

GEOTEK

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GEOTECHNICAL INVESTIGATION

Love Circle Windmills
Nashville, Tennessee

for

Total Quality Environmental
Cookeville, Tennessee

GPN: 11-6634-A
April 11, 2011

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April 11, 2011

Total Quality Environmental
125 Security Place
Cookeville, TN 38506

ATTN: Dan Dyer


**SUBJ: Geotechnical Report
Love Circle Windmills
Nashville, Tennessee
GPN: 11-6634-A**

Dear Mr. Dyer:

Geotek Engineering Company, Inc. (Geotek) is pleased to submit the attached report documenting the geotechnical investigation for the above-referenced project. Should you have any questions or need additional information, feel free to call.

Sincerely,

GEOTEK ENGINEERING COMPANY, INC.



Kenneth L. McCurdy, P.E.
Senior Engineer



John Rami Mishu, P.E., P.G.
Principal Engineer

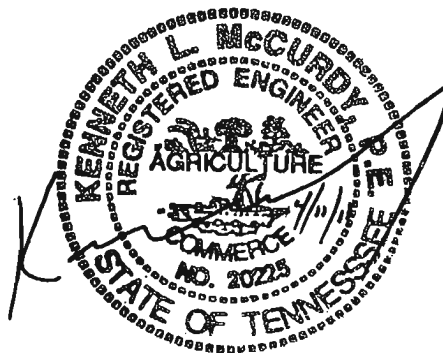


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GEOTECHNICAL INVESTIGATION

Love Circle Windmills

Nashville, Tennessee

1.0 INTRODUCTION

1.1 General

This report presents the results of our geotechnical investigation for two windmill towers at the Vanderbilt Love Circle Renewable Technologies Center in Love Circle Park, Nashville, Tennessee as shown in Figure 1. The site's location on the USGS topographic map is shown in Figure 2. The project site is shown on an aerial photograph in Figure 3. The general site layout is shown in Figure 4.

This work was authorized by Dan Dyer of Total Quality Environmental and was carried out in general accordance with our proposal letter dated October 18, 2010.

1.2 Scope of Services

The purpose of this investigation was to obtain site-specific subsurface data and to provide certain geotechnical recommendations for the proposed project. The complete scope of geotechnical services provided is comprised of the following:

- Performing a field investigation of the subsurface soil by test-pits.
- Conducting a reconnaissance of the site and immediate area including an evaluation of rock outcrops and topographic features.
- Reviewing readily available maps and other publications of the regional geology.
- Performing appropriate laboratory tests of samples obtained from the site.
- Providing recommendations for foundations best suited to the windmill towers and subsurface conditions, including allowable bearing capacities and anticipated settlements.

2.0 METHODS OF INVESTIGATION

2.1 Field Exploration

The field phase of work was conducted on March 15, 2011. The field study at the site included two test-pits. Their approximate locations are shown in Figures 3 and 4.

The field test-pit locations were staked by Geotek field personnel who measured distances and visually estimated angles from recognizable property features and other evident landmarks shown on drawings provided by Total Quality Environmental. The test-pit locations noted in this report should be considered approximate in view of the methods used.

The test-pits were opened using a backhoe to excavate a narrow trench approximately 2 ft wide by 6 ft long and to the required depths. These open excavations were then examined by a member of our professional staff who logged the observed stratigraphy, photographed the trench, and obtained bag samples of the excavated soils. These samples were transported to our laboratory for evaluation

and testing. The test-pits were subsequently backfilled in semi-compacted layers to the original surface to the extent possible. If this test-pit backfill material is exposed during subsequent construction in the area, it might not be as stable as needed, and might require some minor stabilization.

Water-level readings were taken in the test-pits at the time of trenching. Groundwater levels may fluctuate due to recent rainfall, seasonal conditions, construction activity, and other site-specific factors. Since these conditions may change with time, the water-level information presented on the test-pit logs represents the conditions only at the time each measurement was taken.

Final test-pit logs are included in Appendix A of this report. Field logs, laboratory data, and visual examination of samples in our lab were used in preparation of these final logs. These logs represent only those interpretations and descriptions that can be made of the conditions encountered in the specific locations investigated at that time.

2.2 Laboratory Testing

Laboratory tests were performed on selected soil samples estimated to be most representative of *in-situ* conditions. Test procedures used were in general accordance with specifications of the American Society of Testing and Materials (ASTM) and are described briefly herein. All test data are included in Table B-1 found in Appendix B of this report.

Natural moisture content tests (ASTM D2216) were performed on most samples. The results of these tests are listed in Table B-1 along with descriptions of the samples.

Atterberg limits tests were performed to determine the plasticity characteristics of the soil (ASTM D4318). The soil's Plasticity Index (PI) is representative of this characteristic, and is bracketed by the Liquid Limit (LL) and the Plastic Limit (PL). Results of these tests are shown in Table B-1. The Atterberg Limits are primarily classification tools, but when considered in conjunction with natural moisture content, provide an indirect evaluation of soil consistency and volume-change potential.

In conjunction with the classification testing program, all soil samples were examined in our laboratory and described per ASTM D2488. If Atterberg Limits and gradation test data are available, the samples are also classified on the basis of their grain-size, texture, and plasticity in accordance with the Unified Soil Classification System (USCS). The soil descriptions are in conformance with this methodology, and the estimated group symbols (according to the USCS) are included in parentheses following the soil descriptions.

3.0 SUBSURFACE CONDITIONS

3.1 General Geology

The project site lies in the Central Basin Physiographic Province of Middle Tennessee. The Central (or Nashville) Basin is an oval-shaped topographic depression approximately located in the geographic center of the state. The terrain is gently rolling to hilly with some nearly level areas.

Elevations range from 500 to 700 ft, but many isolated "knobs" and associated ridges rise 200 to 400 ft above the general level of the basin, most commonly along the outer part of the basin. The Central Basin was formed as a result of erosion of the gently arched Nashville Dome and is underlain by essentially flat-lying limestones, siltstones, and shales of Ordovician age. This erosion has been facilitated by the solutional removal of the calcium carbonate rock, which occurs most rapidly along rock joints. Many areas of the Central Basin are also characterized by a lack of surface drainage and by Karst features such as caves and sinkholes. The bedrock typically weathers in-place into a cohesive "residual" soil. Additionally, the knobs are generally capped by more weather resistant Mississippian- and Devonian-aged siltstone, chert, and shale bedrock.

