IMPORTANT

Notice Regarding This PDF File Transmission

To the Authorized Recipient:

This PDF file is an electronic transmission of our **final** report. An authenticated copy(s) will also be issued to you for your files. The authenticated copy(s) will be affixed with original signatures and/or our corporate seal. While we have taken precautions to assure a complete and secure electronic transmission, please be certain to check this report against the authenticated copy for conformity.

GILES ENGINEERING ASSOCIATES, INC.

Geotechnical Engineering Exploration and Analysis

Proposed 1550 Prospect Apartments
1550 North Prospect Avenue
Milwaukee, Wisconsin

Prepared for:

Heartland Advisors, Inc.
Milwaukee, Wisconsin

December 11, 2013
Project No. 1G-1310022

GILES
ENGINEERING ASSOCIATES, INC.

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

- Atlanta, GA
- Baltimore/Wash. DC
- Dallas, TX
- Los Angeles, CA
- Milwaukee, WI
- Orlando, FL

December 11, 2013

Heartland Advisors, Inc.
789 N. Water St.
Milwaukee, WI 53202

Attention: Mr. Bill Nasgovitz

Subject: Geotechnical Engineering Exploration and Analysis
Proposed 1550 Prospect Apartments
1550 North Prospect Avenue
Milwaukee, Wisconsin
Project No. 1G-1310022

Dear Mr. Nasgovitz:

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

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

WISCONSIN
JEFFREY S.
MILLER
E-15511
WALES,
WI
PROFESSIONAL ENGINEER

Very truly yours,

GILES ENGINEERING ASSOCIATES, INC.

Jeffrey Scott Miller
12-11-2013

Jeffrey Scott Miller P.E.
Sr. Project Manager

Paul J. Giese

Paul J. Giese, P.E.
Geotechnical Division Manager

Distribution: Heartland Advisors, Inc.
Attn: Mr. Bill Nasgovitz (2 USPS)
Streigel-Agacki Studio
Attn: Mr. Joel Agacki (1 USPS, 1 via email: joel@sa-studio.com)

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

TABLE OF CONTENTS

GEOTECHNICAL ENGINEERING EXPLORATION AND ANALYSIS

PROPOSED 1550 PROSPECT APARTMENTS
1550 NORTH PROSPECT AVENUE
MILWAUKEE, WISCONSIN
PROJECT NO. 1G-1310022

APPENDICES

GEOTECHNICAL ENGINEERING EXPLORATION AND ANALYSIS

PROPOSED 1550 PROSPECT APARTMENTS
1550 NORTH PROSPECT AVENUE
MILWAUKEE, WISCONSIN
PROJECT NO. 1G-1310022

EXECUTIVE SUMMARY

This Executive Summary provides limited geotechnical engineering information regarding the proposed project. Since this Executive Summary is abbreviated, it must be read in complete context with the following report.

Subsurface Conditions

- At the test boring locations, asphalt pavement with an aggregate base course, over existing fill was encountered to a depth of 2± feet. The existing fill is composed of silty clay and clayey silt. Beneath the existing fill at the test boring locations, hard to very stiff consistency silty clay was encountered to at least the maximum depths explored of 31± to 61± feet in depth.
- Although free water was not encountered during drilling, it is estimated that the water table was about 13± to 22± feet below-grade at the test borings when the Geotechnical Subsurface Exploration Program was conducted.

Assumptions for the Proposed Apartment Building

- Details of the building framing and structural loading were not developed at the time of preparation of this report, other than the preliminary architectural drawings.
- Assumptions for the building configuration, structural loads, and building elevations used for preparation of the recommendations for this report are based on the preliminary drawings provided.
- The building configuration shown on the preliminary architectural drawings consists of an apartment building with five stories above grade and one story partially below grade. The below grade level is planned for automobile parking extending to about 4 feet below the existing ground surface grade, with the first story about 4 feet above the existing ground surface grade, also for automobile parking. The upper four stories are planned for apartments.
- The apartment building footprint is planned adjacent to the southwest and southeast sides of the existing 2½-story residential building. Giles assumes the proposed apartment building will be supported by interior columns and exterior perimeter bearing walls, without support by the existing building. The structural load assumptions are total column loads ranging from 350 kips to 400 kips, total bearing wall loads of about 9.4 kips per lineal foot, and 100 psf live plus dead load for the lowest parking level floor.
- The lowest level parking level slab elevation is assumed at El. 75± which is approximately 4 feet below the general existing surface grade at El. 79±.

EXECUTIVE SUMMARY (Continued)

Foundations

- Based on the conditions encountered at the test boring locations, the assumed building structural loads and lowest level elevation, a spread footing foundation system is recommended, supported within the native soils below the unsuitable bearing existing fill soils are recommended for the building.

Floor Slab

- With proper sub-grade preparation as described in the report, the native clayey soils at the test boring locations are anticipated to be suitable to support a ground-bearing floor slab.

Stability of Existing Slope

- Analysis results indicate an acceptable stability of the existing slope before, during, and after construction of the proposed building. Certain recommendations to maintain stability during and after construction are provided in the report.

GEOTECHNICAL ENGINEERING EXPLORATION AND ANALYSIS

PROPOSED 1550 PROSPECT APARTMENTS
1550 NORTH PROSPECT AVENUE
MILWAUKEE, WISCONSIN
PROJECT NO. 1G-1310022

1.0 SCOPE OF SERVICES

This report provides the results of the *Geotechnical Engineering Exploration and Analysis* that Giles Engineering Associates, Inc. ("Giles") conducted regarding the proposed development. The *Geotechnical Engineering Exploration and Analysis* included several separate, but related, service areas referenced hereafter as the Geotechnical Subsurface Exploration Program, Geotechnical Laboratory Services, and Geotechnical Engineering Services. The scope of each service area was narrow and limited, as directed by our client and in consideration of the proposed project. The scope of each service area is briefly explained later.

Geotechnical engineering-related recommendations for design and construction of the foundation, ground-bearing floor slab, and below-grade walls for the proposed building are provided in this report. Stability analyses were performed of the existing descending slope at the eastern portion of the site for conditions before, during, and after construction. Site preparation recommendations are given; however, those recommendations are only preliminary since 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 finalized details of the proposed development. Environmental considerations are not included in the scope of services.

2.0 SITE DESCRIPTION

The site is located at the street address of 1550 N. Prospect Avenue in Milwaukee, Wisconsin. It is occupied by an existing 2½-story with basement residential building along the northeastern property line, and an asphalt parking lot pavement within the western two-third portion of the property. An existing descending slope is present within the eastern one-third portion of the site, with weed, brush, and tree vegetation. The site topography in the western portion of the site ascends about 2½ feet to the east above the surface of N. Prospect Avenue, and is relatively level to the descending slope crest in the eastern-third of the property.

3.0 PROJECT DESCRIPTION

Specific details of the proposed apartment building were not developed and therefore unavailable at the time of preparation of this report, other than preliminary architectural drawings. Assumptions for the building configuration, structural loads, and building elevations used for preparation of the recommendations for this report are based on the preliminary drawings provided. The building configuration shown on the preliminary architectural drawings consists of an apartment building with five stories above grade and one story partially below grade. The below grade level is planned for automobile parking extending to about 4 feet below the existing ground surface grade, with the first story about 4 feet above the existing ground surface grade, also for automobile parking. The upper four stories are planned for apartments.

The apartment building footprint is planned adjacent to the southwest and southeast sides of the existing 2½-story residential building, and will extend to within several feet of the southwest property line, and northeastward to the descending slope crest. Giles assumes the proposed apartment building will be supported by interior columns and exterior perimeter bearing walls, without support by the existing building. The structural load assumptions are total column loads ranging from 350 kips to 400 kips, total bearing wall loads of about 9.4 kips per lineal foot, and 100 psf live plus dead load for the lowest parking level floor. The lowest level parking level slab elevation is assumed at El. 75± which is approximately 4 feet below the general existing surface grade at El. 79±.

