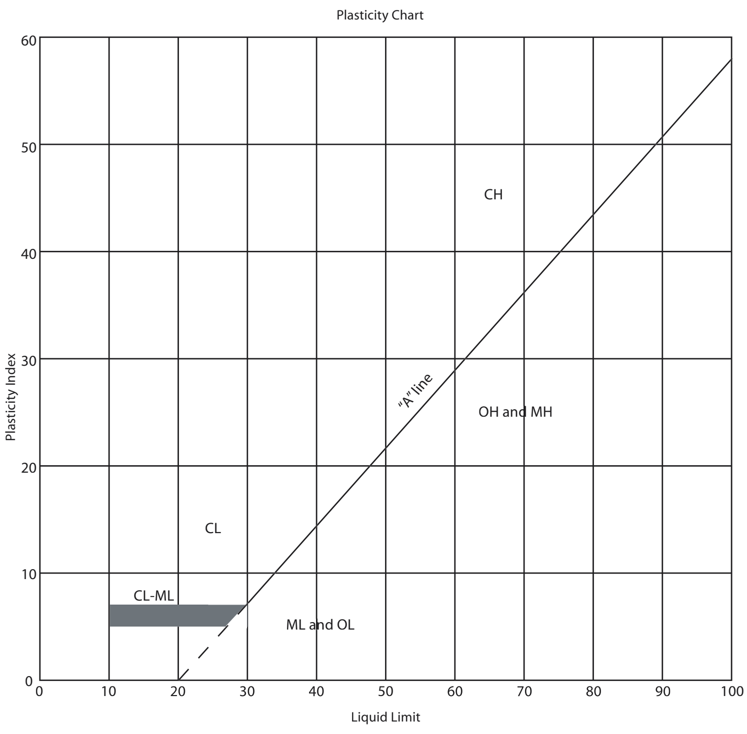
CHARACTERISTICS AND RATINGS OF UNIFIED SOIL SYSTEM CLASSES FOR SOIL CONSTRUCTION *

Class	Compaction Characteristics	Max. Dry Density Standard Proctor (pcf)	Compressibility and Expansion	Drainage and Permeability	Value as an Embankment Material	Value as Subgrade When Not Subject to Frost	Value as Base Course	Value as Temporary Pavement	
								With Dust Palliative	With Bituminous Treatment
GW	Good: tractor, rubber-tired, steel wheel or vibratory roller	125-135	Almost none	Good drainage, pervious	Very stable	Excellent	Good	Fair to poor	Excellent
GP	Good: tractor, rubber-tired, steel wheel or vibratory roller	115-125	Almost none	Good drainage, pervious	Reasonably stable	Excellent to good	Poor to fair	Poor	
GM	Good: rubber-tired or light sheepsfoot roller	120-135	Slight	Poor drainage, semipervious	Reasonably stable	Excellent to good	Fair to poor	Poor	Poor to fair
GC	Good to fair: rubber-tired or sheepsfoot roller	115-130	Slight	Poor drainage, impervious	Reasonably stable	Good	Good to fair **	Excellent	Excellent
SW	Good: tractor, rubber-tired or vibratory roller	110-130	Almost none	Good drainage, pervious	Very stable	Good	Fair to poor	Fair to poor	Good
SP	Good: tractor, rubber-tired or vibratory roller	100-120	Almost none	Good drainage, pervious	Reasonably stable when dense	Good to fair	Poor	Poor	Poor to fair
SM	Good: rubber-tired or sheepsfoot roller	110-125	Slight	Poor drainage, impervious	Reasonably stable when dense	Good to fair	Poor	Poor	Poor to fair
SC	Good to fair: rubber-tired or sheepsfoot roller	105-125	Slight to medium	Poor drainage, impervious	Reasonably stable	Good to fair	Fair to poor	Excellent	Excellent
ML	Good to poor: rubber-tired or sheepsfoot roller	95-120	Slight to medium	Poor drainage, impervious	Poor stability, high density required	Fair to poor	Not suitable	Poor	Poor
CL	Good to fair: sheepsfoot or rubber-tired roller	95-120	Medium	No drainage, impervious	Good stability	Fair to poor	Not suitable	Poor	Poor
OL	Fair to poor: sheepsfoot or rubber-tired roller	80-100	Medium to high	Poor drainage, impervious	Unstable, should not be used	Poor	Not suitable	Not suitable	Not suitable
MH	Fair to poor: sheepsfoot or rubber-tired roller	70-95	High	Poor drainage, impervious	Poor stability, should not be used	Poor	Not suitable	Very poor	Not suitable
CH	Fair to poor: sheepsfoot roller	80-105	Very high	No drainage, impervious	Fair stability, may soften on expansion	Poor to very poor	Not suitable	Very poor	Not suitable
OH	Fair to poor: sheepsfoot roller	65-100	High	No drainage, impervious	Unstable, should not be used	Very poor	Not suitable	Not suitable	Not suitable
Pt	Not suitable		Very high	Fair to poor drainage	Should not be used	Not suitable	Not suitable	Not suitable	Not suitable