The site's near-surface geology is dominated by the Leipers-Catheys Formation of Ordovician age. The rock of this formation is primarily a limestone that is argillaceous, nodular, and shaley with some cross-bedding and zones of brown phosphate pellets. The grain varies from fine- to coarse-grained, and the color varies from light- to dark-gray but is sometimes brown-gray. The upper 3 to 10 feet of the bedrock is often highly weathered with a high probability of floating boulders, weathered rock layers, clay seams, and some soil-filled crevices. Soil seams between the upper beds of the rock unit are also common. The Leipers-Catheys Formation is sometimes associated with karst development and has the potential for sinkhole development.

The soil residuum over the Leipers-Catheys Formation is typically 1- to 10-ft thick. It is largely derived from the physical and chemical weathering of the "parent" bedrock. As a result of this weathering, the soil has retained some of the original structure and sub-structure of the bedrock. Typically, this consists of a very fine-grained soil matrix (silt or clay) having a complex sub-structure of boulders, chert fragments, chert beds, sand lenses, and shale layers.

3.2 Site Conditions

The project site is on the side of a sloping mound at the top of Love Circle Park. The mound was created by construction, and subsequent backfilling, of an underground water reservoir at the top of a natural knoll as detailed in Figure 5. This figure was taken from plans for the "Nashville Water Department High Service Reservoir," dated November 1925, and depicts the original and proposed ground-surface profiles and the proposed reservoir structure. The project site is located on the side-slope of the mound. Based on a comparison between the original ground surface and the planned fill surface shown in Figure 5, it appears that about 8 feet of fill would have been required at the location now planned for the windmills. The current ground surface at the site is moderately sloping with approximately 5 ft of relief from east to west over a distance of about 15 ft. Topographic contours are shown in Figures 2 and 5. The site contains a maintained, grass-covered lawn for ground cover.

The site is bounded to the west by an approximate 4-ft high concrete retaining wall at the bottom of the slope and by Love Circle Drive below the wall. An existing weather station tower is located south of the project site. Photographs of the site are included following the figures and preceding Appendix A.

3.3 Subsurface Stratigraphy

The subsurface profile described herein is based primarily on the field and laboratory test data found in this report. Appendix A contains the individual test-pit logs. Appendix B includes the laboratory test data. A generalized subsurface profile is summarized as follows.

The surficial soil stratum at the test-pit locations was a 6-in. thick topsoil layer.

Below the surficial topsoil layer, the test-pits encountered fill consisting of a mixture of primarily limestone rock fragments with silty clay and topsoil to a depth of about 8.0 feet. This finding is consistent with the fill thickness that would have been required to raise the grade at the project location from the original ground surface to the planned fill surface, as shown in the historical plans in Figure 5. The limestone rock fragments comprise an estimated 60 to 90 percent of the fill and generally range in size from gravel-size to 24-in. boulders. The silty clay and topsoil are generally soft and contain some coal ash and cinder materials. The fill appears to be relatively loose with some voids between the rock fragments.

Below the rock-soil fill mixture, Test-Pit TP-2 encountered a 1.5-ft thick layer of soft residual silty clay. Below the rock-soil fill mixture in TP-1 and the residual soil layer in TP-2, the test-pits encountered a 1.0- to 1.5-ft thick layer of highly-weathered limestone bedrock with soft clay layers.

Atterberg Limits testing yielded Liquid Limits of 36 to 52 percent and Plasticity Indices of 15 to 19 percent, indicating that the native soils and soil matrix in the fill are clays of moderate to high plasticity. Based on these test results, the soils can be classified to fall primarily in the "CL" and "MH" USCS groups. Detailed soil descriptions are provided in Table B-1 and the test-pit logs.

Compressive strength values from the Index-Penetration tests ranged between 0.5 and 3.0 ksf, indicating very-soft to stiff consistency native soils and soil matrix within the fill. However, apparent strength values from the relationship between the Plasticity-Index and Moisture-Content would suggest stiffer consistencies. These discrepancies are likely due to (1) the presence of limestone fragments in the soil and/or (2) loosely-compacted soil within the layer of fill.

Refusal was encountered on limestone bedrock at depths of 9.0 and 11.0 feet in the test-pits. Refer to the test-pit logs in Appendix A and the Table B-1 in Appendix B for more information.

Backhoe refusal is defined as the depth at which the backhoe can no longer deepen a narrow test-pit because of a similar obstruction. Rock coring procedures are generally required to determine

the character and continuity of the refusal material; and these factors must be considered when evaluating the depth to refusal.

Groundwater was not encountered at the time of trenching. The groundwater level at this site, at time studied, appears to have been below the depths investigated, although isolated perched conditions may have existed. Based upon our area experience, we believe that groundwater activity is most likely limited to the fill/native-soil or soil/bedrock interfaces and the deeper rock crevices and fractures.

We emphasize that (a) cohesive soils such as found at this site are of low permeability and require a long time to yield water and (b) groundwater levels will vary depending on seasonal and climatic conditions, on construction activities, and on other site-specific factors.

4.0 COMMENTS AND RECOMMENDATIONS

4.1 Project Information

The geotechnical comments and recommendations presented in this report are based on (a) the results of the subsurface investigation and laboratory testing program presented herein and (b) site layout and structural information provided by Total Quality Environmental.

We received a proposed site layout drawing prepared by Total Quality Environmental dated February 4, 2011. We also received foundation details for the wind tower foundations prepared by Clark Engineering Company dated April 2, 2010.

We understand that the planned construction consists of two 60-ft high windmills (wind towers). The windmill foundations will generally consist of approximately 16-ft square spread footings with concrete piers.

Our geotechnical recommendations are highly sensitive to changes in proposed structural loads, foundation location, and subgrade elevations. These assumptions should be verified and reviewed prior to the final design, and appropriate changes made in the recommendations.

4.2 Windmill Tower Foundations

Spread footing foundation systems are suitable assuming they will be designed, constructed, and monitored in accordance with the recommendations presented herein. We recommend that the footings be founded entirely on the limestone bedrock encountered at depths of 9 to 11 feet below the ground surface on the upslope sides of Test-Pits TP-1 and TP-2 (6.5 to 8 feet below the downslope side of the test-pits). The footings for the windmill towers should be designed using a maximum allowable bearing pressure of 6,000 psf.