4.0 GEOTECHNICAL SUBSURFACE EXPLORATION PROGRAM

The purpose of the Geotechnical Subsurface Exploration Program was to explore the subsurface conditions. Four test borings were drilled in the proposed building area. Test Boring Nos. H1 and H2 were drilled to 61± feet in depth, and Test Boring Nos. H3 and H4 were drilled to 31± feet in depth. The test borings were drilled on November 21, 2013 and November 22, 2013. The test boring locations were positioned on-site relative to existing site features. The approximate test boring locations are shown on the *Test Boring Location Plan* (Figure 1) enclosed in Appendix A.

The ground elevations at the test borings were determined by Giles by interpolation of the existing site elevation contour lines shown on the Client's "Existing Site Plan", prepared by Jahnke and Jahnke Associates, Inc. dated April 13, 2005. The test boring elevations are noted on the *Records of Subsurface Exploration* (enclosed in Appendix A), which are logs of the test borings. The test boring elevations are considered accurate to ± one contour line which is ±1 foot.

Samples were collected from the test borings, at certain depths, using a split-barrel sampler during Standard Penetration Testing (SPT) described in Appendix B, along with descriptions of other field procedures. Immediately after sampling, select portions of the SPT samples were retained in jars that were labeled at the site for identification. The retained samples were transported to Giles' geotechnical laboratory as part of the Geotechnical Subsurface Exploration Program.

5.0 GEOTECHNICAL LABORATORY SERVICES

The retained samples were delivered to our geotechnical laboratory and 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 2488-75) as a general guide. The classifications are shown on the *Records of Subsurface Exploration*, along with horizontal lines that show estimated depths of material change. Field-related information pertaining to the test borings is also shown on the *Records of Subsurface Exploration*. For simplicity and abbreviation, terms and symbols are used on the *Records of Subsurface Exploration;* the terms and symbols are defined on the *General Notes*.

Unconfined compression, calibrated penetrometer resistance, and moisture content tests were performed on select soil samples to evaluate the soils general engineering properties. The test results are on the *Records of Subsurface Exploration*. Laboratory procedures are briefly described in Appendix C.

6.0 MATERIAL CONDITIONS

Since material sampling at the test borings was discontinuous, it was necessary for Giles to estimate conditions between sample intervals. The estimated conditions at the test borings are briefly discussed in this section and are described in more detail on the *Records of Subsurface Exploration*. Also, the conclusions and recommendations in this report are based on the estimated and encountered conditions.

6.1. Surface Materials and Existing Fill

At the test boring locations, a 2± to 4± inch thick asphalt pavement with an 8± inch thick aggregate base course is present. Under the pavement base course, existing fill was encountered to a depth of 2± feet. The existing fill is composed of silty clay and clayey silt.

6.2. Native Soil

Native soils beneath the existing fill at the test boring locations generally consist of hard to very stiff consistency silty clay to at least the maximum depths explored of 31± to 61± feet in depth. The native silty clay to a depth of 9± feet contains fissures. At the location of Test Boring No. H1, very dense silty sand and gravel was encountered below 54± feet in depth and to at least the maximum depth explored of 61± feet.

7.0 GROUNDWATER CONDITIONS

The level of the long-term water table was about 13± to 22± feet below-grade at the test boring locations when the Geotechnical Subsurface Exploration Program was conducted. Free water did not accumulate within the test borings during drilling or at the time of drilling completion. Groundwater conditions will fluctuate and groundwater may become perched above the water table, especially in the existing fill.

The estimated water table depth and/or perched water depth is only an approximation based on the colors and relative water content of the retained soil samples. The actual water table depth may be higher or lower than estimated.

8.0 CONCLUSIONS AND RECOMMENDATIONS

8.1. Site Design Considerations

The very stiff to hard consistency silty clay native soils encountered at the test boring locations are considered suitable for shallow depth spread foundation support of the columns and perimeter walls. Suitable bearing native soils are anticipated to be present at shallow depth below the lowest parking level within the area of the proposed building. New foundations near the basement of the existing building and descending slope may need to be extended in depth to suitable bearing native soils.

The structural design of the proposed apartment building is recommended to include expansion joints at any connections with the existing residential building. The purpose of the recommended expansion joints is to account for the potential differential settlement between the existing building and the new proposed building.

8.2. Seismic Design Considerations

A soil Site Class C is recommended for seismic design. By definition, Site Class is based on the average properties of subsurface materials to a depth of 100 feet below the ground surface. Since 100-foot test borings were not requested or authorized for the project, it was necessary to estimate the Site Class based on the test borings, presumed area geology, and Table 1613.5.2 of the 2006 International Building Code.

8.3. Slope Stability Analysis and Construction Considerations

The existing descending slope is considered to be stable along the southeast portion of the site. The stability is based on the results of a slope analysis. The results of the analysis are presented on Figure 2 enclosed in Appendix A. The factors of safety values obtained by the analysis indicate the slope is stable against deep rotational and translational slide failures considering the current conditions.

The subsurface conditions encountered at Test Boring Nos. H1 and H2 drilled nearby the crest of the descending slope were used to characterize the slope subsurface conditions. The engineering properties of the subsoils used in the slope stability analysis calculations were indirectly determined by estimation by using the results of field and laboratory tests performed by Giles on the samples obtained from Test Boring Nos. H1 and H2, and experience from other projects as a guideline. The slope topography used in the analysis was obtained from the existing site elevation contour lines shown on the Client’s “Existing Site Plan”, prepared by Jahnke and Jahnke Associates, Inc. dated April 13, 2005.

8.4. Building Foundation Recommendations

The proposed building is recommended to be supported by a shallow depth spread foundation system. The spread foundations are recommended to be founded directly on suitable bearing native soil and/or lean-mix concrete backfill placed continuous from a suitable bearing native soil sub-grade to replace unsuitable bearing soils. The foundations are recommended to be designed using a 6,000 psf maximum, net, allowable soil bearing capacity. Strip footing pads are recommended to be at least 18 inches wide and isolated column pads are recommended to be at least 24 inches wide for geotechnical considerations, regardless of the calculated foundation bearing stress. Foundation walls could be built of cast-in-place concrete or concrete masonry units. It is recommended that a structural engineer or architect provide specific foundation details including footing dimensions, reinforcing, and other details.

Spread Footing Embedment Depth Recommendations

The continuous wall foundations that are adjacent to and less than 4 feet distance from the existing building basement foundations are recommended to be founded on suitable bearing native soils or lean-mix concrete backfill extended in depth to at least the existing foundation bearing grade. Parallel foundations should be at least 1.5 footing widths apart (whichever is larger, either existing or new foundations) to help reduce the additional stress imposed on the existing foundations, unless it is determined that the over-lapping stress imposed by the existing and new foundations are within acceptable limits for the existing soil. Where parallel foundations are greater than 4 feet apart, the new foundations may be placed at higher elevations (less embedment), the new foundation should not be founded at a bearing grade within a 45-degree envelope extending upward from the outside edges of the existing foundations.

Based on the building configuration shown on the preliminary architectural drawings, the continuous wall foundations and possibly the column foundations along the descending slope at the southeast end of the proposed building are recommended to be founded on suitable bearing native soils or lean-mix concrete backfill extended in depth to at least El. 73 which is about 6 feet below the existing ground surface grade. If the southeast building wall line is relocated in the final planning for the project, the foundation embedment recommendation may need to be revised.

It is understood that a minimum 48-inch foundation depth is required by the local building code. Therefore, footings for perimeter walls and other exterior elements of the proposed structure are recommended to bear at least 48 inches below the finished ground grade. Interior footings could be directly below the floor slab if the building will be heated and support soil will not freeze.

