GEOTECHNICAL INVESTIGATION

FOR

SIX PROPOSED MAUSOLEUMS

LOUISVILLE MEMORIAL GARDENS EAST

LOUISVILLE, KENTUCKY

FOR

RENAISSANCE DESIGN BUILD, INC.

1012 SOUTH FOURTH STREET

LOUISVILLE, KENTUCKY 40203

BY

GREENBAUM ASSOCIATES, INC.

994 LONGFIELD AVENUE

LOUISVILLE, KENTUCKY 40215

JANUARY 22, 2013

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APPENDIX

1.0 Introduction

Renaissance Design Build, Inc. has been contracted to design a site for six mausoleums in Louisville Memorial Gardens East. The site is located to the west of a small reservoir on the cemetery, which is located at 11601 Ballardsville Road in Jefferson County, Kentucky, near the Oldham County line. The site is gently sloping and is covered by turf.

We were contracted by Renaissance Design Build, Inc. to carry out a geotechnical investigation directed at determining foundation support characteristics of the materials upon which these mausoleums will be supported. Work was coordinated through Mr. Nathan Grimes, PE, of Renaissance Design Build, Inc. who is acting as civil engineer on this project.

2.0 General Geology

The soils in this area are residuum, the residual product of weathering of the local bedrock. In this case the soil is lean and fat clay and is underlain by the Louisville Limestone. The Kentucky Geological Survey describes the Louisville Limestone as follows:

> Dolomitic limestone and dolomite, yellowish-gray to light-olive-gray, in quarry exposures interval more than 20 feet thick near top of lower half of unit has brownish cast; finely crystalline; argillaceous in zone about 15 to 20 feet above base; pyritic; thin to very thin bedded in upper part, thick bedded near base; bedding defined by stylolites; irregular rubbly bedding common; chert in discontinuous 0.2-foot-thick layer in uppermost few feet. Prominent bench-forming massive beds at about 35 feet and at 60 feet above base of unit, used in obtaining supplementary structural data. Fossils include brachiopods, among which *Pentimerus* is fairly common in a layer about 20 feet above base, algal stromatolites, and corals silicified remains of distinctive "chain" coral *Halysites* aids in distinguishing Louisville residual soils for those of overlying units. Contact with underlying unit abrupt to gradational through less than 1 foot. Sinks develop in unit on uplands.

3.0 Investigation

Six borings were carried out within the proposed mausoleum footprints, all by standard penetration procedures to refusal on bedrock. A track mounted Geoprobe equipped with an automatic hammer was used to carry out the borings. Borings were staked in advance of our arrival by Renaissance Design Build, Inc.

The standard penetration procedure involves driving a standard 2-inch diameter split spoon in the formation at selected intervals using a 140-pound hammer falling through 30 inches. The blow counts for each 6 inches of drive, to a total of 18 inches, are recorded and the number of blows for the 12 inches after the first 6 inches is a standard measure of the condition of the soil. As the split spoon is removed from the ground, it retrieves a sample of the soil in a disturbed condition. Nevertheless, this sample is suitable for certain classification tests and is representative of the soils at the depth tested.

Soil samples were returned to the laboratory where a program of testing was carried out. This testing included a grain size analysis, an Atterberg Limits test and natural moisture determinations.

Grain size determination arrives at a curve of grain size against that fraction of the soil that is finer than that particular grain size. It also allows the determination of the clay fraction, silt fraction, sand fraction, etc. in any particular soil sample. Based on this division of grain sizes, the field soils classifications are refined and the boring logs adjusted. In the case of fine grained soils, the soils are largely silt and clay; thus requiring that the soils be suspended in an aqueous medium and the rate at which the particles drop out is measured in order to arrive at the grain size distribution. Silt and clay grains are so fine that sieve analysis alone will not function in this range. The coarse fraction of this sample is separated from the fine and run through a nest of sieves in order to further detail the grain size distribution in the coarse range.

The Atterberg Limits determination arrives at those moisture contents at which the soil turns from a solid state to a plastic condition (the Plastic Limit) and then from a plastic condition to a liquid condition (The Liquid Limit). The points in question are arrived at by standard procedures that accept specific cohesive and flow properties of the soil as standards for these limits. Knowing the moisture content of the soil in relation to these limits provides a broad measure of the soil strength and soil characteristics. The arithmetic difference between these two limits is called the Plasticity Index and all three together are used for classifying the soils in a number of standard systems.

Natural moisture determinations were run on all of the soil samples recovered. This test arrives at the in-situ moisture content of the soil and is useful for correlating the strength of various samples of like texture and in conjunction with the Atterberg limits, gives a strong measure of the strength range the soils are likely to be found in.

4.0 Findings

4.1 Boring Results

This site is covered by a mantle of 6 to 8 inches of topsoil. Below this, soil is moist, brown lean clay to a depth of 3 to 5 feet, more commonly 3 to 4 feet. Below this is most, yellowish brown or red fat clay, frequently containing ferromagnesian nodules. Limestone bedrock was encountered between 7.3 and 12.2 feet depth, as can be seen in the table in the following paragraph. It can also be seen that soils become progressively stiffer with depth. The lowest N-values are in the top three feet where N-values are in the range of 7 to 14. At the 3.5 to 5 foot level N-values are in the range of 10 to 27 and are yet stiffer at greater depth.

N-values, when corrected for the energy of the automatic hammer, are as shown in the following table.