Due to the loose fill and underlying soft native soils at the site, we do not recommend founding the spread footings on the existing fill, native soils, or highly-weathered bedrock with soil seams.

Where the foundation excavation exposes existing fill, native soils, or layered weathered bedrock, we recommend that the excavation be undercut to intact solid bedrock. The spread footing can then be constructed directly on the bedrock.

The lateral resistance of the foundation should be taken as the sum of the sliding resistance beneath the footing and the net passive resistance against the face of the footing. To calculate the resistance to sliding, a value of 0.40 should be used as the ultimate coefficient of friction between the footing and the underlying bedrock. To calculate the net passive resistance ($k_p - k_a$) provided by the adjacent soil or bedrock against one face of the foundation, we recommend that any lateral resistance provided by the upper 2 feet of the surrounding earth be ignored. For this project, the net passive resistance can be taken as an equivalent fluid pressure of 150 pcf. This assumes that the concrete for the foundation is cast directly against the in-situ earth. We recommend that a minimum factor of safety of 2 be applied to the lateral resistance computed in this manner.

If the concrete foundation is formed and not cast directly against the in-situ earth, we recommend that either (1) the excavation be carefully backfilled with compacted densely-graded crushed stone placed and compacted in accordance with the recommendations presented for new fill in the following section of this report or (2) the net passive resistance against the face of the footing be ignored.

Uplift loads should be resisted by the weight of the foundation and any overlying backfill above the foundation. We recommend that a factor of safety of at least 1.5 be applied to the resistance provided by the weight of the foundation and soil directly above the foundation.

Prior to placement of concrete, the subgrade bearing area should be clean and free of loose soil, debris, and ponded water. The footing excavation should be observed by the geotechnical engineer responsible for design to confirm that conditions appear to reasonably comply with assumed design parameters. Any loosened rock or soft zones encountered should be undercut to solid sandstone bedrock.

Exposure of the footing subgrade to ponded water for even short periods can weaken the subgrade and measurably increase settlements. We therefore recommend that concrete be placed immediately after footing excavation where practical. Otherwise, some undercutting or mud-matting with lean concrete (min. 800 psi) may be needed to protect or repair the subgrade prior to footing construction.

4.3 Earthwork

General - We recommend that all construction operations dealing with earthwork and foundations be observed by experienced geotechnical engineering personnel from our office to verify that the design requirements are fulfilled in the actual construction and to make on-site

recommendations to minimize delays. Please also read "Field Observation - A Message to Clients" found in Appendix C of this report.

Positive site drainage should be established as the first order of work and should be maintained at all times during and after construction to minimize accumulations of water within the footing excavation.

Soil Compaction - Where new fill is required beside or over the tower foundations, we recommend the use of an approved soil with an acceptable plasticity and that is free of organics and debris. This fill material should be compacted in 6-inch thick lifts to a minimum of 95 percent of the Standard Proctor maximum dry density (ASTM D698) and to within ± 2 percent of optimum moisture.

Reduced lift thicknesses may be required for compaction using hand-operated tampers or for "crusher run" stone fills. These compaction recommendations should apply to all miscellaneous backfill (e.g., in utility trenches, narrow backfill zones beside the footing, etc.). At least one density test should be performed per 6-in. compacted lift.

Borrow Soils - Because of the size of the rock fragments in the existing fill and weathered limestone at the site, most of the *in-situ* fill and weathered limestone will not be suitable for re-use as borrow in construction of engineered backfill over or around the windmill foundations. Proper placement of the rock fill would require compaction with large bulldozers and would not be practical for backfilling limited spaces. The limited amount of native soil that might be available could be suitable for use as backfill, but wetting or drying of these soils may be required in order to meet compaction recommendations. Additionally, the use of imported fill, such as crushed base-stone, should be considered during inclement weather conditions depending on construction plans.

Drainage - We reiterate that proper drainage should be provided on the site at all times. Foundation subgrades softened by perched water must be undercut to undisturbed limestone.

Water was not encountered in the test-pits at the time of trenching. However, increased water activity, particularly at the fill-soil and soil-rock interfaces should be anticipated during rainy seasons, and some groundwater infiltration into the footing excavation should therefore be expected. We recommend that the groundwater level be verified just prior to construction.

Excavations and Temporary Slopes - In no case should slope height, slope inclination, or excavation depth (including utility trench excavation depth) exceed those specified in local, state, and federal safety regulations. Specifically, the current OSHA Health and Safety Standards for Excavations (29 CFR Part 1926) should be followed. It is our understanding that these regulations are being strictly enforced.

The temporary stability of all earthen (soil and rock) sideslopes and trenches should be the responsibility of the contractor. For example, temporary slopes in soil may require inclinations of 1(H):1(V) or flatter. Bracing and shoring systems may also be necessary. We would be happy to assist with the design of such earth retention systems or to evaluate temporary slope stability.

5.0 GENERAL QUALIFICATIONS

General - This report has been prepared to assist the engineers with the design of this project. The investigation and recommendations were made in accordance with generally accepted standards and practices of the geotechnical engineering profession (see ASFE publication in Appendix C of this report). The scope is limited to the specific project and location described herein, and our description of the project represents our understanding of the significant aspects relevant to geotechnical engineering considerations.

Construction Monitoring - The recommendations contained in this report were made under the assumption that we will be retained to provide the necessary monitoring of foundation construction. If not, whoever assumes these responsibilities should understand the philosophy of our recommendations, thoroughly review all of our reports, and accept the responsibility of being the Geotechnical Engineer-of-Record. Please also read the "ASFE Publication" and "Field-Observation Memorandum" in Appendix C.

Subsurface Variations - The analysis and recommendations submitted in this report are based upon (a) the data obtained from the test-pits performed at the locations indicated on the location diagram and (b) the other information discussed in this report. In the performance of subsurface explorations, specific information is obtained at specific locations of specific times. It is a well-known fact, however, that variations in subsurface conditions exist on most sites between test-pit locations and with time, particularly with respect to groundwater and with respect to sites that contain random fill. The nature and extent of most variations may not become evident until revealed in the course of construction. If these variations then appear, a re-evaluation of the recommendations of this report may be necessary after performing on-site observations during construction and noting the characteristics of any variations. Other subsurface conditions affecting project performance may not appear until their effect is noticed after construction is completed (see ASFE publication).