It is critical that contractors protect foundation support soil and foundation construction materials (concrete and reinforcing). In addition, the excavations along the interior and exterior sides of continuous wall foundations, foundation walls, column foundations and pedestals below the lowest floor subgrade are recommended to be backfilled with on-site clayey soils placed and compacted as engineered fill in benched excavations immediately after the foundations, walls, and pedestals are capable of supporting lateral pressures from backfill, compaction, and compaction equipment. Forms will be needed where excavations are extended into the clayey soils with sand seams or lenses or in sandy soils and where the soils are not stable.

Spread Foundation Support Soil Requirements

Footing pads are recommended to be directly and entirely supported by suitable-bearing native soil or on lean-mix concrete backfill placed continuous from a suitable bearing native soil subgrade. Based on the recommended 6,000 psf bearing capacity, suitable bearing soils are recommended to have at least a very stiff comparative consistency, average unconfined compressive strength value equal to or greater than 3.0 tsf. It is further recommended that the strength characteristics of soil within the entire foundation influence zone (determined by Giles during construction) meet or exceed the recommended values, unless Giles approves lesser depths.

Suitable bearing native soils for direct spread foundation support or for the subgrade of engineered backfill and indirect foundation support are anticipated to be present at the depths and elevations shown on the table below.

TABLE 1
ANTICIPATED SUITABLE BEARING GRADE DEPTH AND ELEVATION FOR DIRECT/INDIRECT FOUNDATION SUPPORT (1)

Test Boring Location	Anticipated Suitable Bearing Grade	
	Depth Below Existing Surface (2) (feet)	Elevation (3)
H1	4±	75.5±
H2	3±	76.7±
H3	3±	76.7±
H4	4±	75.3±

1. Maximum net, allowable bearing capacity of 6,000 psf
2. Depth below the approximate existing ground surface at the time of drilling.
3. Referenced to the elevations shown on Client's "Existing Site Plan", Jahnke and Jahnke dated 4-13-2005

For design and construction estimating purposes, the suitable bearing grade may be interpreted linearly between the test boring locations. The actual suitable bearing grade may differ because the subsurface conditions may differ beyond the test boring locations; as such, the geotechnical engineering recommendations in this report are predicated upon Giles evaluating the suitability of the foundation support soils during construction to check that the foundations are supported within and underlain by suitable bearing materials.

It is recommended that Giles evaluate foundation support soil immediately before foundation construction. The purpose of the recommended evaluation is to confirm that the foundation will be properly supported and confirm that the support soil is similar to the conditions described on the *Records of Subsurface Exploration*. In the event that another firm performs the recommended foundation evaluation of foundation support soil, they should use appropriate means and methods and Giles must be notified if the composition or strength characteristics of foundation support soil differ from those shown on the *Records of Subsurface Exploration* so that alterations to our recommendations can be made if needed.

Soil that is within the foundation influence zone but does not meet the recommended strength criteria, or is otherwise unsuitable, is recommended to be replaced. Unsuitable bearing material is recommended to be replaced with lean-mix Portland cement concrete backfill with a minimum 28-day compressive strength of 500 psi. Giles can provide recommendations pertaining to soil over-excavation and replacement at the time of construction. As an option to soil replacement, strip footing pads could be stepped or thickened to extend through unsuitable bearing materials and isolated column pads could be uniformly thickened. It is recommended that a structural engineer or architect provide the specific details of stepped or thickened footings.

Estimated Spread Foundation Settlement

The post-construction total and differential settlements of a spread footing foundation designed and constructed based on the recommendations of this report are estimated to be less than about 1.0 inch and 0.5 inch, respectively. The post-construction angular distortion is estimated to be less than about 0.0021 inch per inch across a distance of 20 feet or more. Estimated settlements are based on the assumption that soils will be thoroughly tested and approved by a licensed qualified geotechnical engineer during foundation construction. The estimated settlements are considered within tolerable limits for the planned development provided they are properly considered in the architects and structural engineers design.

Existing Construction Considerations

An expansion joint should separate the adjacent existing building and the new addition, since some differential movement is expected to occur at this juncture. Precautions must be taken so that excavations for the new construction do not undermine existing footings and floor slabs or otherwise compromise the existing structure support. Depending on the actual details of the existing building and proximity of the new foundation excavations to the existing building foundations, underpinning may be needed. If voids occur below existing footings or floor slab, Giles should be contacted to observe the conditions and provide recommendations. In general, voids should be immediately filled with a concrete dry pack or injection of a non-shrink expansive sand and cement slurry under appropriate pressure to maintain contact between the structure and supporting soils.

If the new and existing footings will bear at different elevations, the project structural engineer should evaluate the stresses to be imposed on the lower foundation system and confirm that the structural integrity of the existing building and new addition will be maintained. Parallel foundations should be at least 1.5 footing widths apart (whichever is larger, either existing or new) to help reduce the additional stress imposed on the existing foundations, unless it is determined that the over-lapping stress imposed by the existing and new foundations are within acceptable limits for the existing soil. Where parallel foundations are wider than 4 feet apart, their different elevations should not place them within a 45-degree envelope extending upward and downward from the outside edges of the existing foundations. Care must be taken to protect the existing building during construction of the addition. The existing building should be braced, where needed.

8.5. Floor Slab Recommendations

The lowest parking level floor of the building is recommended to be a ground-bearing concrete slab for the eastern, spread foundation supported portion of the building. The building configuration shown on the preliminary architectural drawings indicate the lowest parking level floor surface is planned at El. 75 which is at about 4 feet below the existing site surface. It is assumed that the base course sub-grade will be at El. 73.5±. Based on the assumed sub-grade elevation and with proper sub-grade preparation, it is anticipated that the soils encountered at the test boring locations will be suitable for floor slab support. Over-excavation of unsuitable bearing soil may be necessary to develop a suitable floor slab sub-grade considering existing fill was found at the test boring locations, and possibly adjacent to the existing building basement and in the vicinity of the descending slope crest. Unsuitable bearing soils are recommended to be replaced with engineered fill that is selected, placed, and compacted according to the recommendations presented in this report.

Assuming a maximum 100 psf floor load, the slab is recommended to be at least 4 inches thick with at least a 2,000 pounds per square inch (psi) 28-day compressive strength concrete mix. The floor slab is recommended to be reinforced with welded wire fabric to help control shrinkage cracking. In lieu of welded wire fabric, the floor slab concrete could alternatively contain an appropriate concrete admixture, such as fiber mesh to help control shrinkage cracking. It is recommended that a structural engineer or architect specify the floor slab thickness, reinforcing, joint details and other parameters.

Estimated Ground-Supported Floor Slab Settlement

The post-construction total and differential settlements of the floor slab constructed in accordance with the recommendations of this report are estimated to be less than about 0.5 inch and 0.3 inch, respectively, over a distance of about 20 feet. Estimated settlements are based on the assumption that soils will be thoroughly tested and approved by Giles during floor slab construction. The estimated settlements are considered within tolerable limits for the planned development provided they are properly considered in the architects and structural engineers design.

Parking Level Floor Drainage Base Course

For both the ground-supported slab of the lowest parking level floor, a minimum 8-inch-thick, free-draining aggregate, drainage base course is recommended to be directly below the lowest parking level floor slab due to the potential for perched water and surface runoff water level accumulations on the site. The base course will serve as a drainage layer and will also serve as a capillary break. The base course is recommended to be continuous with the below-grade wall drainage system. Also, it is recommended that Giles test and approve base course aggregate before it is placed.

A non-woven geotextile is recommended to be placed on the subgrade soils below the base course throughout the entire floor area. A minimum 10-mil vapor retarder is recommended to be directly below the floor slab concrete course for concrete curing and the effects of moisture on future flooring materials. The vapor retarder is recommended to be in accordance with ASTM E 1745-97, which is entitled: *Standard Specification for Plastic Water Vapor Retarders*

Used in Contact with Soil or Granular Fill under Concrete Slabs. If the base course has sharp, angular aggregate, protecting the retarder with a geotextile (or by other means) is recommended.