	B-1	B-2	B-3	B-4	B-5	B-6
1 – 2.5 feet	8	7	14	10	13	13
3.5 – 5 feet	10	17	22	18	25	27
6 – 7.5 feet	17	35	39	38	46	50/3"
8.5 – 10 ft.	42	38	50/3"	48	55	
Refusal	12.0'	10.5'	9.8'	11.0'	12.2'	7.3

Although no groundwater was encountered in any of the borings upon completion of drilling, water may be perched above the bedrock surface seasonally.

4.2 Laboratory Results

One sample of soil was tested and classified. The sample taken from boring B-4 from 3.5 feet to 5 feet depth was found to be sandy lean clay containing 36 percent sand, 24 percent silt and 40 percent clay. An Atterberg limits test on this sample indicated a liquid limit of 48, a plastic limit of 20 and a plasticity index of 28. This soil is classified as CL by the Unified system and as A-7-6 by the AASHTO system.

4.3 Seismicity

By the 2007 edition of the Kentucky Building Code, this is a very dense soil and soft rock profile, site class C. The Spectral Response Acceleration Coefficients, for this area, as provided by U.S.G.S., FEMA Design Parameters are $S_S = 0.232$ and $S_1 = 0.113$.

5.0 Recommendations

5.1 Foundations

The proposed mausoleums may be supported on spread footings bearing on shallow soil or structural fill placed in accordance with section 5.3 of this report. These foundations may be designed based on an allowable net bearing capacity of 2,500 pounds per square foot. These foundations, however, must not bear directly on fat clay.

Once foundation bearing surfaces are exposed, an engineer or senior engineering technician from this office should be present to view all bearing surfaces. If soft areas are encountered, undercut will need to extend to firm material or to a level determined to be acceptable by the geotechnical engineer and should be refilled with lean clay fill or lean concrete. If clay at the foundation bearing level is found to be fat, it should be excavated to a foot below the bearing level and should be replaced with lean clay compacted to at least 98 percent of the soils maximum dry density as determined by the Standard Proctor (ASTM D698).

Soil bearing foundations must bear at least 30 inches below finished grade in order to insulate the bearing strata from freezing. Continuous footings must be at least 16 inches wide and isolated footings must be at least 24 inches wide.

Settlement of foundations designed based on the above criteria should be well below that which is considered acceptable for this type of construction; that is total settlement should be less than one inch and differential settlement should be less than three quarters of an inch.

For shallow foundations, friction along the base of the footing can be used to resist lateral forces. A friction coefficient of 0.3 may be used, which assumes that the footing concrete is placed directly against the natural cut faces. The coefficient of friction value recommended is an ultimate value and a minimum factor of safety of 1.5 must be applied when determining the allowable sliding resistance.

5.2 Floor Slabs

Prior to placement of the fill in the slab area, the subgrade must be proofrolled and carefully examined by a geotechnical engineer for areas of soft soil. If soft soils are encountered, they must be undercut and refilled in accordance with instructions given by the geotechnical engineer's on-site representative. If fat clay is present in the slab subgrade, it must be excavated to at least one foot below subgrade elevation and should be replaced with lean clay compacted to at least 98 percent of the soils maximum dry density as determined by the Standard Proctor (ASTM D698).

Once necessary corrections are made, a conventionally designed slab-on-grade should perform satisfactorily. The floor slab should be structurally separated from the walls, columns and foundations and liberally jointed to minimize the stress caused by possible differential settlement between the slabs and the foundations and between adjacent slabs. A vapor barrier must be incorporated into the design and at least four inches of Dense Graded Aggregate (DGA) should underlie the slab. The floor slab may be designed based on a Modulus of Subgrade Reaction of 125 pounds per cubic inch.

5.3 Site Preparation and Earthwork

All fill should be placed in lifts not exceeding 8 inches in uncompacted thickness and must be compacted to at least 98 percent of the soils maximum dry density as determined by the Standard Proctor (ASTM D-698). Soil moisture content should be within 2 percent of optimum as determined from the Standard Proctor.

Soil from any off-site borrow sources should be tested and approved by this office prior to being used on the site. Satisfactory borrow materials are those falling in one of the following classifications: GC, SM, SC, ML, or CL. Soil types MH, CH and OH soils and peat are unsatisfactory borrow materials.

The site should be maintained in a well-drained condition both during and after construction. Site grading should provide for drainage of surface run-off from the building and pavement.

The placement of compacted fill should be carried out by an experienced excavator with the proper materials. The excavator must be prepared to adapt his procedures, equipment and materials to the type of project, to weather conditions, and the structural requirements of the engineer. Methods and materials used in summer may not be applicable in winter; soil used in proposed fill may require wetting or drying for proper placement and compaction. Conditions may also vary during the course of a project or in different areas of this site. These needs should be addressed in the project drawings and specifications.

During freezing conditions, the fill must **not** be frozen when delivered to the site. It also must not be allowed to freeze during or after compaction. Since the ability to work the soil while keeping it from freezing depends in part on the soil type, the specifications should require the contractor to submit a sample of his proposed fill before construction starts, for laboratory testing. If the soil engineer determines that it is not suitable, it should be rejected. In general, silty sand, clayey sand, and cohesive/semi-cohesive soils should not be used as fill under freezing conditions. All frozen soil of any type should be rejected for use as compacted fill.