* "The Unified Classification: Appendix A - Characteristics of Soil, Groups Pertaining to Roads and Airfields, and Appendix B - Characteristics of Soil Groups Pertaining to Embankments and Foundations," Technical Memorandum 357, U.S. Waterways Ixperiment Station, Vicksburg, 1953.

** Not suitable if subject to frost.

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

Major Divisions			*Group Symbols*		*Typical Names*	*Laboratory Classification Criteria*		
Coarse-grained soils (more than half of material is larger than No. 200 sieve size)	Gravels (More than half of coarse fraction is larger than No. 4 sieve size)	Clean gravels (little or no fines)	GW		Well-graded gravels, gravel-sand mixtures, little or no fines	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: *Borderline* cases requiring dual symbols[b]	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \text{ x } D_{60}}$ between 1 and 3	
			GP		Poorly graded gravels, gravel-sand mixtrues, little or no fines		Not meeting all gradation requirements for GW	
		Gravels with fines (appreciable amount of fines)	GM[a]	d	Silty gravels, gravel-sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Limits plotting within shaded area, above "A" line with P.I. between 4 and 7 are *borderline* cases requiring use of dual symbols
				u				
			GC		Clayey gravels, gravel-sand-clay mixtures		Atterberg limits above "A" line or P.I. greater than 7	
	Sands (More than half of coarse fraction is smaller than No. 4 sieve size)	Clean sands (Little or no fines)	SW		Well-graded sands, gravelly sands, little or no fines		$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} \text{ x } D_{60}}$ between 1 and 3	
			SP		Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for SW	
		Sands with fines (Appreciable amount of fines)	SM[a]	d	Silty sands, sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Limits plotting within shaded area, above "A" line with P.I. between 4 and 7 are *borderline* cases requiring use of dual symbols
				u				
			SC		Clayey sands, sand-clay mixtures		Atterberg limits above "A" line or P.I. greater than 7	
Fine-grained soils (More than half material is smaller than No. 200 sieve size)	Silts and clays (Liquid limit less than 50)		ML		Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity			
			CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays			
			OL		Organic silts and organic silty clays of low plasticity			
	Silts and clays (Liquid limit greater than 50)		MH		Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts			
			CH		Inorganic clays of high plasticity, fat clays			
			OH		Organic clays of medium to high plasticity, organic silts			
	Highly organic soils		Pt		Peat and other highly organic soils			

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[a] Division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is based on Atterberg limits, suffix d used when L.L. is 28 or less and the P.I. is 6 or less; the suffix u is used when L.L. is greater than 28.

[b] Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group sympols. For example GW-GC, well-graded gravel-sand mixture with clay binder.