8.6. Below-Grade Wall Recommendations

A drainage system should surround the below-grade walls and be connected with the drainage system below the lowest parking level floor slab. Also, the below-grade walls should be designed to withstand earth pressures and lateral pressures from surcharges near the walls. Drainage system recommendations and geotechnical design parameters for below-grade walls are provided below.

Drainage System Recommendations

A drainage system is recommended to remove water near the below-grade walls. It will lessen the potential for water pressure build-up against the below-grade walls, which could cause wall movement, wall distress, and interior water or moisture problems. The drainage system is recommended to use drainage aggregate backfill, or a drainage geocomposite. The drainage aggregate alternate is recommended to be in accordance with the WDOT Standard Specifications, Section 501.3.6.4.5 Size No.1, sometimes locally known as crushed, clear, No.1 stone. The drainage geocomposite alternate is recommended to consist of TENAX TENFLOW 770-2, or alternate drainage geocomposite approved for use by Giles prior to installation. Both alternate drainage materials are recommended to be connected to continuous drainpipes adjacent to the bottom of the below-grade walls.

Drainage aggregate should abut the below-grade walls as part of the drainage system. The aggregate will serve as drainage media against the below-grade walls. The aggregate layer should be at least 2 feet wide, measured horizontally from the below-grade walls. Also, the aggregate layer should begin at the lowest parking level floor slab subgrade or at the base of the deeper footing pads and it should continuously abut the below-grade walls. The drainage aggregate could consist of a well-graded free draining aggregate. An open-graded aggregate could also be used, but if used, a geotextile is recommended to be placed between the drainage aggregate and soil of the excavation to provide separation.

Backfill that is placed adjacent to below-grade walls, and will also provide structural support, should be compacted in accordance with the *Guide Specifications* enclosed. Compaction should be performed to between 92 and 95 percent of the maximum dry density obtained by the Standard Proctor compaction test (ASTM D698) or to an in-place density specified by the project structural engineer. The drainage aggregate should be compacted in maximum 8 to 12-inch-thick lifts (measured loose). Heavy compaction equipment, such as mechanical rollers, should not operate within about 10 feet of the below-grade walls because high lateral pressures could develop and the walls could move and possibly fail. Hand-operated compaction equipment, such as vibratory plates, should be used within about 10 feet of below-grade walls. Below-grade walls should be braced during construction, backfilling, and compaction. The bracing should remain in-place until the below-grade floor slab and main deck are installed.

Continuous drainpipes should be installed along the interior and exterior sides of the perimeter strip footing pads; creating interior and exterior drainage loops. The drain pipes should be minimum 4-inch-diameter perforated pipes specifically designed for drainage applications. The interior and exterior drain pipe systems should connect to a central sump crock or crocks, or connected to the storm sewer system for removal. Drain pipe bleeders should be cast in the perimeter strip footing pads to serve as water conduits between interior and exterior drainpipes. The bleeder pipes should be about 3 inches in diameter and about 6 to 8 feet-on-center. Proper damp-proofing should also be applied to the exterior walls. Dependent on water levels encountered during construction, multiple sump crocks or other specialized designs may also be necessary. Additionally, some increased and possibly continuous sump pump activity may occur during wet weather periods. Consideration should be given to use of a generator or battery operated back-up sump pump to reduce water problems in the event of a power failure.

Lateral Pressure Design Parameters

A structural engineer should design the below-grade walls to resist lateral pressures from the adjacent soil and any surface surcharges. Assuming that aggregate will continuously abut the below-grade walls as recommended, an equivalent "at-rest" fluid pressure of 75 psf per foot of depth may be used for below-grade wall design. If the recommended drainage system is not installed along the below-grade walls and soil that is not free-draining, such as silty clay, abuts the below-grade walls, a higher lateral pressure, likely in the range of 90 to 100 psf per foot depth, may develop. Silty or clayey soil should not be within about two feet of the below-grade walls due to high earth pressures, potential frost damage, and insufficient drainage, which could cause the below-grade walls to become damp or wet. Lateral pressures caused by any surcharge loads should be added to the "at-rest" fluid pressure recommended above. We can provide supplemental recommendations regarding surcharge loads on a case-by-case basis. Below-grade walls that are not designed to resist the actual pressures will be prone to dampness and/or lateral movement and potential distress.

Cast-in-place Portland cement concrete may be used to construct the below-grade walls. Basement wall construction and reinforcing should be, at a minimum, in accordance with Chapter 18 of the 2006 International Building Code (IBC). Wall design and construction must include adequate reinforcing to resist lateral pressures.

8.7. Generalized Site Preparation Recommendations

This section deals with site preparation. 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 gives 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.

Site Preparation Recommendations Relative to the Slope Stability

The slope stability can be affected by future events because the slope face is susceptible to water runoff-caused soil erosion, especially if vegetation is removed. Some movement of the slope face may occur in the future, due to several affects. These affects include soil creep down-slope due to the moderately steep ±31° angle of the bluff, and an increase in the amount of the average monthly/yearly precipitation. An increase in the precipitation should be expected, because it is currently low relative to the average historic values. The stability of the immediately adjacent properties to the northeast and southwest has an affect on the stability of this property. Both stability and instability on the adjacent properties can have a similar affect on the stability of this property, and conversely.

The processes for the proposed apartment building and pavement construction can have a detrimental affect on the stability of the slope on and east of this property, and also on the adjacent properties. Prior to construction, temporary and/or final grading of the site area west of the crest of the descending slope using earth cut rather than filling is recommended, if needed, to divert surface runoff water from precipitation, snow melt, and construction activity to a storm sewer system, or northwestward and away from the slope crest. Thereafter, construction vehicle parking, stockpiled demolition and new construction materials, and stockpiled soils are recommended to be prohibited from the area between the proposed building and the slope crest, and eastward on the slope face. This includes prohibition of placement on the slope face below the crest of demolition debris, excavated soil, water discharge from sump pumps, and water and concrete discharge from concrete delivery trucks. Construction vehicles needed for demolition and/or new construction such as backhoes, loaders, cranes, bulldozers, dump trucks, and delivery trucks for cast concrete and construction materials are recommended to be parked within the central and northwestern portions of the site.

The demolition and new construction activities with restrictions described above are not expected to cause a detrimental affect on the bluff stability. As a precautionary measure, however, the ground surface along the northwest side of the slope crest is recommended to be periodically observed by the Client and contractors for the development of cracks generally parallel to the slope crest line. If cracks are observed, Giles is recommended to be notified as soon as possible upon noticing the development of cracks to evaluate the conditions and provide recommendations. The cracks can be a signal of slope instability.

Clearing, Grubbing and Stripping

Site preparation will require complete removal and proper disposal of the existing asphalt pavement construction, including underground utilities that are not reused. Remains of previous construction were not encountered at the test boring locations; however remains may have been buried beneath the surface in the past. It should be noted that any construction remnants left in-place may cause excavation difficulties for new utilities, landscape plantings and future construction.

All surface vegetation, organic material, rubble, and debris materials are recommended to be removed from the site, with handling and removal methods in accordance with local, state, and federal regulations for the materials. The existing pavement should be properly disposed of off-site or pulverized into a maximum 3-inch sized material and stockpiled on-site for use as fill, subbase course, sub-grade stabilization material or reclaimed asphalt pavement (RAP) material. The existing base course material could also be reused as a subbase course material and possibly base course material.

Proof-Rolling and Fill Placement for Surface Parking Pavement

After the recommended clearing, grubbing, and stripping, and once the site is cut (lowered) as needed, the sub-grade is recommended to be proof-rolled with a fully-loaded, tandem-axle dump truck or other suitable construction equipment to help locate unstable soil based on sub-grade deflection caused by the wheel loads of the proof-roll equipment. It is recommended that Giles observe proof-roll operations and evaluate the sub-grade stability based on those observations.