It is important that compacted fill be protected from freezing after it is placed. The excavator should be required to submit a plan for protecting the soil. The plan should include details on the type and amount of material (straw, blankets, extra loose fill, topsoil, etc.) proposed for use as frost protection. The need to protect the soil from freezing is ongoing throughout construction and applies both before **and** after concrete is placed, until backfilling for final frost protection is completed. Foundations placed on frozen soil can experience heaving and significant settlement, rotation, or other movement as the soil thaws. Such movement can also occur if the soil is allowed to freeze **after** the concrete is placed and then allowed to thaw. The higher the percentage of fines (clay and silt, P-200 material) in the fill, the more critical is the need for protection from freezing.

The contractor should be required to adjust the moisture content of the soil to within a narrow range near the optimum moisture content (as defined by the applicable Proctor or AASHTO Test). In general, fill should be placed within 2% of optimum moisture. The need for moisture control is more critical as the percentage of fines increases. Naturally occurring cohesive/semi-cohesive soil are often much wetter than the optimum. Placing and attempting to compact such soils to the specified density may be difficult. Even if compacted to the specified density, excessively wet soils may not be suitable as pavement subgrades due to pumping under applied load. This is especially true when wet cohesive/semi-cohesive soil is used as backfill in utility trenches and like situations. Excessively wet soil in thick fill sections may cause post-construction settlement beyond that estimated for fill placed at or near (±2%) the optimum moisture content.

5.4 Earth Pressures and Retaining Walls

Any retaining walls should be constructed with a drainage blanket of sand or a synthetic drainage material. Synthetic drainage media should be available from suppliers of geotextile. The wall should be drained at its base by a perforated PVC underdrain or weepholes at a spacing of not more than 10 feet. Where a relatively thin drainage blanket is used, the retaining wall should be designed based on a coefficient of active earth pressure (K_a) of 0.36 and a soil unit weight (γ_w) of 130 pounds per cubic foot. This results in an equivalent fluid pressure of 47 pounds per cubic foot. Where granular backfill completely fills the area defined by a plane extending upward from the base of the wall at a 45 degree angle, the retaining wall may be designed based on a coefficient of active earth pressure (K_a) of 0.27 and a soil unit weight (γ_w) of 130 pounds per cubic foot. This results in an equivalent fluid pressure of 35 pounds per cubic foot.

However, where the wall is restrained from movement, as in the case of building basement walls bearing against the basement slab or building frame, the wall must be designed based on the "at rest" earth pressure. The coefficient of "at rest" earth pressure (K_0) is 0.47 with a soil unit weight (γ_w) of 130 pounds per cubic foot in the case of a thin drainage blanket behind the wall, resulting in an equivalent fluid of 61 pounds per cubic foot unit weight. Where granular backfill completely fills the area defined by a plane extending upward from the base of the wall at a 45 degree angle, the retaining wall may be designed based on a coefficient of "at rest" earth pressure (K_0) of 0.43 and a soil unit weight (γ_w) of 130 pounds per cubic foot. This results in an equivalent fluid pressure of 56 pounds per cubic foot.

Surcharge above the wall will add additional load. A uniform surcharge must be multiplied by the appropriate coefficient of earth pressure to determine the additional load applied to the wall.

Any retaining wall design must use appropriate factors of safety. It is critical that drainage be provided as mentioned earlier in this section in order to avoid hydrostatic pressure. Hydrostatic pressure would increase pressure against the wall substantially.

5.5 Pavement

Pavement subgrade should be examined and proofrolled as described under "Floor Slabs". If soft areas are encountered, the soft soils will need to be undercut and refilled in accordance with the instructions of the geotechnical engineer's on-site representative. If fat clay is encountered in the pavement subgrade, the top foot should be replaced with lean clay as described in section 5.2.

A pavement analysis was conducted using a life cycle of 20 years and a cumulative 18-kip equivalent single axle load of 20,000 for light traffic loads and 80,000 for moderate traffic loads. Recommendations are provided for both flexible and rigid pavement systems. However, rigid pavement should be used in special truck traffic areas, such as dumpster pads. Any dumpster pad should be large enough so that all the wheels of the collection truck are supported on the concrete pavement during lifting of the waste bin. The concrete pavement should extend throughout the areas that require extensive turning and maneuvering of the delivery vehicles and garbage trucks. Dumpster pads, loading areas and other heavily loaded pavement areas that are not designed to accommodate these conditions often experience localized pavement failures, particularly if flexible pavement sections are used.

The minimum recommended thickness for both hot mixed asphalt concrete (HMAC) and reinforced Portland cement concrete (PCC) pavement sections are presented in the following table for the described light, moderate and special traffic condition.

Recommended Pavement Section					
Component	Light		Moderate		Special
	Rigid	Flexible	Rigid	Flexible	Rigid
Reinforced Portland Cement Concrete (PCC)	5 inches		6 inches		7 inches
Hot Mixed Asphalt Concrete (HMAC)		3 inches		4 inches	
Crushed Limestone Base (DGA)	4 inches	8 inches	4 inches	8 inches	4 inches
Prepared Subgrade	6 inches	6 inches	6 inches	6 inches	6 inches

The Portland cement concrete should be air-entrained and conform to ASTM C-94 (Standard Specifications for Ready-Mixed Concrete) and have a minimum compressive strength of 4,000 pounds per square foot. Reinforcing should meet the requirements of ACI.

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Hot mix asphalt concrete and Dense Graded Aggregate should meet the requirements of the Kentucky Transportation Cabinet. The top inch of asphalt should be a surface mix, the remainder being a base mix.

Prepared subgrade should be compacted to 98 percent of the soils maximum dry density as measured by the Standard Proctor.