GENERAL NOTES

SAMPLE IDENTIFICATION

All samples are visually classified in general accordance with the Unified Soil Classification System (ASTM D-2487-75 or D-2488-75)

DESCRIPTIVE TERM (% BY DRY WEIGHT)

Trace:	1-10%
Little:	11-20%
Some:	21-35%
And/Adjective	36-50%

PARTICLE SIZE (DIAMETER)

Boulders:	8 inch and larger
Cobbles:	3 inch to 8 inch
Gravel:	coarse - ¾ to 3 inch
	fine – No. 4 (4.76 mm) to ¾ inch
Sand:	coarse – No. 4 (4.76 mm) to No. 10 (2.0 mm)
	medium – No. 10 (2.0 mm) to No. 40 (0.42 mm)
	fine – No. 40 (0.42 mm) to No. 200 (0.074 mm)
Silt:	No. 200 (0.074 mm) and smaller (non-plastic)
Clay:	No 200 (0.074 mm) and smaller (plastic)

SOIL PROPERTY SYMBOLS

Dd:	Dry Density (pcf)
LL:	Liquid Limit, percent
PL:	Plastic Limit, percent
PI:	Plasticity Index (LL-PL)
LOI:	Loss on Ignition, percent
Gs:	Specific Gravity
K:	Coefficient of Permeability
w:	Moisture content, percent
qp:	Calibrated Penetrometer Resistance, tsf
qs:	Vane-Shear Strength, tsf
qu:	Unconfined Compressive Strength, tsf
qc:	Static Cone Penetrometer Resistance (correlated to Unconfined Compressive Strength, tsf)
PID:	Results of vapor analysis conducted on representative samples utilizing a Photoionization Detector calibrated to a benzene standard. Results expressed in HNU-Units. (BDL=Below Detection Limit)
N:	Penetration Resistance per 12 inch interval, or fraction thereof, for a standard 2 inch O.D. (1⅜ inch I.D.) split spoon sampler driven with a 140 pound weight free-falling 30 inches. Performed in general accordance with Standard Penetration Test Specifications (ASTM D-1586). N in blows per foot equals sum of N-Values where plus sign (+) is shown.
Nc:	Penetration Resistance per 1¾ inches of Dynamic Cone Penetrometer. Approximately equivalent to Standard Penetration Test N-Value in blows per foot.
Nr:	Penetration Resistance per 12 inch interval, or fraction thereof, for California Ring Sampler driven with a 140 pound weight free-falling 30 inches per ASTM D-3550. Not equivalent to Standard Penetration Test N-Value.

DRILLING AND SAMPLING SYMBOLS

SS:	Split-Spoon
ST:	Shelby Tube – 3 inch O.D. (except where noted)
CS:	3 inch O.D. California Ring Sampler
DC:	Dynamic Cone Penetrometer per ASTM Special Technical Publication No. 399
AU:	Auger Sample
DB:	Diamond Bit
CB:	Carbide Bit
WS:	Wash Sample
RB:	Rock-Roller Bit
BS:	Bulk Sample
Note:	Depth intervals for sampling shown on Record of Subsurface Exploration are not indicative of sample recovery, but position where sampling initiated

SOIL STRENGTH CHARACTERISTICS

COHESIVE (CLAYEY) SOILS

COMPARATIVE CONSISTENCY	**BLOWS PER FOOT (N)**	**UNCONFINED COMPRESSIVE STRENGTH (TSF)**
Very Soft	0 - 2	0 - 0.25
Soft	3 - 4	0.25 - 0.50
Medium Stiff	5 – 8	0.50 - 1.00
Stiff	9 – 15	1.00 - 2.00
Very Stiff	16 – 30	2.00 - 4.00
Hard	31+	4.00+

NON-COHESIVE (GRANULAR) SOILS

RELATIVE DENSITY	**BLOWS PER FOOT (N)**
Very Loose	0 - 4
Loose	5 - 10
Firm	11 - 30
Dense	31 - 50
Very Dense	51+

DEGREE OF PLASTICITY	**PI**
None to Slight	0 - 4
Slight	5 - 10
Medium	11 - 30
High to Very High	31+

DEGREE OF EXPANSIVE POTENTIAL	**PI**
Low	0 - 15
Medium	15 - 25
High	25+

GILES ENGINEERING ASSOCIATES, INC.

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.***

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