Geotechnical Engineering Exploration and Analysis

Proposed Multi-Purpose Building
Lakeside Park
Fond du Lac, Wisconsin

Prepared for:

City of Fond du Lac
Fond du Lac, Wisconsin

February 23, 2021
Project No. 1G-2102003

GILES
ENGINEERING ASSOCIATES, INC.

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

- Atlanta, GA
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February 23, 2021

City of Fond du Lac
160 South Macy Street
Fond du Lac, WI 54935

Attention: Mr. Josh Musack
Deputy Procurement Officer

Subject: Geotechnical Engineering Exploration and Analysis
Proposed Multi-Purpose Building
Lakeside Park
Fond du Lac, Wisconsin
Project No. 1G-2102003

Dear Mr. Musack:

As requested, Giles Engineering Associates, Inc. conducted a *Geotechnical Engineering Exploration and Analysis* for the proposed project. The accompanying report describes the services that were performed, and it provides geotechnical-related findings, conclusions, and recommendations that were 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.

Colleen Finley

Colleen M. Finley, E.I.T.
Staff Professional I

D M. Cornale

DAVID M.
CORNALE
E-43336
GREENFIELD
WI

David M. Cornale, P.E.
Sr. Geotechnical Consultant

Distribution: City of Fond du Lac
Attn: Mr. Josh Musack (jmusack@fdl.wi.gov)

Excel Engineering
Attn: Mr. Jason Keifenheim (1 via email: Jason.k@excelengineer.com)
Attn: Mr. Tom Schermerhorn (1 via email: Tom.s@excelengineer.com)

N8 W22350 Johnson Drive • Suite A1 • Waukesha, WI 53186
262/544-0118 • Fax 262/549-5868 • E-Mail milwaukee@gilesengr.com

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GEOTECHNICAL ENGINEERING EXPLORATION AND ANALYSIS

PROPOSED MULTI-PURPOSE BUILDING
LAKESIDE PARK
FOND DU LAC, WISCONSIN
GILES PROJECT NO. 1G-2102003

1.0 SCOPE OF SERVICES

This report provides the results of the *Geotechnical Engineering Exploration and Analysis* that Giles Engineering Associates, Inc. (“Giles”) conducted for the proposed project. The *Geotechnical Engineering Exploration and Analysis* 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 based on our understanding and assumptions about the project. Service areas are briefly described later. Environmental-related consulting services were beyond Giles’ scope for this project.

Geotechnical-related recommendations are provided in this report for design and construction of the foundations and at-grade floor for the proposed building, and pavement for drives and parking areas. Furthermore, site preparation recommendations are included, but those recommendations are only preliminary, as the means and methods of site preparation will depend on factors that were unknown when this report was prepared. Those factors include, but are not limited to, the weather before and during construction, subsurface conditions that are exposed during construction, and final details of the proposed project.

2.0 SITE DESCRIPTION

The subject site is in Lakeside Park, northeast of the intersection of Promen Drive and Lighthouse Drive, in Fond du Lac, Wisconsin. The site is shown on the *Test Boring Location Plan*, enclosed as Figure 1 in Appendix A. During our field services, the site was mostly tree and grass-covered with an asphalt concrete roadway and parking lot. The site is directly south of Lake Winnebago and a harbor and lighthouse are east of the site. Topographically, the site was relatively flat and level. Ground elevations at the test borings (described later) varied between ±El. 99.3 and ±El. 100.7; these elevations are referenced to Giles’ adopted benchmark, shown on the *Test Boring Location Plan*.

3.0 PROJECT DESCRIPTION

Proposed Building

A two-story building is planned to be constructed at the site. The proposed building location is shown on the *Test Boring Location Plan*. It is understood that the building will be constructed from a combination of structural steel, concrete, masonry and glulam timbers with a wood-truss roof system. It is understood that the building is planned to have a ground-bearing floor slab with no below-grade spaces.

The proposed building will assumedly be supported by bearing walls and columns, but the maximum foundation loads were not provided. The maximum foundation loads are, therefore, assumed to be 5,000 pounds per lineal foot (plf) from bearing walls and 40,000 pounds per column. The maximum floor load was not provided, but is assumed to be 100 pounds per square foot (psf). A patio is also planned to surround the proposed building on the north and east sides of the building. The proposed patio material construction is not known. It is assumed that the patio will be constructed using concrete paver units or similar material.

The floor elevation for the proposed building was not provided; therefore, to complete this report it was necessary to assume the floor elevation. This report assumes that the first floor of the building will be at El. 101; referenced to Giles' temporary benchmark, shown on the *Test Boring Location Plan*. Additionally, it is assumed that the patio surface elevation will be at approximately El. 100. Based on the assumed floor and patio elevations, and the existing topography, only minor grading is expected to be necessary to construct the proposed building.

Proposed Pavement Areas

The proposed development will also include the construction of parking areas and drives, as shown on the *Test Boring Location Plan*. It is assumed that asphalt-concrete pavement is planned for the parking areas and drives, except that Portland cement pavement is expected in higher stress areas. Because Giles was not provided with traffic information, the pavement recommendations provided herein are based on arbitrarily assumed traffic conditions. The recommendations also assume that only minor grading (two-foot maximum) will be necessary in future pavement areas.

4.0 GEOTECHNICAL SUBSURFACE EXPLORATION PROGRAM

Eight geotechnical test borings were conducted at the site to explore subsurface conditions. Test Borings 1 and 2 were in the proposed building area and were advanced to ±21 feet below-ground; Test Borings 3 and 4 were in the proposed patio area and advanced to ±11 feet below-ground; Test Borings 5 through 8 were in future pavement areas and were advanced to ±6 feet below-ground. Test boring locations were positioned on-site from the existing roadway, apparent property lines, and other site features, and by estimating right angles. Approximate locations of the test borings are shown on the *Test Boring Location Plan*.

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 labeled at the site for identification. A Standard Penetration Resistance value (N-value) was determined from each SPT. N-values are reported on the *Test Boring Logs*, enclosed in Appendix A, which are records of the test borings.

The boreholes were backfilled upon completion; however, backfill materials 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 others; settlement and heave of backfill materials should be repaired immediately. Giles will not monitor or repair boreholes. Ground elevations at the test borings were determined by differential leveling referenced to Giles' temporary benchmark, shown on the *Test Boring Location Plan*. Test boring elevations are noted on the *Test Boring Logs* and are considered accurate within about one foot.

5.0 GEOTECHNICAL LABORATORY SERVICES

Soil samples that were retained from the test borings were transported to Giles' geotechnical laboratory, where they 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*.