Because unanticipated subsurface conditions may occur, the designers should consider including a "changed condition" clause in their contracts both with the general contractor and with sub-contractors involved in earthwork and foundation construction. In some cases, we believe that the inclusion of this clause will permit contractors to give lower prices because they will not need to provide as much in contingencies as they normally would. Equitable adjustment of changed conditions can minimize conflicts and litigation, with the attendant delays and costs. Furthermore, by immediately

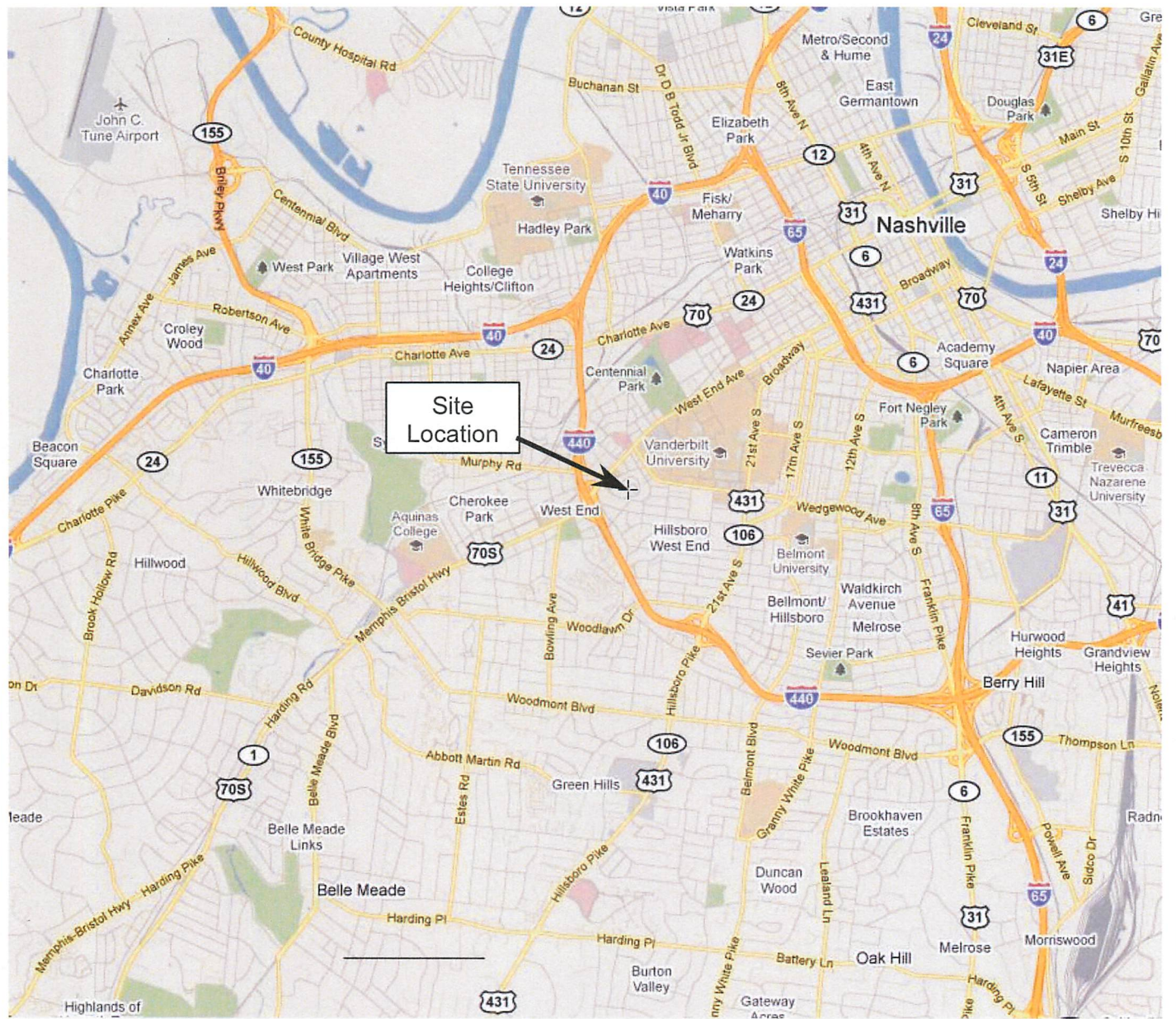
recognizing and adjusting the contract price at the time the changed conditions are encountered, the problem of trying to recreate facts (should litigation develop later) is eliminated. A suggested wording for a changed conditions clause is given in Appendix C of this report.

Safety - The owner and the contractor should make themselves aware of, and become familiar with, applicable local, state, and federal safety regulations, including the current OSHA excavation safety standards. Construction site safety is generally the sole responsibility of the contractor, who shall also be solely responsible for the means, methods, and sequencing of construction operations. No information provided in this report should be interpreted to mean that we are assuming responsibility for construction site safety or the contractor's activities; such responsibility is not being implied and should not be inferred.

Environmental Issues - The scope of our services did not include any environmental assessment or investigation for the presence (or absence) of wetlands or hazardous/toxic materials in the soil, surface water, groundwater or air, either on, below, or around this site. Any statements in this report or on the test-pit logs regarding odors noted or unusual or suspicious items or conditions observed are strictly for the information of our client.

Design Review - Finally, we recommend that we be authorized to review the project plans and specifications to confirm that the recommendations contained in this report have been interpreted in accordance with our intent. Without this review, we cannot be responsible for misinterpretation of our data, our analysis, and our recommendations, or how these are incorporated into the final design.

FIGURES



Site Location Map

Love Circle Windmills
Nashville, Tennessee

Geotek Proj. No. 11-6634-A

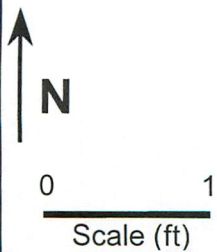


Figure 1



Source: USGS Topographic Map, Nashville West Quadrangle, Tennessee (1998).