Soil that yields excessively or ruts during proof-rolling, or shows other signs of instability, is recommended to be replaced with engineered fill. As an option to replacement, unsuitable soil could be scarified to a sufficient depth (likely 6 to 12 inches, or more), moisture-conditioned (uniformly moistened or dried), and compacted to the required in-place density. Unsuitable soil could also be modified with hydrated lime or Portland cement, or mechanically stabilized with coarse aggregate and/or with geogrids or geotextiles. It is recommended that soil improvement recommendations be provided by Giles based on the conditions during construction.

The site is recommended to be raised, where necessary, to the planned finished grade with engineered fill immediately after the sub-grade 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). And, 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 geotechnical test titled: *Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort* (ASTM D698). That test is hereafter referenced as: *The Standard Proctor Compaction test*. As an exception, the in-place dry density of engineered fill for the top one foot of the pavement sub-grade is recommended to be compacted to at least 100 percent of the fill's maximum dry density. Item Nos. 4 and 5 of the *Guide Specifications* give more information pertaining to selection and compaction of engineered fill.

The water content of fill material is recommended to be uniform and within a narrow range of the optimum moisture content, as described in Item No. 5 of the *Guide Specifications*. The optimum moisture content is to be determined by the Standard Proctor Compaction test.

Engineered fill that does not meet the density and water content requirements is recommended to be replaced 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 Giles confirms that the previous lift was properly placed and compacted. Sub-grade soil may need to be recompacted immediately before construction since equipment traffic and adverse weather may reduce soil stability.

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. Considering the measured water contents of the tested soil samples, site soil may need to be moisture-conditioned prior to use as engineered fill. Also, miscellaneous building debris and other unsuitable materials may need to be sorted from on-site soil 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, aggregate fill (or other fill material with a low water-sensitivity) will likely 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 due to adverse weather, which commonly occurs during late fall, winter, and early spring. At least some over-excavation and/or stabilization of unstable soil should be expected if construction is during or after adverse weather. Some over-excavation is expected to be needed even if construction is during favorable, dry weather due to the existing fill. Because site preparation is weather dependant, bids for site preparation, and other earthwork activities, should consider the time of year that construction will be conducted.

In an effort 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. Also, contoured sub-grades are recommended to be rolled with a smooth-drum compactor, before precipitation, to “seal” the surface. Furthermore, construction traffic should be restricted to certain aggregate-covered areas in an effort to reduce traffic-related soil disturbance. Pavement construction should begin immediately after suitable support is confirmed.

Dewatering

The level of the long-term water table was estimated to be about 13± to 22± feet below-grade at the test boring locations when the Geotechnical Subsurface Exploration Program was conducted. Perched water conditions may also be present at shallower depths at the time of construction. Based on the assumed lowest parking level floor at El. 75± which is about 4 feet below the site surface grade, excavations for the lowest parking level floor and foundations are anticipated to be shallower than the ground water and or perched water levels.

Filtered sump-pumps placed in the bottoms of excavations, or other conventional dewatering methods are expected to be adequate above and within one foot of the water table and/or perched water levels. However, more complex dewatering techniques, such as well points or other methods approved by a Giles geotechnical engineer are expected to be needed if excavations extend below the water table and/or perched water levels, especially if water bearing sand seams or layers are encountered. If dewatering is required, it is recommended that water be directed to the northwest, away from the slope.

Excavation Stability and Considerations

Stability and caving problems may occur due to variable fill materials and due to the strength properties of the clayey native soils containing fissures. 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 or braced to maintain or develop 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 was encountered at the test boring locations. Considering the existing fill, and existing construction, unsuitable bearing materials may have been buried beneath the site surface during previous grading. Questionable materials, where encountered, are recommended to be evaluated by Giles to determine if removal and replacement with engineered fill is necessary. Disposal of any unsuitable material should be in accordance with local, state and federal regulations for the material type. Alteration to the recommendations of this report may be needed, if conditions different than those noted on the *Records of Subsurface Exploration* are revealed.

Existing Utilities

All existing utilities are recommended to be located, and any planned to be maintained should be relocated outside the proposed building area, if possible. Utilities that are not reused should be capped-off and removed or properly abandoned in-place in accordance with local codes and ordinances. The excavations for utilities to be removed that are in the influence zone of new construction are recommended to be backfilled with engineered fill placed under engineering controlled conditions. Underground utilities that are to be reused or abandoned in-place should be evaluated by the plumbing contractor, and utility backfill should be evaluated by Giles, to determine their potential effect on the new development. Grading operations must be done carefully so that existing utilities are not damaged or disturbed. Utility invert elevations, depths and sizes should be checked relative to the planned foundation elevations to determine what specific concerns are present.

8.9. Recommended Construction Materials Testing Services

This report was prepared assuming that Giles will perform Construction Materials Testing (“CMT”) services during construction of the proposed development. In general, CMT services are recommended (and expected) to at least include observation and testing of: pavement support soil; concrete; asphalt, and other construction materials. 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 based on Giles’ proposal, which is dated October 30, 2013 and is referenced by Giles’ proposal number 1G-1310062. The actual services for the project varied somewhat from those described in the proposal because of the conditions that were encountered while performing the services and in consideration of the proposed project.

This report is strictly based on the project description given earlier in this report. Giles must be notified if any parts 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 *Records of Subsurface Exploration*. Giles must be notified if the subsurface conditions that are encountered during construction of the proposed development differ from those shown on the *Records of Subsurface Exploration* because this report will likely need to be revised. General comments and limitations of this report are given in the appendix.