Calibrated penetrometer resistance, unconfined compressive strength (without controlled strain), and moisture content tests were performed on select cohesive samples to evaluate their general engineering properties. In addition, three Loss-On-Ignition (LOI) tests were performed on samples from Test Borings 1, 3, and 4 to determine the organic content of the soil. Test results are on the *Test Boring Logs*. Because the laboratory strength tests were conducted on SPT samples, results of the penetrometer resistance tests are considered to be approximations and were, therefore, used as supplemental information. Laboratory procedures are briefly described in Appendix C.

6.0 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*. The conclusions and recommendations in this report are only based on the estimated conditions.

6.1. Surface Materials

Topsoil fill was at the surface of each test boring and was about 10 to 12 inches thick, except at Test Boring 5 where asphalt-concrete was at the surface. The topsoil fill generally consisted of silty clay and lean clay and typically included little amounts of sand and organic matter. The asphalt-concrete at Test Boring 5 was about 7 inches thick and was underlain by a granular base course.

6.2. Fill Materials

Material classified as fill was beneath the surface materials at each test boring and extended to depths ranging between ±2½ to ±6 feet below-ground, except at Test Borings 6, 7, and 8 where fill material extended to the ±6-foot termination depth. The fill material generally consisted of lean clay with variable amounts of sand, gravel, and organic matter. Granular fill material was also encountered at Test Borings 2 and 5 and consisted of sand (variable gradations) and gravel and sand. The fill material at Test Boring 5 between 1½ and 4 feet below-ground was classified as cinder fill and contained slag, cinders, and ash. Glass and wood debris were also encountered within the fill material. Based on field and laboratory testing, the fill material had relatively low strength characteristics.

6.3. Native Soil

Organic Native Soil: Organic soils were encountered beneath the fill materials at Test Borings 1 through 5 to depths between ±9 and ±10 feet below-grade. The organic soils consisted of organic silt and amorphous peat. Three samples of the organic soils were tested for organic content. The tested organic soils had measured organic contents of 18%, 35%, and 39% percent, based on LOI tests. The organic silt at Test Boring 1 contained silty fine sand lenses. The organic native soils had very low strength characteristics and are considered to be highly compressible.

Inorganic Native Soil: Inorganic native soil was encountered beneath the fill materials at Test Boring 3 between ±6 and ±7½ feet below-ground. Inorganic native soil was below the organic native soil at Test Borings 1 through 4. In general, the native soil consisted of lean clay with up to and estimated little amount of sand. The inorganic native soil had very stiff to stiff comparative consistencies, based on laboratory testing.

7.0 GROUNDWATER CONDITIONS

Water and wet soils were encountered within the test borings at depths ranging between ±2 and ±6 feet below grade at the test borings during drilling. Also, at the time of the geotechnical exploration program, Lake Winnebago had an ice elevation of approximately El. 97.2; referenced to Giles' adopted benchmark. Based on the encountered water levels, the relative moisture content of retained soil samples, and the colorations of retained soil samples, it is estimated that the groundwater table was between ±2 and ±4 feet below-grade at the test borings during our field services, which approximately corresponds to be between ±El. 95 and ±El. 98. However, the site is likely subject to shallower perched-groundwater, where water collects/flows above the groundwater table, especially within existing fill. Perched groundwater could be significant. Groundwater conditions at the site will fluctuate, especially seasonally, with weather events, and with fluctuation of Lake Winnebago.

Giles' estimate of the groundwater conditions at the site is only an approximation based on the free water encountered at the test borings, and the colors and moisture conditions of the retained soil samples. Groundwater conditions could differ from the conditions described above and the water table could be higher than estimated. A more precise estimate of the groundwater conditions could be determined by installing (and monitoring) observation wells at the site. Giles could install and monitor observation wells after receiving authorization to conduct those additional services.

8.0 CONCLUSIONS AND RECOMMENDATIONS

8.1. Site Development Considerations

Foundations and Floor Slabs

Due to intolerable settlements associated with the existing fill and organic native soil that was encountered at the test borings, a spread-footing foundation and ground-bearing slab, bearing upon existing site soils is not recommended for the proposed building. Instead, the building foundation and floor could consist of the following:

- Grade-beams and pile caps, along with a structural floor, supported by a deep foundation system, such as helical piers
- Spread footings and a ground-bearing slab supported by existing soils that are improved by compacted aggregate piers
- Spread footings and a ground-bearing slab supported by new engineered fill used to completely replace the unsuitable existing fill and organic soils to a suitable bearing native lean clay subgrade

Due to construction related issues associated with construction dewatering and soil disposal, it is anticipated that removal and replacement of the unsuitable soils will be less economical, as compared to the use of a deep foundation or ground improvement options. Therefore, recommendations regarding ground improvement using compacted aggregate piers and deep foundation (helical pier) support of the building foundations and floor are provided in this report.

The building may be supported by a deep foundation system consisting of helical piers that extend through the existing fill and organic native soil and bear a sufficient depth into the underlying higher-strength lean clay. For this option, the building ground floor is recommended to be a structural slab supported by helical piers. Geotechnical-related design and installation recommendations for helical piers are provided later in this report. Additionally, the deep foundations for the proposed building could consist of driven pipe piles. Higher vertical and lateral capacities are expected to be possible for a driven pile foundation, which could potentially result in a more economical foundation system for new foundation construction, dependent on the actual building foundation loads; however, deeper borings would need to be drilled in order to provide driven pipe pile recommendations. As an alternative, it is anticipated that the building could be supported by spread footings and a ground-bearing floor slab following suitable ground improvement with compacted aggregate piers. A specialty ground improvement contractor is recommended to be contacted for specific design of ground improvement for building spread footing and ground-bearing floor slab support.

Long-Term Settlement

Long-term settlement of the ground surface should be expected due to primary consolidation and secondary compression of the low-strength soil (secondary compression is compression that occurs independent of load). Even areas of the site that are not raised, but are underlain by low-strength organic soils, will likely undergo long-term settlement due to secondary compression. Settlement could be significant and variable, and will likely continue during the service life of the development. Surface grades might need to be re-established to correct drainage problems that occur due to settlement. Also, sidewalks and pavement might need to be raised and otherwise repaired or replaced due to settlement. Additionally, periodic filling and leveling might be needed to restore grades within the improvement areas.

Utilities

Because of the expected long-term settlement, utility conduits beneath the proposed building are recommended to be hung from the structural floor slab, and are recommended to be fitted with flexible couplings where utilities enter the building.

Methane Considerations

Organic (peat) soils were encountered in the test borings. The lateral extent of the organic soils is unknown, but is anticipated to generally exist throughout the site. Due to the presence of organic soil, a potential for the accumulation of methane or noxious gases within the structure exists at this site. In general, methane production depends upon various conditions including physical composition of the soil, presence of moisture and time of year. Methane will migrate laterally and vertically beneath a site through subsurface “conduits,” such as layer or lenses of permeable soil (sands) or trench backfill, especially during winter months when the site is overlain by frozen soil. Appropriate precautions must be taken during construction due to possible explosive or noxious gases. Evaluation of the property for the presence of methane or noxious gases is recommended.