USGS Topographic Map

Love Circle Windmills
Nashville, Tennessee

Geotek Proj. No. 11-6634-A



0 4000

Scale (ft)

Figure 2



Test-Pit Locations on Aerial Photo

Love Circle Windmills
Nashville, Tennessee

Geotek Proj. No. 11-6634-A



0 50
Scale (ft)

Figure 3

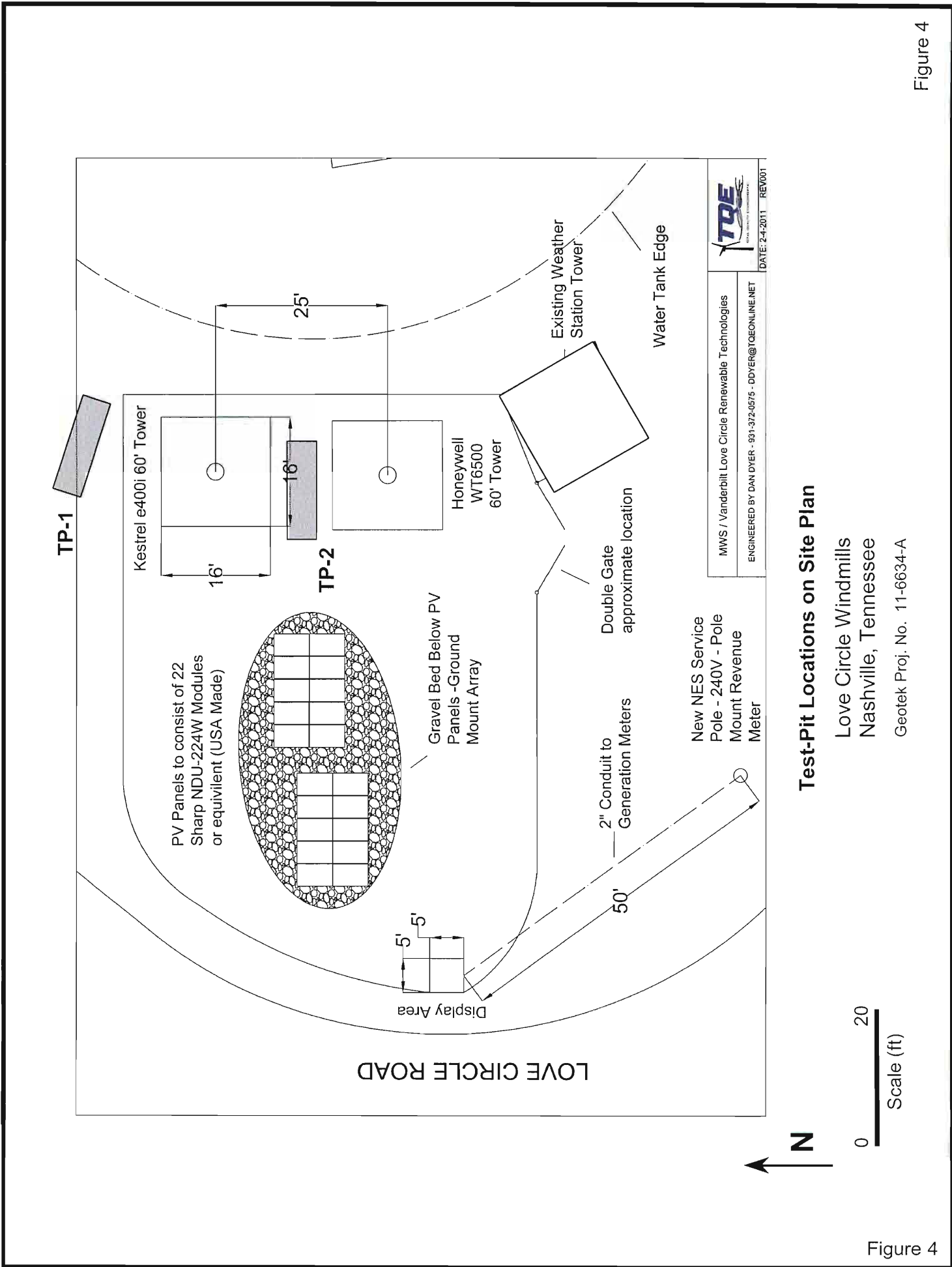


Figure 4

Figure 4

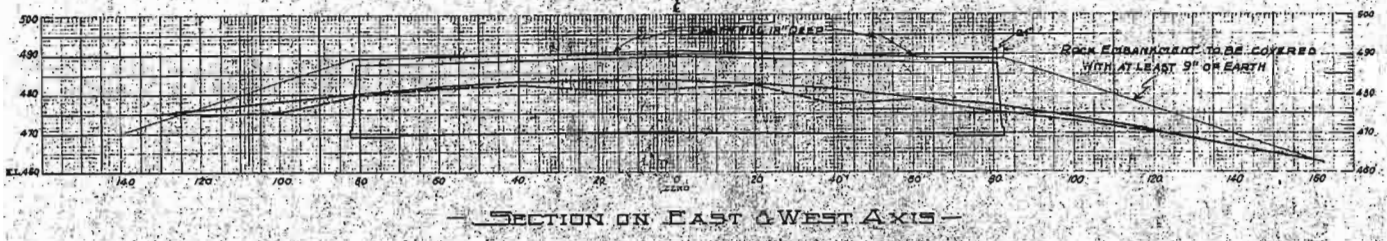


Figure 5a: East-West Cross-Section Showing Original Ground and Surrounding Fills

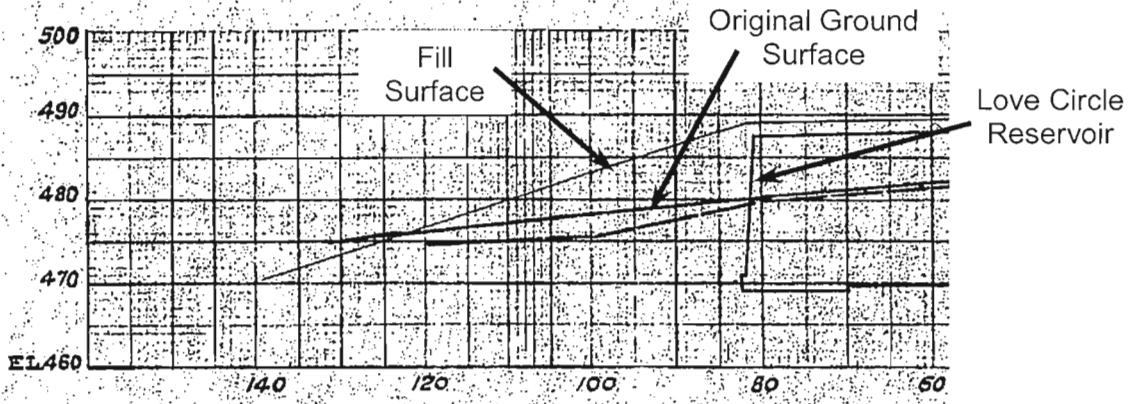


Figure 5b: Western End Cross-Section Showing Original Ground and Surrounding Fills