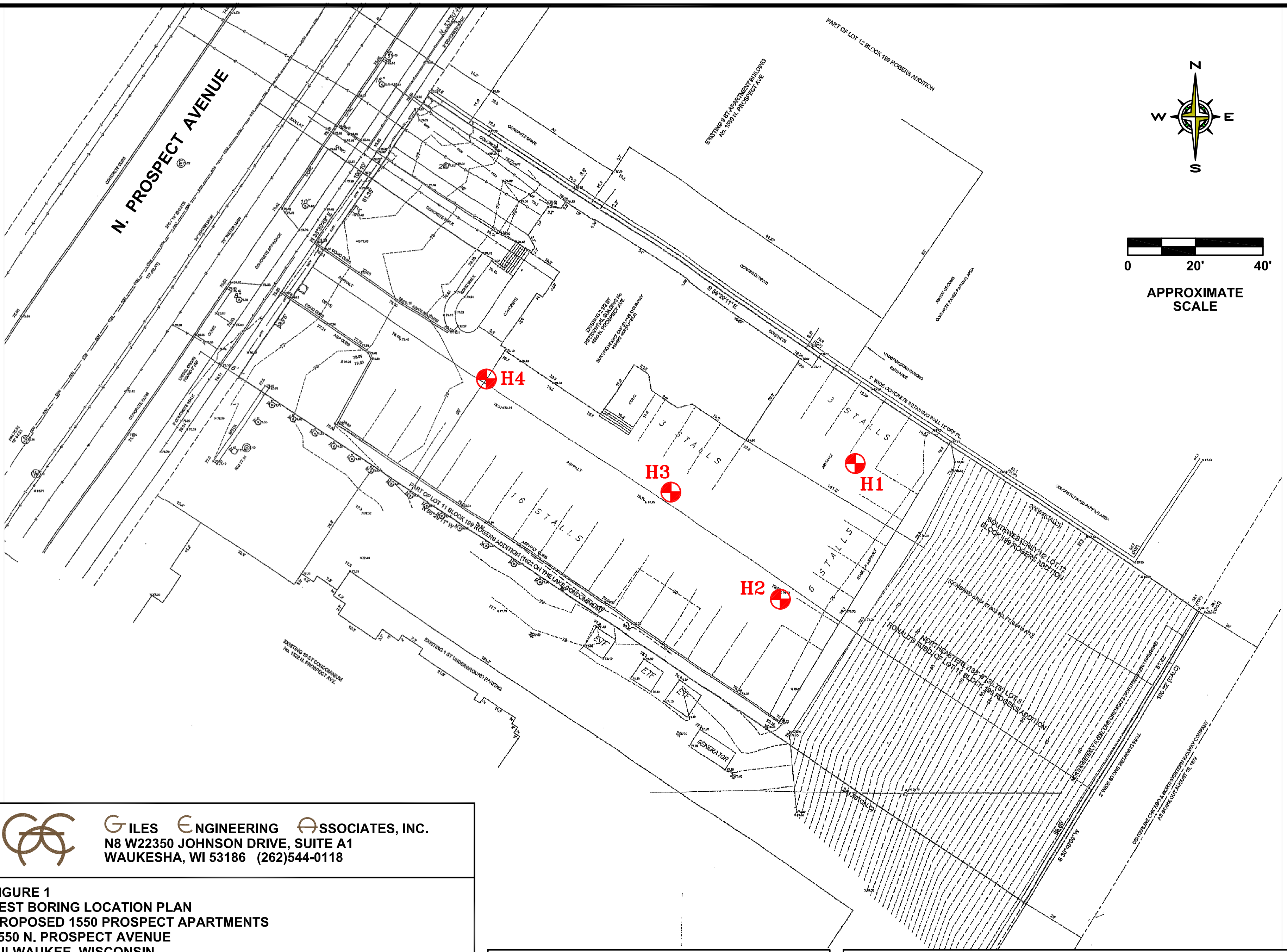
1G-1310022-report/13Geo04/jsm

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.



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

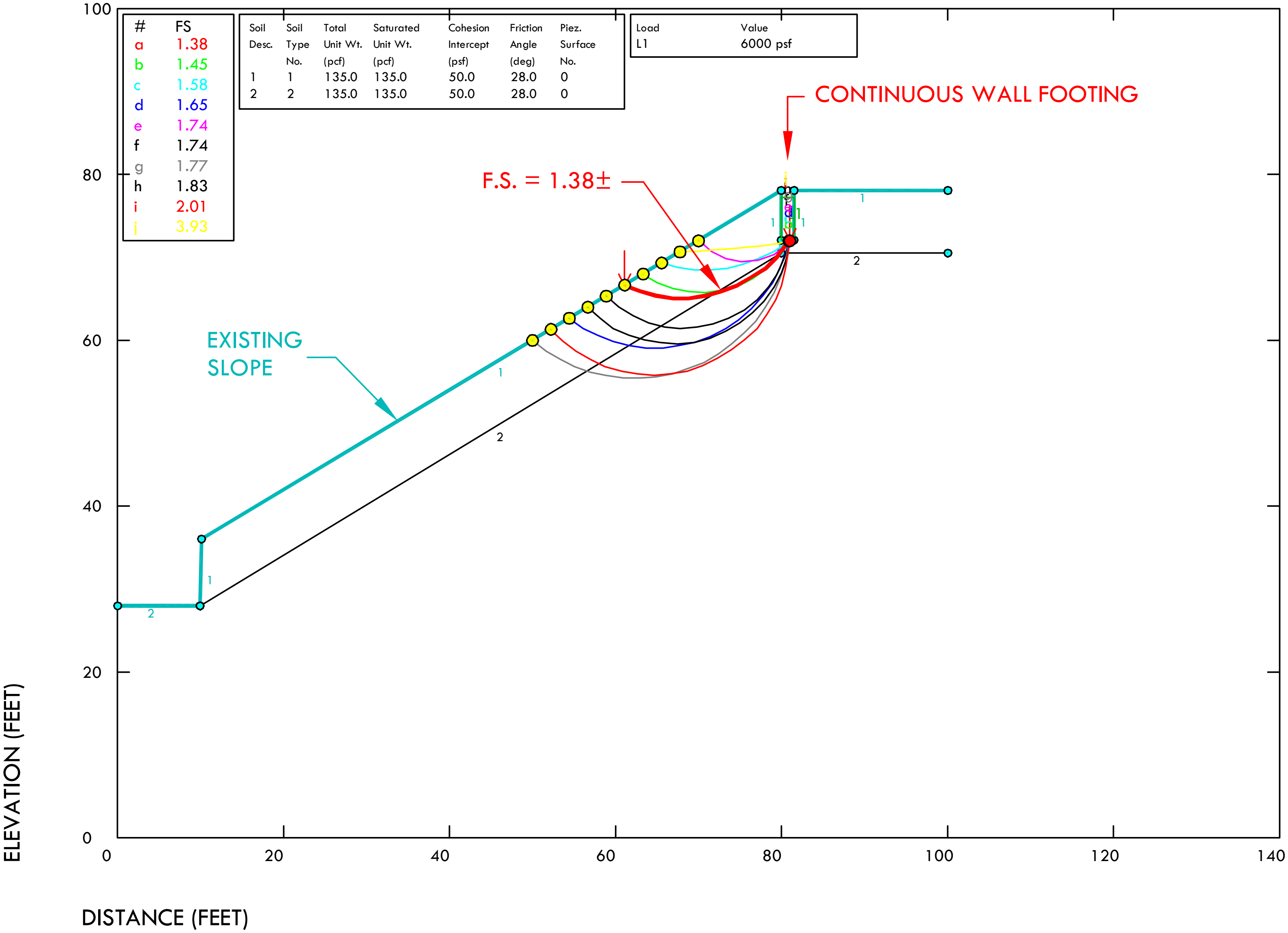
FIGURE 1
TEST BORING LOCATION PLAN
PROPOSED 1550 PROSPECT APARTMENTS
1550 N. PROSPECT AVENUE
MILWAUKEE, WISCONSIN

DESIGNED	DRAWN	SCALE	DATE	REVISED
JSM	JSZ	approx. 1"=40'	11-23-13	--
PROJECT NO.: 1G-1310022			CAD No. 1g1310022-blp	

LEGEND:

H1 ◐	GEOTECHNICAL TEST BORING

NOTES:
1.) TEST BORING LOCATIONS ARE APPROXIMATE.
2.) BASE MAP DEVELOPED FROM THE "EXISTING SITE PLAN" DATED 4-13-05, PREPARED BY JAHNKE & JAHNKE ASSOCIATES, INC.



#	FS
a	1.38
b	1.45
c	1.58
d	1.65
e	1.74
f	1.74
g	1.77
h	1.83
i	2.01
j	3.93

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Piez. Surface No.
1	1	135.0	135.0	50.0	28.0	0
2	2	135.0	135.0	50.0	28.0	0

Load	Value
L1	6000 psf

SLOPE STABILITY ANALYSIS
PROPOSED 1550 PROSPECT APARTMENTS
1550 N. PROSPECT AVENUE
MILWAUKEE, WISCONSIN

GILES ENGINEERING ASSOCIATES, INC.
N8 W22350 JOHNSON DR.; WAUKESHA, WI, 53186
(414)-544-0118

FIGURE 2
SLOPE STABILITY ANALYSIS RESULTS

DESIGNED	DRAWN	APPROVED	SCALE	DATE
JSM	JSM		NONE	12-10-2013

PROJECT NO.: 1G-1310022
CAD NO.: 1310022-F2

RECORD OF SUBSURFACE EXPLORATION

BORING NO. & LOCATION: H1
SURFACE ELEVATION: 79.5
COMPLETION DATE: 11/22/13
FIELD REPRESENTATIVE: Beauford Jones

PROJECT: Proposed 1550 Prospect Apartments
PROJECT LOCATION: 1550 North Prospect Avenue
Milwaukee, Wisconsin
GILES PROJECT NUMBER: 1G-1310022

GILES ENGINEERING ASSOCIATES, INC.
Atlanta Dallas
Los Angeles Milwaukee
Orlando Washington, D.C.