Patio Design Considerations

Due to the existing fill and underlying compressible soils, some post-construction settlement of the patio is anticipated. Measures that can mitigate the anticipated settlement include the use of a relatively flexible patio material, such as concrete paving units that are placed on a subgrade that is stabilized with an aggregate layer containing a biaxial geogrid. The recommend subbase course layer provided in Section 8.6 for asphalt concrete pavement could be used below a patio area consisting of concrete paver units to assist stabilization of the subgrade and reduce differential movement of the patio. However, even with a geogrid reinforced stabilization layer some settlement of the patio is expected.

8.2. Seismic Design Considerations

A soil Site Class D is recommended for seismic design. By definition, Site Class is based on the average properties of subsurface materials to 100 feet below-ground. Because 100-foot test borings were not requested or authorized, it was necessary to estimate the Site Class based on the test borings, presumed area geology, and the International Building Code.

8.3. Helical-Pier Foundation Recommendations

The proposed building is recommended to be supported by a deep foundation system consisting of helical piers that extend through unsuitable (organic) soil, and bear a sufficient depth into the underlying higher-strength lean clay. The proposed building could be supported by helical piers with interconnected grade-beams and pier caps, or because of the recommended structural floor (described below), the proposed building could be supported by a structural mat/slab foundation supported by appropriately spaced helical piers. It is recommended that the uppermost helix of each pier bear at least 3 feet into suitable-bearing higher-strength lean clay, which was encountered at about 9 and 10 feet below-ground at Test Borings 1 through 4, respectively. Also, to achieve the recommended compression capacities (provided below), the uppermost helix of each helical pier must be at least 13 feet below the finished ground surface, regardless of the depth that suitable soil is encountered. The actual depth/elevation of each uppermost helix must be determined on a pier-by-pier basis, depending on resistance (torque) measured during pier installation. Piers might need to be advanced much deeper than planned. It is recommended that a geotechnical engineer observe the helical-pier installation procedures and confirm that each pier is extended to a sufficient depth.

Estimated maximum allowable compression (downward) capacity for various helical-pier combinations are provided in the table below. The helix diameter increases from the bottom. The estimated maximum allowable compression capacity shown in the following table includes a factor of safety of 2.0. If higher or lower compression capacities are needed, contact Giles for alternate helical-pier configurations. Also, it is recommended that Giles re-evaluate the compression capacities once the helical-pier manufacturer is chosen, since helix area varies based on manufacture. It is important to note that the capacities in the following table are based on the conditions at the actual test boring location; soil conditions likely differ away from the test boring.

TABLE 1
ESTIMATED MAXIMUM ALLOWABLE COMPRESSION CAPACITY (PER HELICAL PIER)

Helix Configuration	Estimated Maximum Allowable Compression Capacity
Pier with a 10-inch and 12-inch helix	17 kips
Pier with an 8-inch, 10-inch, and 12-inch helix	22 kips
Pier with a 10-inch, 12-inch, and 14-inch helix	32 kips

- Helix sizes (8-inch, 10-inch, etc.) represent helix diameter. The smallest helix is at the bottom and helix size increases moving upward.
- The estimated compression capacity assumes that the uppermost helix of each pier will be embedded at least 13 feet and at least 3 feet into suitable-bearing native soil; the actual depth/elevation of each uppermost helix is to be determined on a pier-by-pier basis during installation.

The vertical spacing between helices (of an individual pier) is recommended to be at least three (3) times the diameter of the next lowest helix. The center-to-center spacing between adjacent helical piers is recommended to be at least four (4) times the diameter of the largest helix of that pier or the adjacent pier. If possible, more space should be between the piers. If piers will be closer than recommended, the maximum allowable compression capacity of those piers might need to be reduced and/or alternate helix configurations may be necessary. A structural engineer should specify the locations of helical piers based on load requirements and structural details of the proposed column foundations.

A minimum 48-inch foundation-embedment depth is required by the local building code. Therefore, the bottom of each grade beam and pile cap is recommended to be at least 48 inches below the finished ground grade. Based on the shallow water table, and the assumed floor elevations given above, it is recommended that the bottom of grade beams and pile caps be at or above El. 97. Interior grade-beams and pile caps could be directly below the at-grade structural floor, assuming the building (including any seasonal areas) will be heated and underlying soils will not freeze.

It was not within Giles' scope to select or specify the diameter or wall thickness of the helical-pier shafts. A structural engineer, manufacturer of the helical piers, or helical-pier installation contractor should specify the minimum diameter and wall thickness of each helical-pier shaft to prevent excessive lateral deflection and buckling. Couplings for the helical-pier shafts should also be specified to control lateral deflection and buckling. Alternative methods of increasing resistance to buckling, such as annular grouting in the low-strength soil, may be used. Helix style should be selected based on the expected conditions.

Giles did not evaluate the corrosion potential of site soil. Appropriate precautions should be taken to protect the helical piers against corrosion. Care must be taken not to damage helical-piers during transportation, handling, and/or installation. Damage such as scratches could accelerate corrosion. Corrosion potential of site soil can be provided if requested.

The compression capacity of each pier must be verified in the field based upon pier length, overburden, and installation torque at final depth. Torque must be monitored during pier installation to confirm that the piers have sufficient compression capacity. The measured capacity (determined from torque testing and a selected torque factor) is recommended to be at least 200 percent of the required (allowable) compression capacity. It is recommended that a geotechnical engineer observe the installation procedures and verify that the in-place compression capacity of each pier at least meets the specified measured capacity, or twice the required allowable capacity. The length of each helical pier and torque measured during installation should be recorded for each helical pier.

If it is determined that a pier does not have sufficient compression capacity, the pier should be drilled deeper until the necessary capacity is reached or an appropriate lower capacity assigned to the pier; alternatively, the pier could be removed and a pier with additional and/or larger helices installed in its place. It is recommended that Giles provide supplemental recommendations during construction if compression capacities are not achieved.

It is recommended Giles review the final foundation plans prior to construction to confirm that the recommendations provided in this report have been properly interpreted and to evaluate the overall foundation system that will be used to support the proposed building.

Estimated Helical Pier Foundation Settlement

The post-construction total and differential settlements of a helical pier foundation designed and constructed based on this report are estimated to be less than about 0.75 inch and 0.375 inch, respectively. The post-construction angular distortion is estimated to be less than about 0.0015 inch per inch across a distance of 20 feet or more. Estimated settlements are considered within tolerable limits for the proposed structure provided they are appropriately considered in the structural design and assume that foundation-support soil will be thoroughly tested and approved by a geotechnical engineer during foundation excavation and foundation construction.