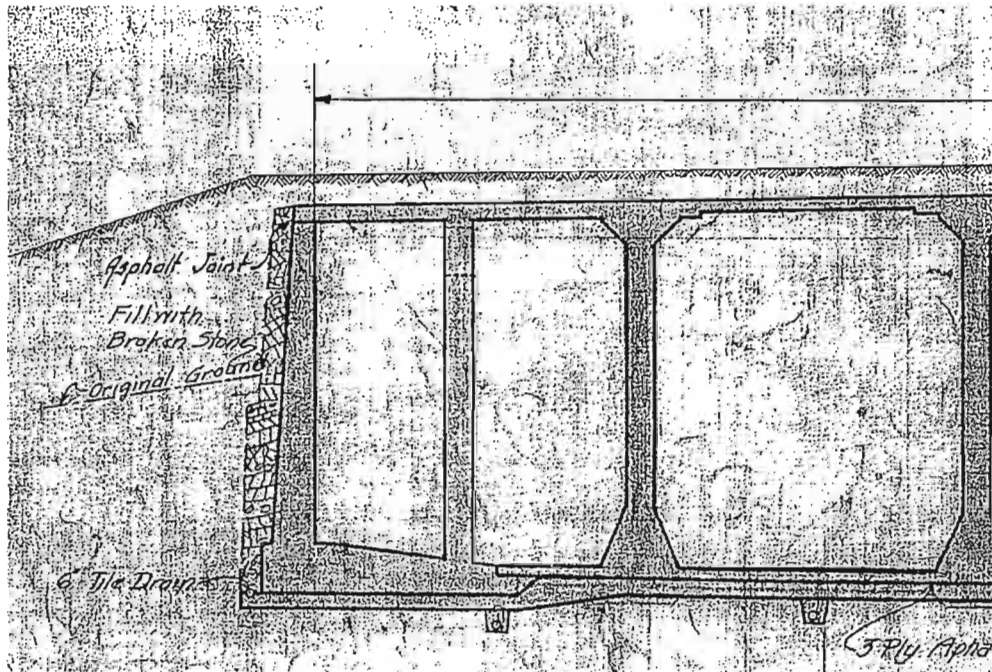


Figure 5c: Western Cross-Section Showing Original Ground, Reservoir Structure, and Surrounding Fills

Historic Plans of the Love Circle Reservoir

Love Circle Windmills
Nashville, Tennessee

Geotek Proj. No. 11-6634-A

Source: Nashville Water Department High
Service Reservoir, November 1925.

Figure 5

SITE PHOTOGRAPHS



Photo 1: The Love Circle site is shown in this panorama photograph. The arrows point to the approximate locations for the planned windmill towers. An underground water reservoir is located inside the hilltop. The construction of the approximately 20 ft deep reservoir was apparently about half into the hill and about half behind a wedge of fill.



Photo 2: This photo shows the backhoe opening Test Pit TP-1. The dashed line shows the approximate location of the edge of the below ground tank. Per the 1925 plans, about 18 inches of soil caps the tank top. The fill slope is about 3:1 to 4:1 (H:V).

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Photo 3: The fill encountered in Test Pit TP-1 was loose and when placed back into the excavation in compacted layers, left this drop off due to not having enough material to level the ground at the test-pit. The drop off was about 1.5 ft tall on the uphill side.



Photo 4: The Test-Pit TP-1 was finished off with about 5 yd³ of topsoil brought in from off site.

APPENDIX A

TEST-PIT LOGS LOG CLASSIFICATION SYSTEM

LOG OF TEST PIT

Test Pit No: TP-1

Project: Love Circle Windmills
Site Location: Nashville, Tennessee
Project No.: 11-6634-A
Client: Total Quality Environmental
Cookeville, Tennessee

Logged By: Stephen Capps
Date: March 15, 2011
Location: See Figures 3 & 4

Depth (ft)	Description
0 - 0.5 ⁽³⁾	TOPSOIL, brown (Fill)
0.5 - 8.0 ⁽³⁾	ROCK-SOIL FILL MIXTURE (about 90% of fill is rock). Rock portion is limestone ranging from gravel to small boulder size. Larger pieces of the rock are primarily in small slab pieces of 8- to 24-in. size by several inches thick. Soil portion is brown silty clay or topsoil. Some coal ash and cinder materials were seen at the bottom of the fill. The fill appears to be loose and some small voids exist between the small slab pieces.
8.0 - 9.0 ⁽³⁾	WEATHERED LIMESTONE ROCK w/ silty clay soil, brown Rock was layered and appeared to be in undisturbed layers overlying bedrock.
9.0 ⁽³⁾	REFUSAL on Limestone Bedrock, slightly sloping downhill

- Notes:
1. No groundwater encountered during the excavation of the test-pit.
 2. Photos of the test-pit are on the following pages.
 3. The above depths are from the upper end of the test-pit. Refusal on the downhill end was at about 6.5 ft. The ground surface was estimated to slope at 3:1 to 4:1 (H:V).
 4. The fill was apparently loose as was confirmed when the material was packed back into the test-pit excavation with the backhoe bucket. There was not enough material to fill the pit to the original ground level. About 5 yd³ of topsoil was brought from offsite to fill the test-pit to the original ground surface level (see Photos 3 and 4).