5.6 Temporary Earth Slopes or Cuts

Temporary earth cuts necessary to construct foundations or utility lines should be no deeper than 4 feet without benching or sloping. Cuts deeper than this should be sloped no steeper than one horizontal to one vertical or should have benches every 2 feet of height equating to this slope. If vertical faces deeper than 4 feet are used, bracing designed for short term loads may be used. Excavations should comply with OSHA regulations. If soft soils are encountered, Greenbaum Associates, Inc. should view the cut face prior to personnel entering the excavation.

5.7 Limitations

We strongly recommend that bearing surfaces and compaction be monitored by Greenbaum Associates, Inc. Our technicians will be available to further assist you in providing these and other normally specified quality control services. The report is preliminary until such time as these examinations are completed to confirm conditions consistent with those discovered in the investigation.

The conclusions and recommendations offered in this report are based on the subsurface conditions encountered in the borings. No warranties can be made regarding the continuity of conditions between or beyond borings. If, during construction, soil conditions are encountered that differ from those indicated in this report, a representative of Greenbaum Associates, Inc. should inspect the site to determining if design modification is required.

This study was directed at specific mausoleums and associated pavement at this location to be constructed within a reasonably short period after this study. Use for any other location, structures or substantial changes in construction period may invalidate the recommendations. The geotechnical engineer should be consulted relative to any substantial change in these.

This study is directed at mechanical properties of the soils and includes no sampling, testing or evaluation for environmental considerations.

Important Information about Your

Geotechnical Engineering Report

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

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

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 your ASFE-member geotechnical engineer for more information.

ASFE THE GEOPROFESSIONAL BUSINESS ASSOCIATION

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Renaissance Design Build, Inc.	Greenbaum Associates, Inc.	**Site Location Plan** Six Mausoleums Louisville Memorial Gardens East Greenbaum Project Number: 12-199

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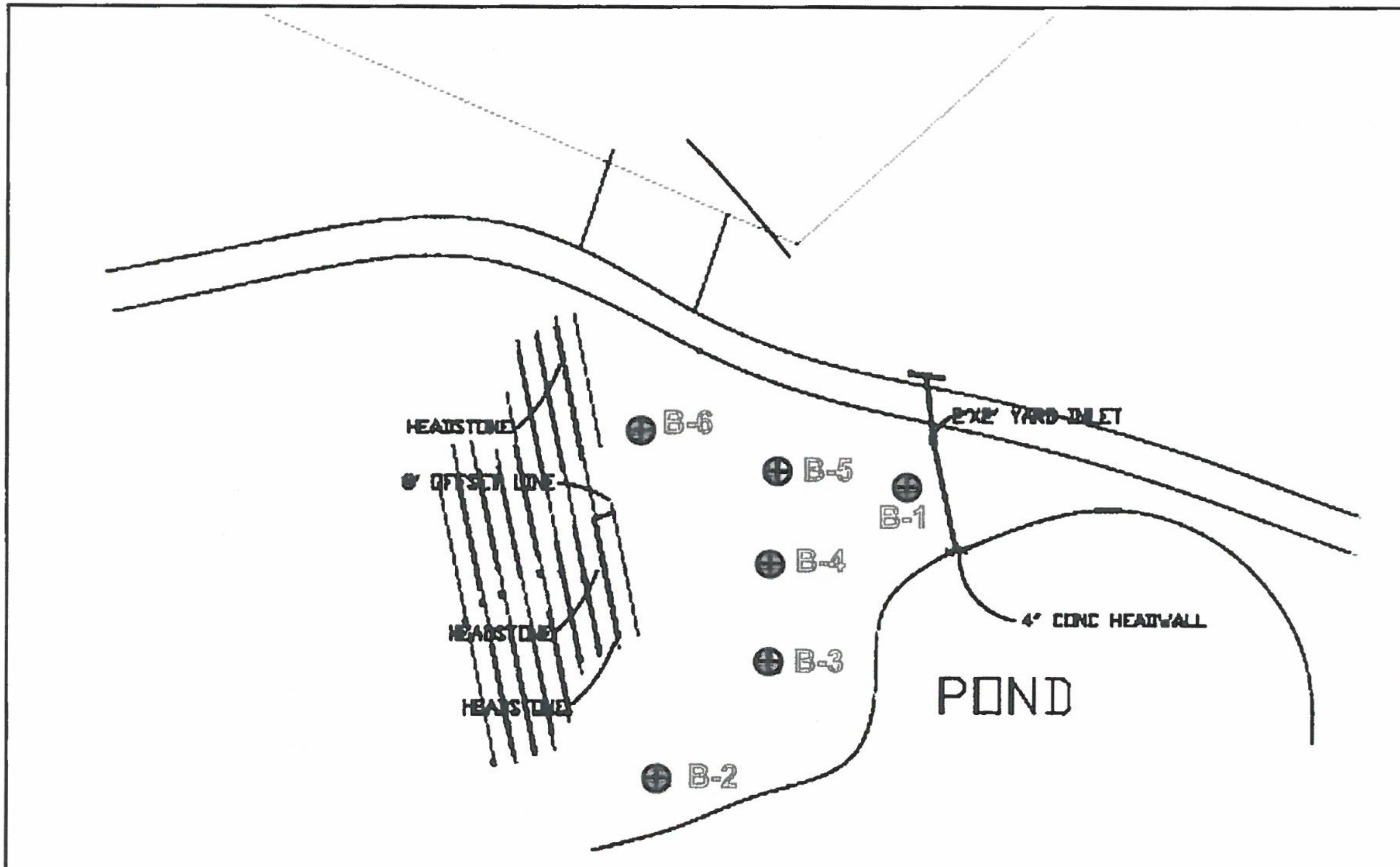