8.4. At-Grade Floor Slab Recommendations

The at-grade floor is recommended to be a structural slab supported by helical piers. A minimum 4-inch-thick base course is recommended to be below the structural floor slabs to serve as a capillary break. Base-course material is recommended to consist of free-draining aggregate. A geotechnical engineer should test and approve base-course aggregate before it is placed. Considering the site conditions, geotextile might need to be below the base course. The need for geotextile should be determined, with the assistance of a geotechnical engineer, at the time of construction.

A minimum 10-mil vapor retarder is recommended to be directly above or below the base course throughout all at-grade floor areas. The location (above or below the base course) of the vapor retarder should be specified by the project architect or structural engineer. Abutting vapor retarder sheets are recommended to be overlapped and taped, and must extend to all foundation walls. Vapor retarders are recommended to be in accordance with ASTM E 1745, entitled *Standard Specification for Plastic Water Vapor Retarders Used in Contact with Soil or Granular Fill under Concrete Slabs*, or other relevant documents. If the base course has sharp, angular aggregate, protecting the retarder with geotextile (or by other means) is recommended. A vapor barrier may be required, based on the results of the recommended methane testing. If it determined that a vapor barrier is required for methane mitigation, it may be used to replace the 10-mil vapor retarder.

8.5. Ground-Improvement Alternative

As an alternative to the helical pier foundation system and structural floor discussed above, the existing fill materials and organic native soils throughout the entire building area may be improved through specialized ground-improvement techniques, such as by installing compacted-aggregate piers or stone columns at predetermined locations throughout the building area. 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 5 to 10 feet into suitable native soil, and will be about 15 to 20 feet long, based on the test borings. However, the actual length and spacing of the ground-improvement elements must be determined by the ground-improvement contractor.

If the entire building area is properly improved through ground improvement, it is expected that the building could be supported by a spread-footing foundation and the at-grade floor of the building could be a ground-bearing concrete slab. For budgeting purposes, with proper ground improvement, it is expected that spread footings could be designed using a maximum, net, allowable bearing capacity in the range of about 3,000 to 5,000 psf, but the ground-improvement contractor must provide the actual bearing capacity for foundation design. A ground improvement contractor should be contacted to further evaluate the feasibility of using ground improvement for building foundation and floor support.

8.6. Pavement Recommendations

Giles was not given traffic information for pavement design; therefore, recommendations for light-duty pavement are provided below and are based on an arbitrarily assumed traffic condition consisting of five 18-kip Equivalent Single Axle Loads (ESALs) per day. The recommended pavement sections are only intended for use in light-duty areas subject to passenger vehicles with infrequent traffic from heavier vehicles, due to occasional deliveries and weekly removal of refuse and recyclables. The recommended pavement sections assume no increase in traffic volume and no changes in vehicle type or traffic pattern. Also, it is assumed that the ESALs noted above will be in one direction for each lane.

It is critical that the project owner, developer, civil engineer, and other design professionals involved with the project confirm that the ESALs noted above are appropriate for the expected traffic conditions, vehicle types, and axle loadings. If requested, Giles can provide supplemental pavement recommendations based on other traffic conditions, vehicle types, and axle loads. The recommended pavement sections could underperform or fail prematurely if the design ESALs are exceeded.

It was not within Giles' scope to conduct California Bearing Ratio (CBR) testing (used to determine soil support parameters for pavement design) on pavement support materials; therefore, to give pavement recommendations, it was necessary to assume the CBR value. Based on the test borings, the recommended pavement sections were developed based on a lean clay subgrade with an assumed field CBR value of 2.5 and a *Modulus of Subgrade Reaction* (K_{V1}) value of 50 psi/in. Engineered fill that is placed in proposed pavement areas is recommended to have a field CBR value equal to or greater than 2.5, and the fill is recommended to be placed and compacted per the recommendations of this report.

As shown in the following tables, and because of the existing fill and organic native soil, a geogrid is recommended to be at the bottom of the pavement subbase, which will also serve as a stabilization layer; geogrid is recommended to be installed in accordance with the manufacturer's recommendations. **During subgrade preparation (discussed later), care must be taken not to unnecessarily over-excavate below the planned pavement subgrade, as the geogrid and relatively thick base course / stabilization layer are intended to account for marginal subgrade conditions. Also, it is recommended that a geotechnical engineer evaluate the pavement subgrade during construction to determine if the base course or subbase should be thicker than shown in the following tables.**

The geogrid is recommended to be placed at the pavement subgrade (bottom of the subbase). A minimum overlap of 16 inches is recommended for adjacent geogrids. Geogrids are recommended to be installed in accordance with WisDOT and manufacturer guidelines.

Asphalt-Concrete Pavement

The following table shows the recommended thicknesses for asphalt-concrete pavement with an aggregate base-course. State specifications are also included in the table. The recommended pavement section is based on the traffic condition described above.

TABLE 2
RECOMMENDED HMA PAVEMENT SECTION

Materials	Pavement Thickness	Wisconsin DOT Standard Specifications
Hot-Mix Asphalt Surface Course	1.5 inches	Section 460
Hot-Mix Asphalt Binder Course	2.5 inches	Section 460
Dense-Graded Aggregate Base Course	6.0 inches	Section 305, 1¼-inch Crushed Stone
Subbase Course	10.0 inches	Section 305, ¾ or 1¼-inch Crushed Stone or Crushed Gravel
Geogrid	Biaxial Type 2 (BX1200); or equivalent geogrid approved by Giles; placed at bottom of subbase	Section 645

Portland Cement Concrete

The following table provides the recommended PCC thicknesses for the traffic conditions given above. The recommended thicknesses assume that the pavement subgrade will be properly prepared.

TABLE 3
RECOMMENDED PORTLAND CEMENT CONCRETE PAVEMENT SECTION

Materials	Pavement Thickness	Wisconsin DOT Standard Specifications
Portland Cement Concrete	6.0 inches	Section 460
Dense-Graded Aggregate Base Course	12.0 inches (Minimum)	Section 305, 1¼-inch Crushed Stone
Geogrid	Biaxial Type 2 (BX1200); or equivalent geogrid approved by Giles; placed at bottom of base course	Section 645

It is recommended that PCC pavement have load-transfer reinforcement, where appropriate. Control-joint spacing should be determined in accordance with the current ACI code. Expansion joints should be provided where pavement abuts fixed objects, such as the building and light poles. It is recommended and assumed that a civil engineer will provide specific recommendations for concrete pavement, including reinforcing details and control-joint spacing. The 28-day compressive strength of concrete is recommended to be at least 4,000 psi and the concrete should be properly air-entrained. Materials and construction procedures for concrete pavement and the aggregate base are recommended to be in accordance with Wisconsin DOT specifications.