Test Pit TP-1

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Nashville, Tennessee
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Test Pit TP-1

Love Circle Windmills
Nashville, Tennessee
GPN: 11-6634-A

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LOG OF TEST PIT

Test Pit No: TP-2

Project: Love Circle Windmills
Site Location: Nashville, Tennessee
Project No.: 11-6634-A
Client: Total Quality Environmental
Cookeville, Tennessee

Logged By: Stephen Capps
Date: March 15, 2011
Location: See Figures 3 & 4

Depth (ft)	Description
0 - 0.5 ⁽⁴⁾	TOPSOIL, brown (Fill)
0.5 - 8.0 ⁽⁴⁾	ROCK-SOIL FILL MIXTURE (about 60% of fill is rock). Rock portion is limestone ranging from gravel to small boulder size. The rock was primarily cobbles (3- to 6-in. sizes) with some larger pieces being small slab pieces of 6- to 18-in. size by several inches thick. Soil portion is SILTY CLAY w/ fine sand traces, chert and limestone fragments, yellow gray-brown, dark-brown, and black, moist and generally soft (pp=0.25-1.5 tsf). The fill appears to be loose.
8.0 - 9.5 ⁽⁴⁾	SILTY CLAY w/ fine sand traces, chert and limestone fragments, yellow-brown, moist and soft (pp=0.75)
9.5 - 11.0 ⁽⁴⁾	WEATHERED LIMESTONE ROCK w/ SILTY CLAY, fine sand traces, yellow-brown, moist and soft (pp=0.5 tsf). Rock was layered and appeared to be in undisturbed layers overlying bedrock.
11.0 ⁽⁴⁾	REFUSAL on Limestone Bedrock, slightly sloping downhill

- Notes:
1. No groundwater encountered during the excavation of the test-pit.
 2. Photos of the test-pit are on the following page.
 3. pp = Pocket Penetrometer reading (indicates soil consistency). This is not allowable bearing value.
 4. The above depths are from the upper end of the test-pit. Refusal on the downhill end was at about 8.0 ft. The ground surface was estimated to slope at 3:1 to 4:1 (H:V).
 5. The fill was apparently loose but slightly denser than the fill encountered in Test-Pit TP-1. About 2 yd³ of topsoil was brought from offsite to fill the test-pit to the original ground surface level.



Test Pit TP-2

Love Circle Windmills
Nashville, Tennessee
GPN: 11-6634-A

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CLASSIFICATION SYSTEM FOR SOIL EXPLORATION

NON-COHESIVE SOILS

(Silt, Sand, Gravel, and Combinations)

<u>DENSITY</u>	<u>SPT N-VALUE</u>	<u>PARTICLE SIZE IDENTIFICATION</u>
Very Loose	5 blows/ft or less	Boulders 8 inch diameter or more
Loose	6 to 10 blows/ft	Cobbles 3 to 8 inch diameter
Medium Dense	11 to 30 blows/ft	Gravel Coarse - 1 to 3 inch
Dense	31 to 50 blows/ft	Medium - 1/2 to 1 inch
Very Dense	51 blows/ft or more	Fine - 1/4 to 1/2 inch
		Sand Coarse - 0.6 mm to 1/4 inch (dia. of pencil lead)
		Medium - 0.2 mm to 0.6 mm (dia. of broom straw)
		Fine - 0.05 mm to 0.2 mm (dia. of human hair)
		Silt 0.06 mm to 0.002 mm (cannot see particles)
<u>RELATIVE PROPORTIONS</u>		
<u>Descriptive Term</u>	<u>Percent</u>	
Trace	1 - 10	
Little	11 - 20	
Some	21 - 35	
And	36 - 50	

COHESIVE SOILS

(Clay, Silt, and Combinations)

<u>CONSISTENCY</u>	<u>SPT N-VALUE</u>	<u>PLASTICITY</u>
Very Soft	3 blows/ft or less	<u>Degree of Plasticity</u>
Soft	4 to 6 blows/ft	<u>Plasticity Index (PI)</u>
Medium Stiff	7 to 12 blows/ft	Low 0 - 7
Stiff	13 to 20 blows/ft	Medium 8 - 22
Very Stiff	21 to 35 blows/ft	High over 22
Hard	35 blows/ft or more	

Classification - On logs are made by visual inspection in general accordance with the Unified Soil Classification System.

Standard Penetration Test (SPT) - Driving a 2.0" O.D., 1 3/8" I.D., sampler a distance of 1.0 foot into undisturbed soil with a 140-pound hammer free falling a distance of 30 inches. It is customary to drive the spoon 6 inches to seat the sampler into undisturbed soil, and then perform the test. The number of hammer blows for seating the spoon and making the tests are recorded for each 6 inches of penetration on the field drill log (e.g., 6/4/6). On the report log, the Standard Penetration Test result (i.e., the N value) is normally presented and consists of the sum of the 2nd and 3rd penetration counts (i.e., N=4+6=10 blows/ft).

Strata Changes - On the boring log, the horizontal lines represent strata changes. A solid line (——) represents an actually observed stratum change. A dashed line (----) represents an estimated stratum change.

Groundwater - Observations were made at the time indicated. Porosity of soil strata, weather conditions, site topography, etc., may cause changes in water-level readings indicated on the logs.

APPENDIX B

LABORATORY TEST DATA

TABLE B-1: SOIL INDEX-CLASSIFICATION TEST DATA (from test-pits)

Test-Pit No.	Sample Depth (ft)	PP (tsf)	Natural Moisture Content	Soil Description
TP-2	1.0 - 5.0	0.25	15	SILTY CLAY w/ fine sand traces, chert, limestone fragments, yellow-gray-brown
	6.0 - 7.0	1.5	33	CLAY w/ silt, fine sand traces, chert, limestone fragments, dark-brown, black (LL=52, PL=33, PI=19) MH
	8.0 - 9.5	0.75	16	SILTY CLAY w/ fine sand traces, chert, limestone fragments, yellow-brown (LL=36, PL=21, PI=15) CL
	9.5 - 11.0	0.5	20	SILTY CLAY w/ fine sand, limestone fragments, yellow-brown

Notes: 1. Natural Moisture Content per ASTM D2216. Atterberg Limits per ASTM D4318.

2. Atterberg Limits Designations:

LL = Liquid Limit (i.e., moisture content where soil becomes viscous fluid)

PL = Plastic Limit (i.e., moisture content where soil enters semi-solid stage)

PI = Plasticity Index (equals LL-PL and is representative of soil's plasticity)

3. PP = Pocket Penetrometer reading (indicates soil consistency). This is not allowable bearing value.

4. Unified Soil Classification System (USCS) designation follows the Atterberg Limits values.

Love Circle Windmill
Nashville, Tennessee
GPN: 11-6634-A

APPENDIX C

**ASFE PUBLICATION
EXAMPLE CHANGED-CONDITION CLAUSE
FIELD-OBSERVATION MEMORANDUM**

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 Geotechnical 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.