Renaissance Design Build, Inc.	Greenbaum Associates, Inc.	**Boring Location Plan** Six Mausoleums Louisville Memorial Gardens East Louisville, Kentucky Greenbaum Project Number: 12-199

Recieved May 18, 2023 Planning and Design 23-DDP-0040

SOIL DESCRIPTION TERMINOLOGY

Soils are identified and classified in this report according to the Unified Soil Classification System with the following modifiers:

RELATIVE DENSITY OF GRANULAR SOILS

Description	Blows/Foot
Very loose	0 to 4
Loose	4 to 10
Medium Dense	10 to 30
Dense	30 to 50
Very Dense	50 to 80
Extremely Dense	80 +

CONSISTENCY OF COHESIVE SOILS

Description	N	q. (tsf)
Very soft	2	0 to 0.25
Soft	2-4	0.25 to 0.50
Medium	4-8	0.50 to 1.0
Stiff	8-15	1.0 to 2.0
Very Stiff	15-30	2.0 to 4.0
Hard	> 30	4.0 to 8.0
Very Hard		8.0 +

PARTICLE SIZES

Components		Size or Sieve No.
Boulders		over 12 inches
Cobbles		3 to 12 inches
Gravel	Coarse	3/4 to 3 inches
	Fine	No. 4 to 3/4 inches
Sand	Coarse	No. 10 to No. 4
	Medium	No. 40 to No. 10
	Fine	No. 200 to No. 40
Fines (silt and clay)		Below No. 200

SOIL MOISTURE

Descriptive Term	
Dry	- Dry of Standard Proctor Optimum
Damp	- Moist, sand only
Moist	- Near Standard Proctor Optimum
Wet	- Wet of Standard Proctor Optimum
Saturated	- Free water in sample

ROCK QUALITY DETERMINATION

The Rock Quality Determination (Deere et. al., 1969) method of determining rock quality as reported here was obtained by summing up the total length of core recovered in each run, counting only those pieces of core which are four inches (10 cm) in length or longer and which are hard and sound. The sum is then represented as a percentage over the length of the run. If the core is broken by handing or by the drilling process, the fresh broken pieces are fitted together and counted as one piece provided that they form the requisite length of four inches (10 cm). RQD is reported as a percentage.

RELATIVE OF RQD AND ROCK QUALITY

RQD (%)	Description of Rock Quality
0 to 25	Very Poor
25 to 50	Poor
50 to 75	Fair
75 to 90	Good
90 to 100	Excellent

NOTE: Recovery as denoted as REC = , is the length of core recovered in a run divided by the length of the run, reported as a percentage.

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HOLE No. B-1

Sheet 1 of 1

Client: Renaissance Design Build, Inc.
Project: Six Mausoleums Louisville Memorial Gardens East
Project No.: 12-199

Boring Location: **See Boring Location Plan** Surface Elevation: **Ground** Station: **n/a**
Drilling Equipment: **Geoprobe equipped with Automatic Hammer** Drilling Method: **Direct Push**
Depth to water immediately: **Dry** Overburden: **12** Rock: **0** Total Depth: **12.0**
Logged By: **Geo Logic, Inc.** Driller: **Geo Logic, Inc.** Date Logged: **1/14/13 - 1/14/13**

DEPTH (feet)	SAMPLE NO.	RECOVERY %	RQD %	MATERIAL DESCRIPTION		ELEVATION (feet)	N VALUE
0				Topsoil (6 inches)	OL	Ground	
				Moist, Medium Stiff, Brown Lean Clay	CL		
	SPT	56					6
				Moist, Stiff, Brown, Yellow and Red Mottled, Fat Clay	CH		
	SPT	44					8
5				Same, Very Stiff, with Ferromagnesian Nodules	CH		
	SPT	68					13
				Same, Hard	CH		
	SPT	78					32
10							
12				AUGER REFUSAL @ 12.0 FEET			

STANDARD PENETRATION TEST
● (blows/ft)
PL ⊢ MC ▲ ⊣ LL
10 20 30 40 50 60 70 80 90

SAMPLER TYPE
SS - Split Spoon
ST - Shelby Tube
HQ - Rock Core, 2-1/2"
NX - Rock Core, 2-1/8"
CU - Cuttings
CT - Continuous Tube

DRILLING METHOD
HSA - Hollow Stem Auger
CFA - Continuous Flight Augers
DC - Driving Casing
RW - Rotary Wash
RC - Rock Core

Hole No. **B-1**

LOG WITH WELL AND SPT GRAPH 12-199.GPJ 08-053.GPJ 1/21/13

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HOLE No. B-2

Sheet 1 of 1

Client: Renaissance Design Build, Inc.
Project: Six Mausoleums Louisville Memorial Gardens East
Project No.: 12-199

Boring Location: **See Boring Location Plan** | Surface Elevation: **Ground** | Station: **n/a**

Drilling Equipment: **Geoprobe equipped with Automatic Hammer** | Drilling Method: **Direct Push**

Depth to water immediately: **Dry** | Overburden: **10.5** | Rock: **0** | Total Depth: **10.5**

Logged By: **Geo Logic, Inc.** | Driller: **Geo Logic, Inc.** | Date Logged: **1/14/13 - 1/14/13**