General Pavement Considerations

The pavement recommendations assume that the subgrade will be prepared per this report, the base course will be properly drained, and a geotechnical engineer will observe and monitor pavement construction. Pavement was designed based on a twenty-year design period. Pavement maintenance along with a major rehabilitation after about 8 to 10 years should be expected. Local codes may require specific testing to determine soil support characteristics and/or minimum pavement section thickness might be required.

8.7. Generalized Site Preparation Recommendations

This section deals with site preparation, including preparation of floor slab, pavement, and engineered fill areas. The means and methods of site preparation will greatly depend on the weather conditions before and during construction, the subsurface conditions that are exposed during earthwork operations, and the finalized details of the proposed development. Therefore, only generalized site preparation recommendations are given. In addition to being generalized, the following site preparation recommendations are abbreviated; the *Guide Specifications* in Appendix D give further recommendations. The *Guide Specifications* should be read along with this section. Also, the *Guide Specifications* are recommended to be used as an aid to develop the project specifications.

Subgrade Evaluation and Fill Placement

The exposed subgrade within all development areas of the site is recommended to be evaluated by a geotechnical engineer after the recommended removal and stripping, and once the site is cut (lowered) as needed. The means and methods of evaluating the subgrade should be determined by the geotechnical engineer based on the site conditions. Possibly the subgrade could be evaluated by proof-rolling with a fully-loaded tandem-axle dump truck (or other suitable construction equipment), which is used to locate unstable areas based on subgrade deflection caused by the wheel loads of the proof-roll equipment. Areas that cannot be proof-rolled are recommended to be evaluated by a geotechnical engineer using appropriate means and methods.

Considering the existing fill and organic native soil that was encountered at the test borings, it is expected that unstable soil will be encountered during the recommended subgrade evaluation. Extensive subgrade improvement will likely be necessary to develop a stable subgrade, at least in some areas. Improvement methods might need to extend several feet below-ground, depending on the conditions that are encountered. However, care must be taken not to unnecessarily over-excavate below the planned subgrade. Areas requiring improvement should be defined during construction with the assistance of a geotechnical engineer. Also, specific improvement methods should be determined during construction on an area-by-area basis, depending on the site conditions and results of the recommended subgrade evaluation.

The development area is recommended to be raised, where necessary, to the planned finished grade with engineered fill immediately after the subgrade is confirmed to be stable and suitable to support the proposed site improvements. Engineered fill is recommended to be placed in uniform, relatively thin layers (lifts). Each layer of engineered fill is recommended to be compacted to at least 95 percent of the fill material's maximum dry density determined from the Standard Proctor compaction test (ASTM D698). As an exception, the in-place dry density of engineered fill within one foot of the pavement subgrade is recommended to be compacted to at least 100 percent of the fill's maximum dry density. The water content of fill material is recommended to be uniform and within a narrow range of the optimum moisture content, also determined by the Standard Proctor compaction test. Item Nos. 4 and 5 of the *Guide Specifications* give move information pertaining to selection and compaction of engineered fill.

Engineered fill that does not meet the density and water content requirements is recommended to be replaced with new fill, or scarified to a sufficient depth (likely 6 to 12 inches, or more), moisture-conditioned, and compacted to the required density. A subsequent lift of fill should only be placed after a geotechnical engineer confirms that the previous lift was properly placed and compacted. Subgrade soil will likely need to be recompacted immediately before construction since equipment traffic and adverse weather may reduce soil stability.

Vibratory Compaction

Because of the shallow groundwater conditions, extreme caution is recommended to be taken when using vibratory compaction equipment at the site. Vibratory compaction could cause soils to become unstable; therefore, in some cases, it might be necessary to use static compaction equipment.

Use of Site Soil as Engineered Fill

Site soil that does not contain adverse organic content or other deleterious materials, as noted in the *Guide Specifications*, could be used as engineered fill. However, site soil will likely need to be moisture conditioned (uniformly moistened or dried) prior to being used as engineered fill. If construction is during adverse weather (discussed in the following section), drying site soil will likely not be feasible. In that case, fill material might need to be imported to the site. Additional recommendations regarding fill selection, placement, and compaction are given in the *Guide Specifications*.

8.8. Generalized Construction Considerations

Adverse Weather

Site soil is moisture sensitive and will 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 that becomes unstable 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 depends on weather, bids for site preparation, and other earthwork activities, should consider the time of year that construction will be conducted.

To protect soil from adverse weather, the site surface is recommended to be smoothly graded and contoured during construction to divert surface water away from construction areas. Contoured subgrades are recommended to be rolled with a smooth-drum, non-vibratory compactor, before precipitation, to “seal” the surface. Furthermore, construction traffic should be restricted to certain aggregate-covered areas to control traffic-related soil disturbance. Foundation, floor slab, and pavement construction should begin immediately after suitable support is confirmed.

Dewatering

Construction dewatering and groundwater control might be necessary. 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 shallow excavations. Multiple sump pumps might be necessary. Excavated sump pits should be fully lined with geotextile and filled with free-draining aggregate, such as crushed stone that meets the gradation requirements of ASTM No. 57 aggregate. Specialized dewatering might be necessary to dewater excavations that extend below the water table. It is recommended that a geotechnical engineer monitor and approve dewatering. Improper dewatering could cause support-related problems at the site and at neighboring properties.

Excavation Stability

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

Existing Fill Considerations

Existing fill materials consisting of cinders, slag and ash were encountered in Test Boring 5. Glass and wood debris was also encountered within the existing fill soils in other test borings. Therefore, special handling and disposal requirements may be necessary for some of the site soils. Questionable materials, where encountered, are recommended to be evaluated by a geotechnical engineer to determine if removal and replacement with engineered fill is necessary. Disposal of unsuitable or other excavated material should be in accordance with local, state and federal regulations for the material type. Alteration to the recommendations of this report might be needed, if conditions different than those noted on the *Test Boring Logs* are revealed.

8.9. Recommended Construction Materials Testing Services

This report was prepared assuming that a geotechnical engineer will perform Construction Materials Testing ("CMT") services during construction of the proposed development. It might be necessary for Giles to provide supplemental geotechnical recommendations based on the results of CMT services and specific details of the project not known at this time.

9.0 BASIS OF REPORT

This report is strictly based on the project description given earlier in this report. Giles must be notified if any part of the project description or our assumptions are not accurate so that this report can be amended, if needed. This report is based on the assumption that the facility will be designed and constructed according to the codes that govern construction at the site.