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EXAMPLE CLAUSE FOR UNANTICIPATED SUBSURFACE CONDITIONS

The owner has had a subsurface investigation performed by a foundation consultant, the results of which are contained in the consultant's report. The consultant's report presents his conclusions on the subsurface conditions based on his interpretation of the data obtained in the investigation. The contractor acknowledges that he has reviewed the consultant's report and any addenda thereto, and that his bid for earthwork operations is based on the subsurface conditions, as described in that report. It is recognized that a subsurface investigation may not disclose all conditions as they actually exist and further, conditions may change, particularly groundwater conditions, between the time of a subsurface investigation and the time of earthwork operations. In recognition of these facts, this clause is entered in the contract to provide a means of equitable additional compensation for the contractor if adverse unanticipated conditions are encountered and to provide a means of rebate to the owner if the conditions are more favorable than anticipated.

"If at any time during earthwork, paving, and foundation construction operations, the contractor encounters conditions that are different than those anticipated by the foundation consultant's report, he shall immediately (within 24 hours) bring this fact to the owner's attention. Once the fact of unanticipated conditions has been brought to the attention of either the owner or the contractor, and the owner and the contractor have concurred, immediate negotiations will be undertaken between the owner and the contractor to arrive at a change in contract price for additional work or reduction in work because of the unanticipated conditions. The contractor agrees that the following unit prices would apply for additional or reduced work under the contract. For changed conditions for which unit prices are not provided, the additional work shall be paid for on a time and materials basis."

Another example of a changed conditions clause can be found in Paper No. 4035 by Robert F. Borg published in ASCE Construction Division Journal, No. C02.

FIELD OBSERVATION A MESSAGE TO CLIENTS

Geotechnical engineering, geoenvironmental design, and other geoprofessional services are traditionally rendered through the Observational Method, the classic technique created by the father of geotechnics, Karl Terzaghi.

No matter how precise it may appear, the subsurface profile is fraught with uncertainty.

The Observational Method views a complete geoprofessional service as an indivisible two-phase process. In the first phase, the geoprofessional advises the client about project-specific risks and a subsurface exploration plan that responds to the client's risk tolerance levels, budget, schedule, and other vital concerns. The geoprofessional then supervises subsurface exploration and evaluates the samples and test data that result in order to prepare a subsurface profile that represents the geoprofessional's opinion of subsurface conditions. No matter how precise it may appear, the subsurface profile is fraught with uncertainty; it is based on a statistically tiny sample of the subsurface zone.

The subsurface profile typically is submitted as part of a report that also includes specific recommendations for construction or remediation. Even if they are not labeled as such, these recommendations are preliminary, because of the uncertainty associated with the subsurface profile on which they are based.

Even if they are not labeled as such, recommendations for construction or remediation are preliminary.

Field observation comprises the second phase of a complete geoprofessional service, permitting those who developed the report to observe excavation and thereby assess the reliability of their subsurface profile and the appropriateness of their preliminary recommendations. Actual conditions often differ from those expected, and that situation can create serious problems unless a qualified individual is available to decide what to do about them, where and when they are found. Decisions such as these are "judgment calls," and the quality of judgment can have a profound impact on the client's bottom line. The geoprofessionals of record are most qualified to make effective judgment calls because they are the individuals who are most familiar with the report and its preliminary recommendations, the exploration plan, the original findings, and the client's risk tolerance levels.

Some clients eliminate the observational method by separating the two geoprofessional service phases and retaining a second firm to perform field observation. A common reason given can be stated in one word: Money. By opening field observation services to all "qualified" bidders, the owner might find a firm that is willing to perform the service for less than the original firm. As experienced owners know, however, the true cost of a professional service goes far beyond fee. Paying less by relying on another firm is hardly consolation when that firm is not in a position to recognize or respond quickly to problem conditions that lead to delays, overruns, claims, or disputes, as demonstrated by a number of case histories available from ASFE.

Paying less by relying on another firm is hardly consolation when that firm is not in a position to recognize or respond quickly to problem conditions.

Geoprosessionals have their own risks to contend with, of course, and for that reason they inquire about field observation at the outset of a project. Ordinarily, geoprosessionals work closely with project owners and their consultants in recommending the most cost-effective approach consistent with the owner's objectives, knowing that the Observational Method permits close oversight and fast reaction should unanticipated conditions be encountered. When the Observational Method will not be used, however, geoprosessionals are advised to lower their risks by relying on a far more comprehensive scope of service than otherwise would be suggested, and/or by recommending construction or remediation measures with a higher safety factor. The result in some cases can be additional project costs that exceed the total fee paid for geoprosessional service; e.g., a \$1.2 million foundation or clean-up as opposed to one costing \$1 million.

Responsibility is another key concern when considering the observational phase of a geoprosessional service. Geoprosessionals are advised to disavow responsibility for problems that arise when others apply - as final - recommendations that clearly are subject to field verification. In essence, these firms say, "Why should we be held accountable for problems that arise because we were prohibited from completing our service and our work was misapplied by others?" Firms retained just to perform field observation are also advised to disavow responsibility for problems. After all, their role is not to complete the original firm's service, but rather to help determine that preliminary recommendations are followed as though they are final.

When the Observational Method is not used..., full responsibility might end up in the owner's lap.

In fact, a client can obtain a complete geoprosessional service *only* by having the original firm perform field observation after it completes subsurface exploration, or by having the second firm confer with the original firm and then accept the original firm's services (and liability for them) as its own. This latter approach can be a somewhat risky proposition for the second firm and actually would be considered illegal in states that regard it as a form of unacceptable plan stamping.

In short, when the Observational Method is not used and a complete geoprosessional service therefore is not performed, full responsibility might end up in the owner's lap.

The least overall cost and risk almost invariably result from professional services performed effectively. When geotechnical and geoenvironmental consultants are not allowed to perform a complete geoprosessional service, their ability to perform effectively is seriously eroded. Speak with your geoprosessional consultant for more information about this important issue or contact ASFE.

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