DEPTH (feet)	GRAPHIC LOG	SAMPLE NO.	RECOVERY %	RQD %	MATERIAL DESCRIPTION		ELEVATION (feet)	STANDARD PENETRATION TEST ● (blows/ft) PL ├ MC ▲ ┤ LL 10 20 30 40 50 60 70 80 90	N VALUE
					Topsoil (8 inches)	OL	Ground		
					Moist, Medium Stiff, Brown Lean Clay	CL			
		SPT	68						5
					Same, Stiff, Yellowish Brown	CL			
		SPT	100						13
5					Moist, Hard, Red Fat Clay	CH			
		SPT	83						27
					Same, Yellow and Red Mottled, with Chert	CH			
		SPT	72						29
10									
					AUGER REFUSAL @ 10.5 FEET				

SAMPLER TYPE		DRILLING METHOD		Hole No.
SS - Split Spoon	NX - Rock Core, 2-1/8"	HSA - Hollow Stem Auger	RW - Rotary Wash	
ST - Shelby Tube	CU - Cuttings	CFA - Continuous Flight Augers	RC - Rock Core	
HQ - Rock Core, 2-1/2"	CT - Continuous Tube	DC - Driving Casing		B-2

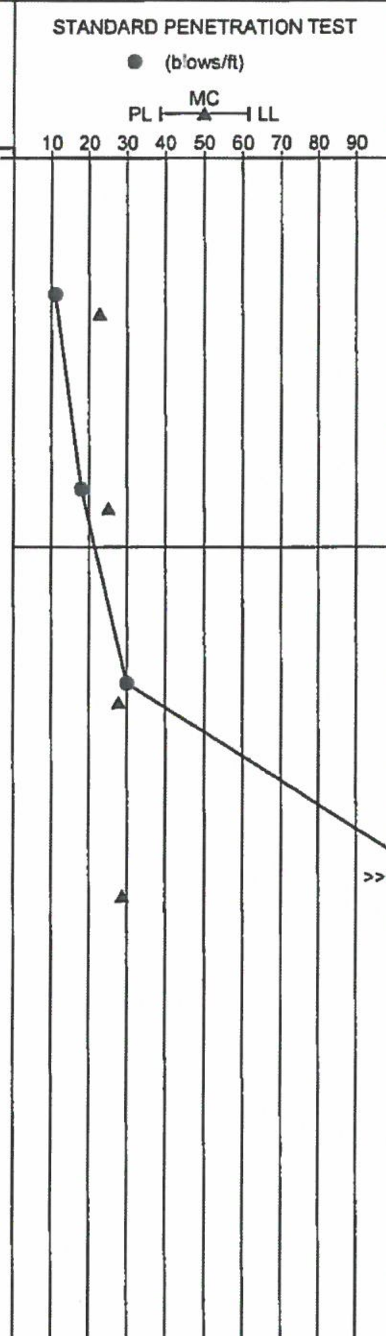
Greenbaum Associates, Inc.
Louisville, KY 40215 (502) 361-8447

HOLE No. B-3

Sheet 1 of 1

Client:	Renaissance Design Build, Inc.
Project:	Six Mausoleums Louisville Memorial Gardens East
Project No.:	12-199
Boring Location:	See Boring Location Plan
Surface Elevation:	Ground
Station:	n/a
Drilling Equipment:	Geoprobe equipped with Automatic Hammer
Drilling Method:	Direct Push
Depth to water Immediately:	Dry
Overburden:	9.8
Rock:	0
Total Depth:	9.8
Logged By:	Geo Logic, Inc.
Driller:	Geo Logic, Inc.
Date Logged:	1/14/13 - 1/14/13

DEPTH (feet)	SAMPLE NO.	RECOVERY %	RQD %	MATERIAL DESCRIPTION		ELEVATION (feet)	N VALUE
				Topsoil (6 inches)	OL	Ground	
				Moist, Stiff, Dark Brown Lean Clay	CL		
	SPT	68					11
				Moist, Very Stiff, Yellowish Brown and Brown Mottled Fat Clay with Ferromagnesian Nodules	CH		
	SPT	94					18
5				Moist, Hard, Yellowish Brown and Gray Mottled Fat Clay	CH		
	SPT	89					30
				Same, with Ferromagnesian Nodules	CH		
	SPT	93					50/ 3"
				AUGER REFUSAL @ 9.8 FEET			



SAMPLER TYPE		DRILLING METHOD		Hole No.
SS - Split Spoon	NX - Rock Core, 2-1/8"	HSA - Hollow Stem Auger	RW - Rotary Wash	B-3
ST - Shelby Tube	CU - Cuttings	CFA - Continuous Flight Augers	RC - Rock Core	
HQ - Rock Core, 2-1/2"	CT - Continuous Tube	DC - Driving Casing		

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Client: Renaissance Design Build, Inc. Project: Six Mausoleums Louisville Memorial Gardens East Project No.: 12-199	**HOLE No. B-4** Sheet 1 of 1