The conclusions and recommendations in this report are based on estimated subsurface conditions as shown on the *Test Boring Logs*. Giles must be notified if the subsurface conditions that are encountered during construction of the proposed development 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.

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.

1G-2102003Report/21Geo01/cmf

APPENDIX A

FIGURES AND TEST BORING LOGS

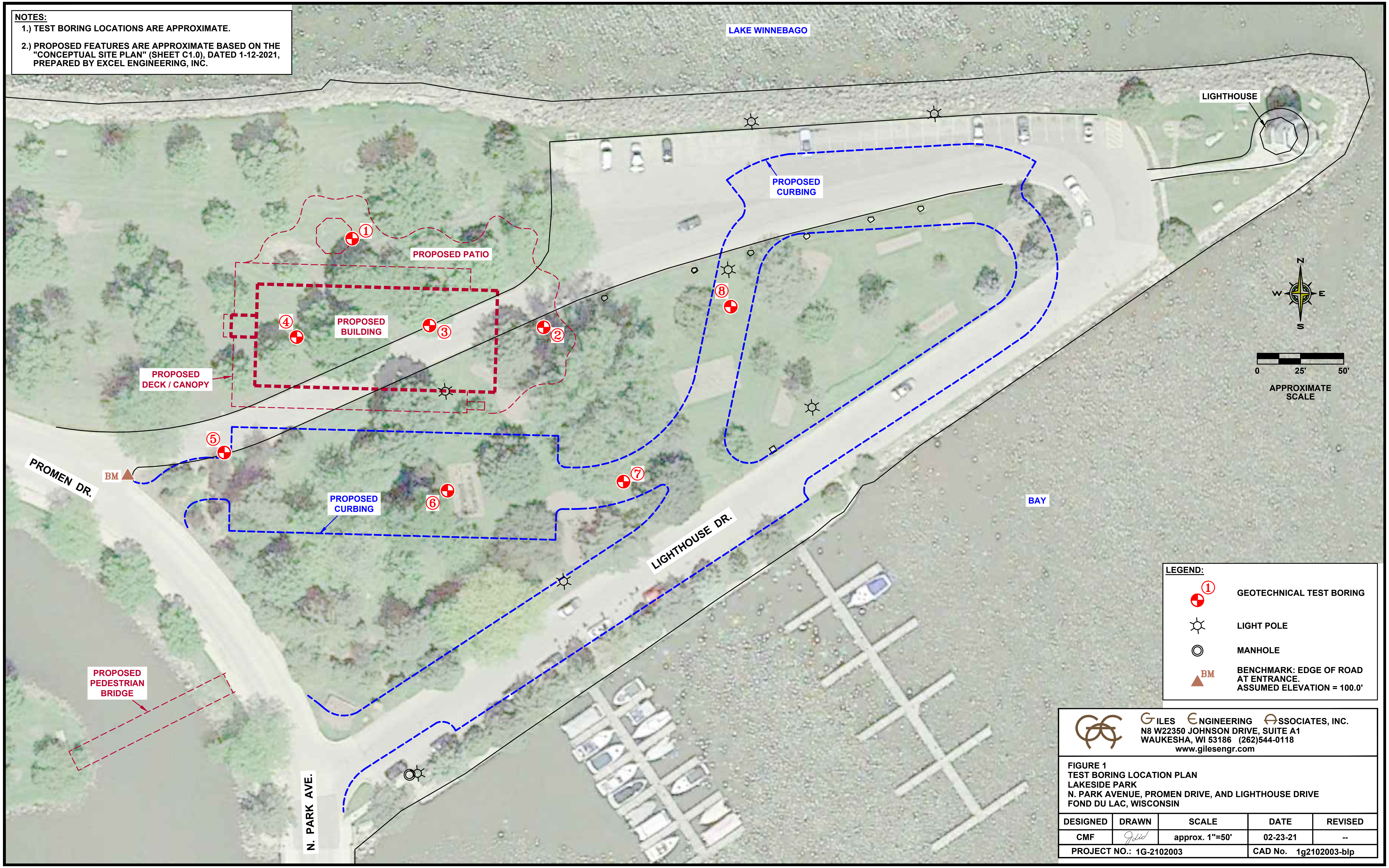
The Test Boring Location Plan contained herein was prepared based upon information supplied by *Giles'* client, or others, along with *Giles'* field measurements and observations. The diagram is presented for conceptual purposes only and is intended to assist the reader in report interpretation.

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.

2.) PROPOSED FEATURES ARE APPROXIMATE BASED ON THE "CONCEPTUAL SITE PLAN" (SHEET C1.0), DATED 1-12-2021, PREPARED BY EXCEL ENGINEERING, INC.



LEGEND:

①	GEOTECHNICAL TEST BORING
☼	LIGHT POLE
◎	MANHOLE
▲ BM	BENCHMARK: EDGE OF ROAD AT ENTRANCE. ASSUMED ELEVATION = 100.0'

GILES ENGINEERING ASSOCIATES, INC.
N8 W22350 JOHNSON DRIVE, SUITE A1
WAUKESHA, WI 53186 (262)544-0118
www.gilesengr.com

FIGURE 1
TEST BORING LOCATION PLAN
LAKESIDE PARK
N. PARK AVENUE, PROMEN DRIVE, AND LIGHTHOUSE DRIVE
FOND DU LAC, WISCONSIN

DESIGNED	DRAWN	SCALE	DATE	REVISED
CMF		approx. 1"=50'	02-23-21	--

PROJECT NO.: 1G-2102003
CAD No. 1g2102003-blp

BORING NO. & LOCATION: 1	# TEST BORING LOG
SURFACE ELEVATION: 99.3 feet	LAKESIDE PARK MULTI-PURPOSE BUILDING
COMPLETION DATE: 02/16/21	N. PARK AVENUE AND LIGHTHOUSE DRIVE FOND DU LAC, WISCONSIN
FIELD REP: KEITH FLOWERS	PROJECT NO: 1G-2102003

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 Fill: Dark Brown lean Clay, little Sand and Organic Matter-Moist			1-AU					28		
Fill: Brown lean Clay, little Sand (Includes Glass)-Moist			2-SS	4				23		(a)
	▽	95								
Black and Gray Organic Silt with Silty fine Sand lenses-Moist to Wet	5		3-SS	4				81		(a)
			4-SS	5				117		LOI=18%
Gray-Brown lean Clay, trace to little Sand-Moist	10	90	5-SS	8	4.5	4.3		24		
	15	85	6-SS	17	3.6	3.5		24		
	20	80	7-SS	10	3.0	3.0		26		

Boring Terminated at about 21 feet (EL. 78.3')