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u **(tsf)**	q_p **(tsf)**	q_s **(tsf)**	w **(%)**	PID	NOTES
4"± Asphalt Pavement 8"± Aggregate Base Course		1-SS	5						
Brown and Dark Gray Silty Clay, little Sand and Gravel (Fill)-Moist		2-SS	8	2.7	2.7		21		
Brown Silty Clay, trace fine to coarse Sand and Gravel (contains Calcareous Deposits in Fissures)-Moist to Damp	5	3-SS	17	7.8	4.5+		17		
		4-SS	14	6.8	4.5+		17		
Brown to Gray Silty Clay, trace fine Sand-Moist	10	5-SS	12	6.2	4.5+		18		
	15	6-SS	11	4.0	3.7		20		
Gray Silty Clay, trace fine Sand-Moist	20	7-SS	11	3.5	2.7		18		
	25	8-SS	10	3.7	3.5		17		
	30	9-SS	10	3.6	3.2		18		
	35	10-SS	14	3.8	3.5		17		

WATER OBSERVATION DATA

▽ WATER ENCOUNTERED DURING DRILLING: None
▼ WATER LEVEL AFTER REMOVAL: None
▨ CAVE DEPTH AFTER REMOVAL: 54.0 ft.
▼ WATER LEVEL AFTER HOURS:
▨ CAVE DEPTH AFTER HOURS:

REMARKS

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

RECORD OF SUBSURFACE EXPLORATION

GILES ENGINEERING ASSOCIATES, INC.

Atlanta Dallas
Los Angeles Milwaukee
Orlando Washington, D.C.

BORING NO. & LOCATION: H1	PROJECT: Proposed 1550 Prospect Apartments
SURFACE ELEVATION: 79.5	PROJECT LOCATION: 1550 North Prospect Avenue
COMPLETION DATE: 11/22/13	Milwaukee, Wisconsin
FIELD REPRESENTATIVE: Beauford Jones	GILES PROJECT NUMBER: 1G-1310022

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u **(tsf)**	q_p **(tsf)**	q_s **(tsf)**	w **(%)**	PID	NOTES
Gray Silty Clay, trace fine Sand-Moist *(continued)*		11-SS	14	3.7	3.5		17		
	45	12-SS	11	3.8	3.2		17		
Gray Silty Clay, little Gravel-Damp									
	50	13-SS	27	5.5	4.5+		16		
Gray Silty fine to coarse Sand and Gravel-Damp	55	14-SS	53						
	60	15-SS	54						

Boring Terminated at 61 Feet

	WATER OBSERVATION DATA	**REMARKS**
∇	WATER ENCOUNTERED DURING DRILLING: None	
∇	WATER LEVEL AFTER REMOVAL: None	
▨	CAVE DEPTH AFTER REMOVAL: 54.0 ft.	
▼	WATER LEVEL AFTER HOURS:	
▨	CAVE DEPTH AFTER HOURS:	

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.

RECORD OF SUBSURFACE EXPLORATION

BORING NO. & LOCATION: H2

SURFACE ELEVATION: 79.7

COMPLETION DATE: 11/22/13

FIELD REPRESENTATIVE: Beauford Jones

PROJECT: Proposed 1550 Prospect Apartments

PROJECT LOCATION: 1550 North Prospect Avenue

Milwaukee, Wisconsin

GILES PROJECT NUMBER: 1G-1310022

GILES ENGINEERING ASSOCIATES, INC.
Atlanta Dallas
Los Angeles Milwaukee
Orlando Washington, D.C.

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u **(tsf)**	q_p **(tsf)**	q_s **(tsf)**	w **(%)**	PID	NOTES
2"± Asphalt Pavement 8"± Aggregate Base Course		1-SS	8						
Brown Silty Clay, little fine to coarse Sand and Gravel (Fill)-Moist		2-SS	13	3.9	4.5+		17		
Brown Silty Clay, trace fine to coarse Sand and Gravel (Contains Calcareous Deposits in Fissures)-Moist to Damp	5	3-SS	12	4.9	4.5+		16		
		4-SS	11	4.1	4.0		17		
Brown to Gray Silty Clay, trace fine Sand-Moist	10	5-SS	13	4.5	4.0		18		
	15	6-SS	11	3.3	2.7		20		
	20	7-SS	7	2.6	2.2		20		
Gray Silty Clay, trace fine Sand-Moist									
	25	8-SS	10	3.7	3.2		19		
	30	9-SS	15	3.6	3.0		18		
	35	10-SS	17	4.5	3.7		17		

WATER OBSERVATION DATA

▽ WATER ENCOUNTERED DURING DRILLING: None

▼ WATER LEVEL AFTER REMOVAL: None

░ CAVE DEPTH AFTER REMOVAL: 57.0 ft.

▼ WATER LEVEL AFTER HOURS:

▒ CAVE DEPTH AFTER HOURS:

REMARKS

(a) Poor Recovery

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

RECORD OF SUBSURFACE EXPLORATION

BORING NO. & LOCATION: H2
SURFACE ELEVATION: 79.7
COMPLETION DATE: 11/22/13
FIELD REPRESENTATIVE: Beauford Jones

PROJECT: Proposed 1550 Prospect Apartments
PROJECT LOCATION: 1550 North Prospect Avenue
Milwaukee, Wisconsin
GILES PROJECT NUMBER: 1G-1310022

GILES ENGINEERING ASSOCIATES, INC.
Atlanta Dallas
Los Angeles Milwaukee
Orlando Washington, D.C.

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u **(tsf)**	q_p **(tsf)**	q_s **(tsf)**	w **(%)**	PID	NOTES
Gray Silty Clay, trace fine Sand-Moist *(continued)*		11-SS	20				17		(a)
Gray Silty Clay to Clay-Moist									
	45	12-SS	22	2.8	2.5		22		
	50	13-SS	28	3.0	2.5		22		
(Contains Gray Silt Lense or Seam at 61 Feet)									
	55	14-SS	15	3.0	2.5		23		
	60	15-SS	11	4.1	3.2		22		

Boring Terminated at 61 Feet

	WATER OBSERVATION DATA	REMARKS
▽	WATER ENCOUNTERED DURING DRILLING: None	(a) Poor Recovery
▼	WATER LEVEL AFTER REMOVAL: None	
▨	CAVE DEPTH AFTER REMOVAL: 57.0 ft.	
▼	WATER LEVEL AFTER HOURS:	
▨	CAVE DEPTH AFTER HOURS:	

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.

RECORD OF SUBSURFACE EXPLORATION

BORING NO. & LOCATION: H3	PROJECT: Proposed 1550 Prospect Apartments
SURFACE ELEVATION: 79.7	PROJECT LOCATION: 1550 North Prospect Avenue
COMPLETION DATE: 11/21/13	Milwaukee, Wisconsin
FIELD REPRESENTATIVE: Beauford Jones	GILES PROJECT NUMBER: 1G-1310022

GILES ENGINEERING ASSOCIATES, INC.

Atlanta Dallas
Los Angeles Milwaukee
Orlando Washington, D.C.

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u (tsf)	q_p (tsf)	q_s (tsf)	w (%)	PID	NOTES
4"± Asphalt Pavement		1-SS	8						
8"± Aggregate Base Course									
Gray Silty Clay, trace Gravel, trace Wood Fragments (Fill)-Moist		2-SS	9	5.1	4.5+		17		
Brown Silty Clay, trace fine to coarse Sand and Gravel (Contains Calcareous Deposits in Fissures)-Moist to Damp	5	3-SS	15	8.6	4.5+		16		
		4-SS	20	6.1	4.5+		17		
Brown to Gray Silty Clay, trace fine Sand-Moist	10	5-SS	15	4.2	4.2		18		
	15	6-SS	12	3.5	3.2		19		
Gray Silty Clay, trace fine Sand-Moist									
	20	7-SS	12	4.3	3.5		18		
	25	8-SS	10	3.0	3.2		21		
	30	9-SS	15	4.7	3.7		17		

Boring Terminated at 31 Feet

	WATER OBSERVATION DATA	REMARKS
▽	WATER ENCOUNTERED DURING DRILLING: None	
▼	WATER LEVEL AFTER REMOVAL: None	
▨	CAVE DEPTH AFTER REMOVAL: 26.0 ft.	
▼	WATER LEVEL AFTER HOURS:	
▨	CAVE DEPTH AFTER HOURS:	

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.