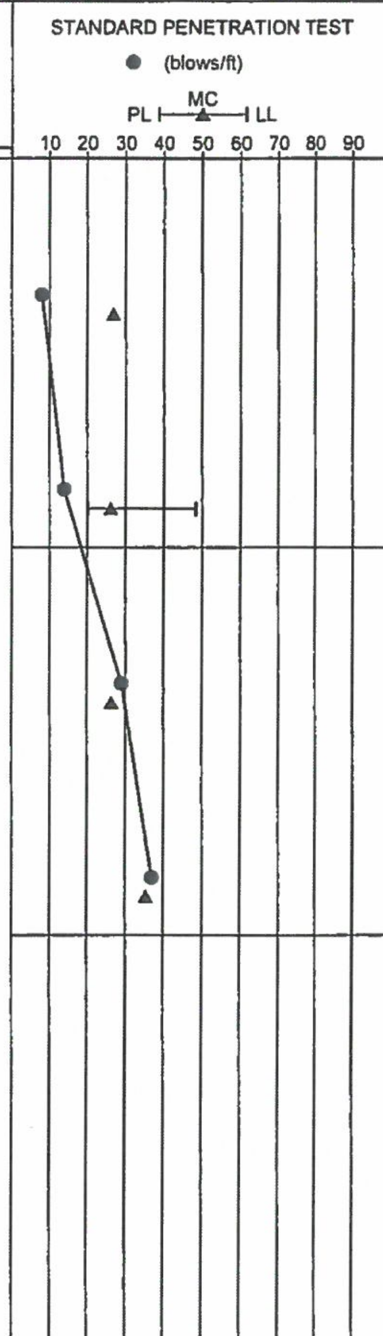
Boring Location: **See Boring Location Plan** Surface Elevation: **Ground** Station: **n/a**

Drilling Equipment: **Geoprobe equipped with Automatic Hammer** Drilling Method: **Direct Push**

Depth to water immediately: **Dry** Overburden: **11** Rock: **0** Total Depth: **11.0**

Logged By: **Geo Logic, Inc.** Driller: **Geo Logic, Inc.** Date Logged: **1/14/13 - 1/14/13**

DEPTH (feet)	GRAPHIC LOG	SAMPLE NO.	RECOVERY %	RQD %	MATERIAL DESCRIPTION		ELEVATION (feet)	N VALUE
					Topsoil (8 inches)	OL	Ground	
					Moist, Stiff, Dark Brown Lean Clay	CL		
		SPT	72					8
					Same, Very Stiff	CL		
		SPT	89					14
5					Moist, Hard, Brown Fat Clay with Ferromagnesian Nodules	CH		
		SPT	83					29
					Same, Reddish Brown	CH		
		SPT	100					37
10								
					AUGER REFUSAL @ 11.0 FEET			



SAMPLER TYPE		DRILLING METHOD		Hole No.
SS.- Split Spoon	NX - Rock Core, 2-1/8"	HSA - Hollow Stem Auger	RW - Rotary Wash	
ST - Shelby Tube	CU - Cuttings	CFA - Continuous Flight Augers	RC - Rock Core	
HQ - Rock Core, 2-1/2"	CT - Continuous Tube	DC - Driving Casing		B-4

LOG WITH WELL AND SPT GRAPH 12-199.GPJ 08-053.GPJ 1/21/13

Recieved May 18, 2023 Planning and Design 23-DDP-0040

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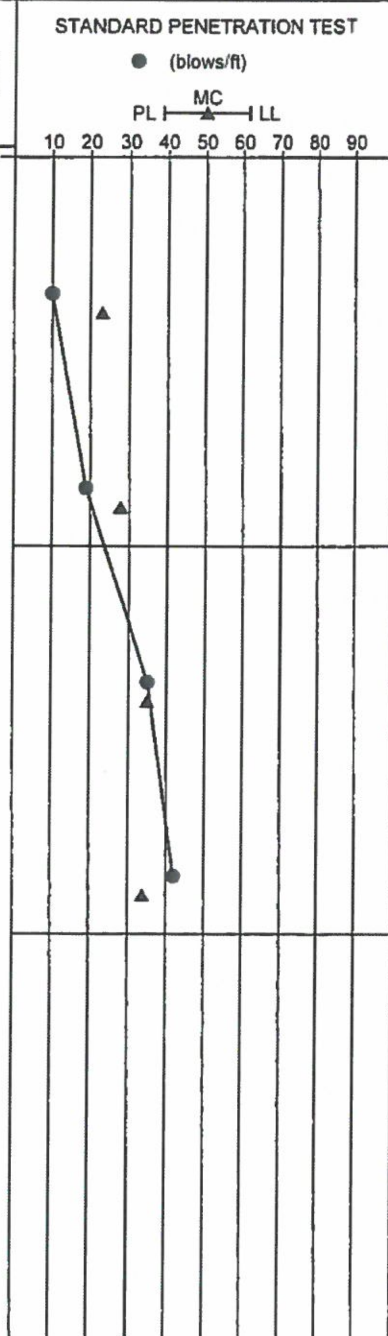
HOLE No. B-5

Sheet 1 of 1

Client: Renaissance Design Build, Inc.
Project: Six Mausoleums Louisville Memorial Gardens East
Project No.: 12-199

Boring Location: **See Boring Location Plan** Surface Elevation: **Ground** Station: **n/a**
Drilling Equipment: **Geoprobe equipped with Automatic Hammer** Drilling Method: **Direct Push**
Depth to water immediately: **Dry** Overburden: **12.2** Rock: **0** Total Depth: **12.2**
Logged By: **Geo Logic, Inc.** Driller: **Geo Logic, Inc.** Date Logged: **1/14/13 - 1/14/13**

DEPTH (feet)	SAMPLE NO.	RECOVERY %	RQD %	MATERIAL DESCRIPTION		ELEVATION (feet)	N VALUE
0				Topsoil (6 inches)	OL	Ground	
				Moist, Stiff, Brown Lean Clay	CL		
	SPT	83					10
				Moist, Very Stiff, Tan and Gray Mottled Fat Clay	CH		
	SPT	56					19
5				Same, Hard, with Ferromagnesian Nodules	CH		
	SPT	94					35
	SPT	78					42
10							
				AUGER REFUSAL @ 12.2 FEET			