	Water Observation Data	Remarks:
▽	Water Encountered During Drilling: 4 ft.	(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: 2	# TEST BORING LOG	**GILES ENGINEERING ASSOCIATES, INC.**
SURFACE ELEVATION: 100.3 feet	LAKESIDE PARK MULTI-PURPOSE BUILDING	
COMPLETION DATE: 02/16/21	N. PARK AVENUE AND LIGHTHOUSE DRIVE FOND DU LAC, WISCONSIN	
FIELD REP: KEITH FLOWERS	PROJECT NO: 1G-2102003	

MATERIAL DESCRIPTION	**Depth (ft)**	**Elevation**	**Sample No. & Type**	**N**	**Q_u (tsf)**	**Q_p (tsf)**	**Q_s (tsf)**	**W (%)**	**PID**	**NOTES**
±12" Topsoil Fill: Black Silty Clay, little Sand and Organic Matter-Moist		100	1-SS	28*				21		
Fill: Gray-Brown and Red-Brown lean Clay, little Sand and Gravel-Moist			2-SS	6				27		(a)
Fill: Brown Silty fine to coarse Sand with lean Clay lenses (Includes Glass and Wood Debris)-Moist to Wet	▽ 5	95	3-SS	4						(b)
Dark Gray Organic Silt-Wet			4-SS	6				123		(b)
Gray-Brown lean Clay, trace to little Sand-Moist	10	90	5-SS	7		3.5		24		
	15	85	6-SS	24		3.3		22		
	20	80	7-SS	12	2.6	2.5		27		

Boring Terminated at about 21 feet (EL. 79.3')

	Water Observation Data	**Remarks:**
▽	Water Encountered During Drilling: 5 ft.	*N-Value Affected by Frost (a) No Split Spoon Recovery; Auger Sample Obtained (b) Poor Sample Recovery
▼	Water Level At End of Drilling:	
░	Cave Depth At End of Drilling:	
▼	Water Level After Drilling:	
▒	Cave Depth After Drilling:	

BORING NO. & LOCATION: 3	# TEST BORING LOG	**GILES ENGINEERING ASSOCIATES, INC.**
SURFACE ELEVATION: 99.8 feet	LAKESIDE PARK MULTI-PURPOSE BUILDING	
COMPLETION DATE: 02/16/21	N. PARK AVENUE AND LIGHTHOUSE DRIVE FOND DU LAC, WISCONSIN	
FIELD REP: KEITH FLOWERS	PROJECT NO: 1G-2102003	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
±12" Topsoil Fill: Black Silty Clay, little Sand and Organic Matter-Moist			1-AU							
Fill: Red-Brown lean Clay, little Sand, trace Organic Matter (Includes Dark Gray fine Sand lenses and Wood)-Moist			2-SS	8		1.5		29		(a)
	5	95	3-SS	3				40		(a)
Light Brown fine Sand-Wet	▽									
Black amorphous Peat-Moist			4-SS	5				197		LOI=35%
Gray-Brown lean Clay, little Sand-Moist	10	90	5-SS	5		3.0		24		

Boring Terminated at about 11 feet (EL. 88.8')

	Water Observation Data	**Remarks:**
▽	Water Encountered During Drilling: 6 ft.	(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:
4

SURFACE ELEVATION:
99.6 feet

COMPLETION DATE:
02/16/21

FIELD REP:
KEITH FLOWERS

TEST BORING LOG

LAKESIDE PARK MULTI-PURPOSE BUILDING

N. PARK AVENUE AND LIGHTHOUSE DRIVE
FOND DU LAC, WISCONSIN

PROJECT NO: 1G-2102003

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 Fill: Black Silty Clay, little Sand and Organic Matter-Moist			1-AU					37		
Fill: Brown and Black lean Clay, little Sand, trace Organic Matter-Moist			2-SS	6		2.2		29		(a)
	▽									
	5	95	3-SS	4				160		
Dark Gray Organic Silt, little fine Sand, trace Shell-Moist to Wet										
Black amorphous Peat-Moist			4-SS	4				218		LOI=39%
	10	90								
Gray-Brown lean Clay, trace Sand-Moist			5-SS	7		4.5+		23		

Boring Terminated at about 11 feet (EL. 88.6')

	Water Observation Data	Remarks:
▽	Water Encountered During Drilling: 4 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:	99.9 feet
COMPLETION DATE:	02/16/21
FIELD REP:	KEITH FLOWERS

TEST BORING LOG

LAKESIDE PARK MULTI-PURPOSE BUILDING

N. PARK AVENUE AND LIGHTHOUSE DRIVE
FOND DU LAC, WISCONSIN

PROJECT NO: 1G-2102003

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**
±7" Asphalt Concrete										
Base Course: Brown fine to coarse Sand and Gravel, little Silt-Moist			1-AU							
Cinder Fill: Black Gravel and Sand, little Silt (Slag, Cinder and Ash)-Moist			2-SS	6				30		
Gray Silty fine Sand-Wet	▽									
Black amorphous Peat-Moist	5	95	3-SS	4						

Boring Terminated at about 6 feet (EL. 93.9')

	Water Observation Data	**Remarks:**
▽	Water Encountered During Drilling: 4 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:
6

SURFACE ELEVATION:
100.4 feet

COMPLETION DATE:
02/16/21

FIELD REP:
KEITH FLOWERS

TEST BORING LOG

LAKESIDE PARK MULTI-PURPOSE BUILDING

N. PARK AVENUE AND LIGHTHOUSE DRIVE
FOND DU LAC, WISCONSIN

PROJECT NO: 1G-2102003

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 Fill: Dark Brown Silty Clay, little Sand and Organic Matter-Moist		100	1-AU					37		
Fill: Brown lean Clay, little Sand, trace to little Organic Matter-Moist			2-SS	8		2.0		24		
	5	95	3-SS	3				38		(a)

Boring Terminated at about 6 feet (EL. 94.4')

	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:
7

SURFACE ELEVATION:
100.7 feet

COMPLETION DATE:
02/16/21

FIELD REP:
KEITH FLOWERS

TEST BORING LOG

LAKESIDE PARK MULTI-PURPOSE BUILDING

N. PARK AVENUE AND LIGHTHOUSE DRIVE
FOND DU LAC, WISCONSIN

PROJECT NO: 1G-2102003

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 Fill: Black Silty Clay, little Sand and Organic Matter-Moist		100	1-AU					52		
Fill: Brown lean Clay, little Sand, trace Organic Matter-Moist			2-SS	8				21		
	5	95	3-SS	6				28		

Boring Terminated at about 6 feet (EL. 94.7')

	Water Observation Data	Remarks:
▽	Water Encountered During Drilling:	
▼	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: 8	# TEST BORING LOG	GILES ENGINEERING ASSOCIATES, INC.
SURFACE ELEVATION: 100.6 feet	LAKESIDE PARK MULTI-PURPOSE BUILDING	
COMPLETION DATE: 02/16/21	N. PARK AVENUE AND LIGHTHOUSE DRIVE FOND DU LAC, WISCONSIN	
FIELD REP: KEITH FLOWERS	PROJECT NO: 1G-2102003	

MATERIAL DESCRIPTION	Depth (ft)	Elevation	Sample No. & Type	N	Q_u (tsf)	Q_p (tsf)	Q_s (tsf)	W (%)	PID	NOTES
±12" Topsoil Fill: Dark Gray-Brown Silty Clay, little Sand and Organic Matter-Moist		100	1-AU							
Fill: Gray-Brown lean Clay, little fine Sand, trace Organic Matter-Wet			2-SS	6						
	5	95	3-SS	5						

Boring Terminated at about 6 feet (EL. 94.6')

	Water Observation Data	Remarks:
▽	Water Encountered During Drilling:	
▼	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.