RECORD OF SUBSURFACE EXPLORATION

GILES ENGINEERING ASSOCIATES, INC.

Atlanta Dallas
Los Angeles Milwaukee
Orlando Washington, D.C.

BORING NO. & LOCATION: H4

SURFACE ELEVATION: 79.3

COMPLETION DATE: 11/21/13

FIELD REPRESENTATIVE: Beauford Jones

PROJECT: Proposed 1550 Prospect Apartments

PROJECT LOCATION: 1550 North Prospect Avenue
Milwaukee, Wisconsin

GILES PROJECT NUMBER: 1G-1310022

MATERIAL DESCRIPTION	Feet Below Surface	Sample No. & Type	N	q_u **(tsf)**	q_p **(tsf)**	q_s **(tsf)**	w **(%)**	PID	NOTES
4"± Asphalt Pavement		1-SS	7	1.0	1.0	0.4	24		
8"± Aggregate Base Course									
Black to Dark Gray Clayey Silt, trace fine Sand (Fill)-Moist		2-SS	8	2.7	2.2		26		
Brown Silty Clay, trace fine to coarse Sand and Gravel (Contains Calcareous Deposits in Fissures)-Moist to Damp	5	3-SS	17	5.5	4.5+		17		
		4-SS	25				15		(a)
Brown to Gray Silty Clay, trace fine Sand-Moist	10	5-SS	18	6.8	4.5+		17		
Gray Silty Clay, trace fine Sand-Moist	15	6-SS	9	4.0	3.2		18		
	20	7-SS	9	2.6	2.5		20		
	25	8-SS	10	2.8	2.5		20		
	30	9-SS	16	3.3	2.7		19		

Boring Terminated at 31 Feet

WATER OBSERVATION DATA

▽ WATER ENCOUNTERED DURING DRILLING: None

▼ WATER LEVEL AFTER REMOVAL: None

▨ CAVE DEPTH AFTER REMOVAL: 28.0 ft.

▼ WATER LEVEL AFTER HOURS:

▨ CAVE DEPTH AFTER HOURS:

REMARKS

(a) Poor Recovery

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

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.

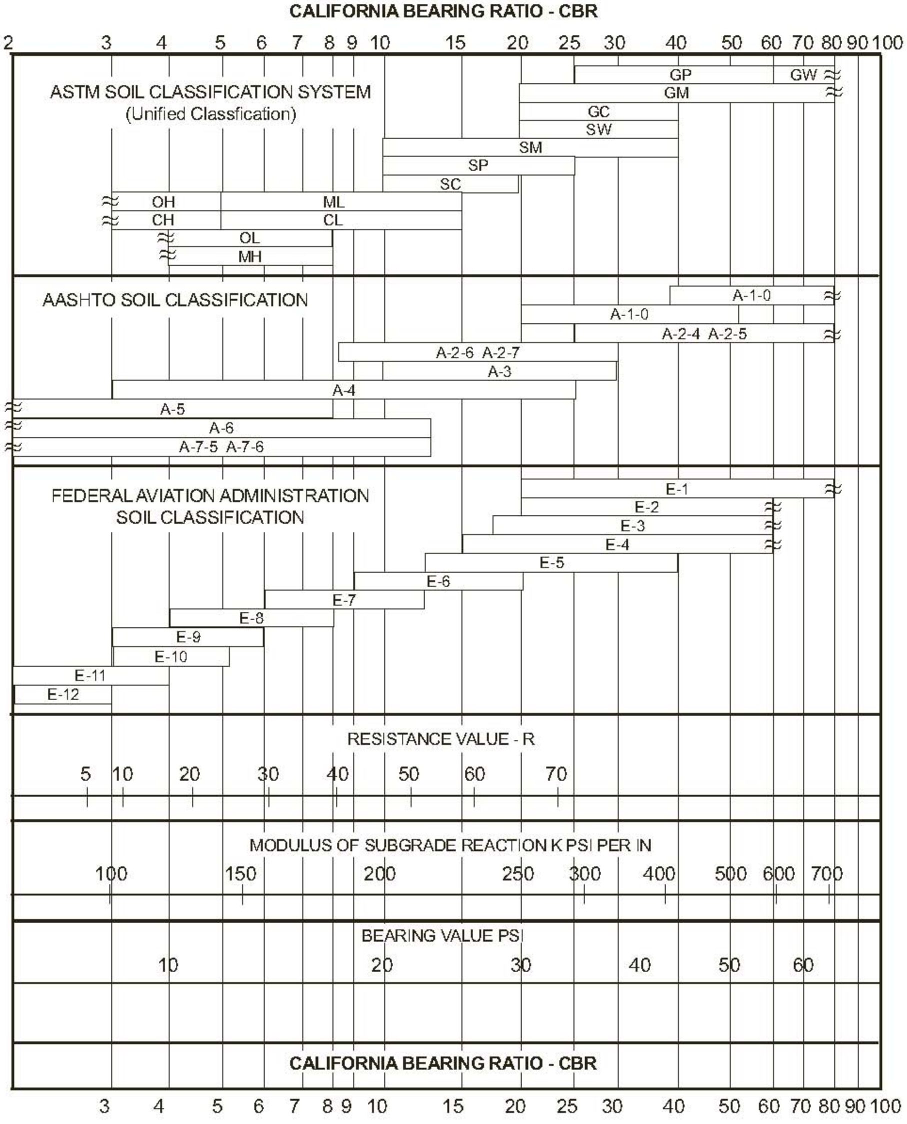
Laboratory Testing

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

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.

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.

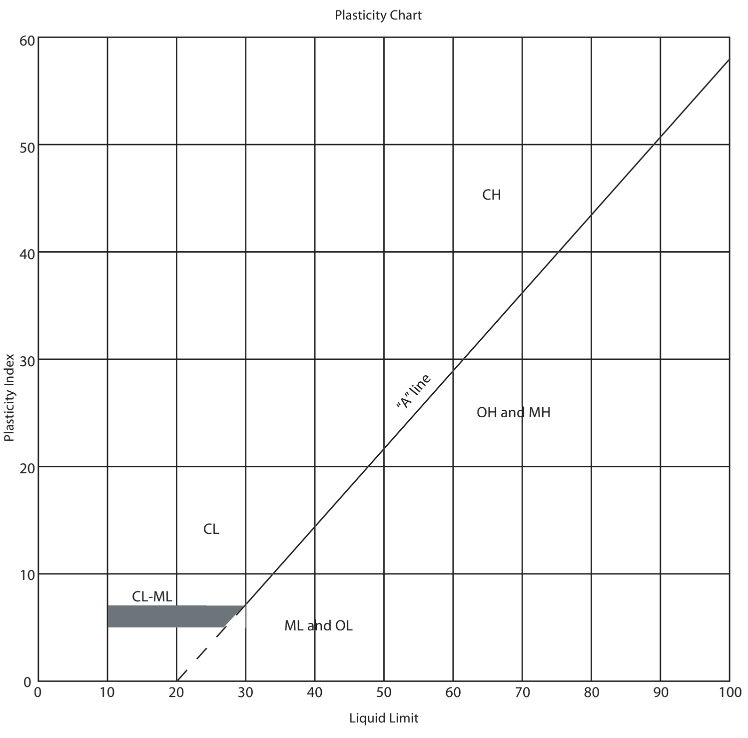
CHARACTERISTICS AND RATINGS OF UNIFIED SOIL SYSTEM CLASSES FOR SOIL CONSTRUCTION *

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			
			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			

---

[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

ATLANTA, GA
(770) 458-3399

DALLAS, TX
(214) 358-5885

LOS ANGELES, CA
(714) 279-0817

MILWAUKEE, WI
(262) 544-0118

ORLANDO, FL
(407) 321-5356

TAMPA, FL
(813) 283-0096

BALTIMORE/WASHINGTON, D.C.
(410) 636-9320