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.

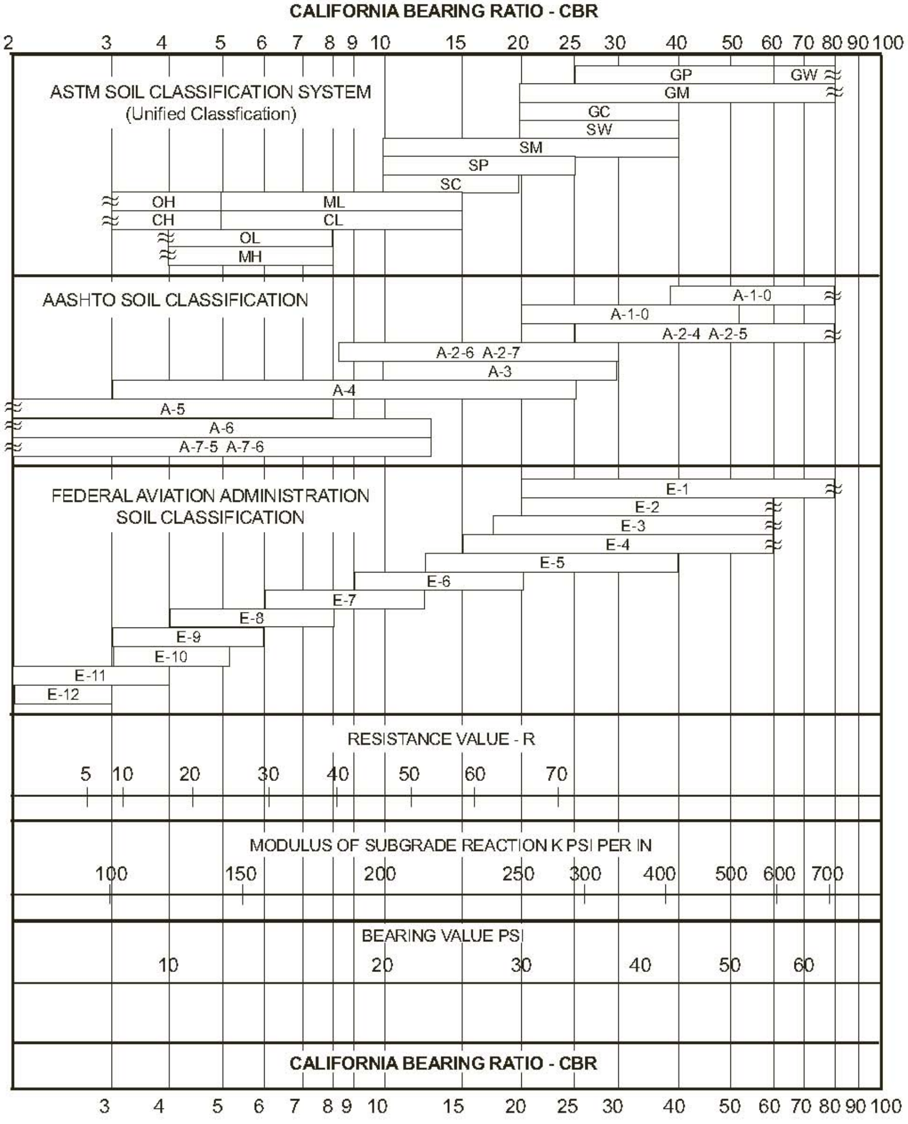
Laboratory Testing

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
YOUR 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.

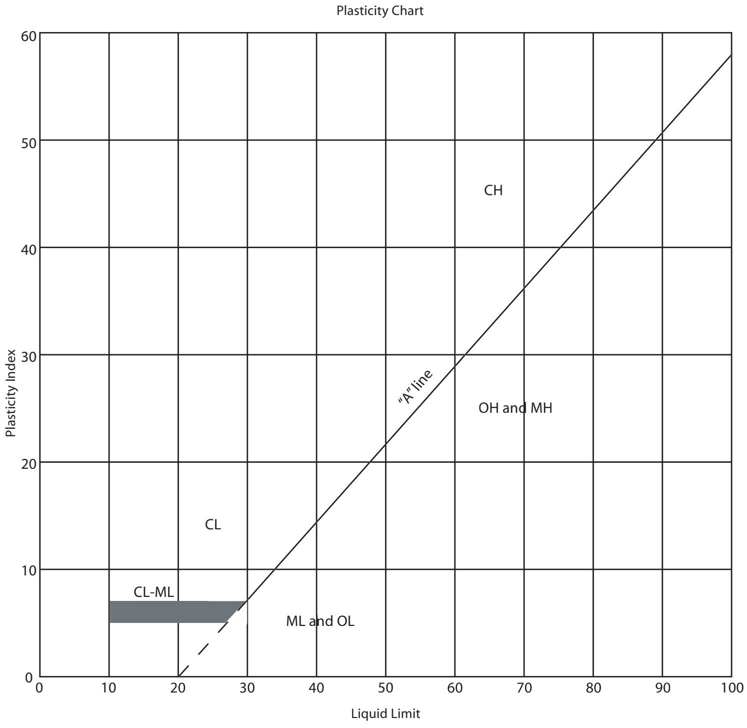
CHARACTERISTICS AND RATINGS OF UNIFIED SOIL SYSTEM CLASSES FOR SOIL CONSTRUCTION *

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; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ 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; $C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ 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	 Plasticity Chart		
			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+

Important Information About Your
Geotechnical Engineering Report

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

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of 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 equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you ASFE-member geotechnical engineer for more information.

ASFE
The Best People on Earth

8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@asfe.org www.asfe.org

Geotechnical, Environmental & Construction Materials Consultants

GILES
ENGINEERING ASSOCIATES, INC.

www.gilesengr.com