SAMPLER TYPE		DRILLING METHOD		Hole No.
SS - Split Spoon	NX - Rock Core, 2-1/8"	HSA - Hollow Stem Auger	RW - Rotary Wash	
ST - Shelby Tube	CU - Cuttings	CFA - Continuous Flight Augers	RC - Rock Core	
HQ - Rock Core, 2-1/2"	CT - Continuous Tube	DC - Driving Casing		B-5

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Client: Renaissance Design Build, Inc.	**HOLE No. B-6**
Project: Six Mausoleums Louisville Memorial Gardens East	
Project No.: 12-199	Sheet 1 of 1

Boring Location: **See Boring Location Plan** Surface Elevation: **Ground** Station: **n/a**

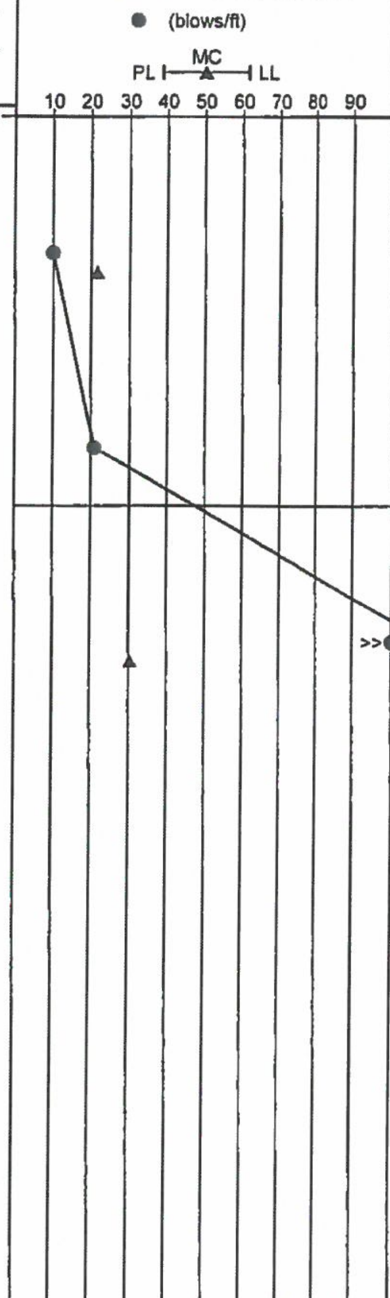
Drilling Equipment: **Geoprobe equipped with Automatic Hammer** Drilling Method: **Direct Push**

Depth to water immediately: **Dry** Overburden: **7.3** Rock: **0** Total Depth: **7.3**

Logged By: **Geo Logic, Inc.** Driller: **Geo Logic, Inc.** Date Logged: **1/14/13 - 1/14/13**

DEPTH (feet)	GRAPHIC LOG	SAMPLE NO.	RECOVERY %	RQD %	MATERIAL DESCRIPTION		ELEVATION (feet)	N VALUE
					Topsoil (6 inches)	OL	Ground	
					Moist, Stiff, Red Lean Clay	CL		
		SPT	100					10
					Moist, Very Stiff, Brown and Tan Mottled Fat Clay	CH		
		SPT	78					21
5								
					Same, Hard	CH		
		SPT	100					50/ 3"
					AUGER REFUSAL @ 7.3 FEET			

STANDARD PENETRATION TEST



SAMPLER TYPE		DRILLING METHOD		Hole No.
SS - Split Spoon	NX - Rock Core, 2-1/8"	HSA - Hollow Stem Auger	RW - Rotary Wash	
ST - Shelby Tube	CU - Cuttings	CFA - Continuous Flight Augers	RC - Rock Core	
HQ - Rock Core, 2-1/2"	CT - Continuous Tube	DC - Driving Casing		B-6

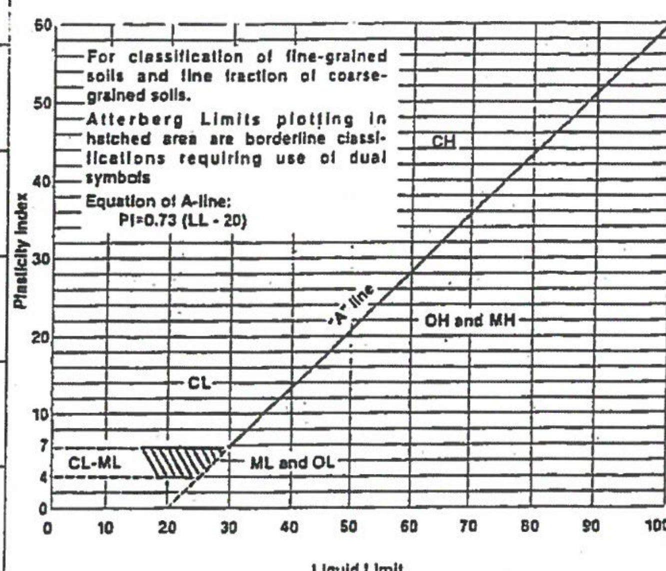
LOG WITH WELL AND SPT GRAPH 12-199.GPJ 08-053.GPJ 1/21/13

Recieved May 18, 2023 Planning and Design 23-DDP-0040

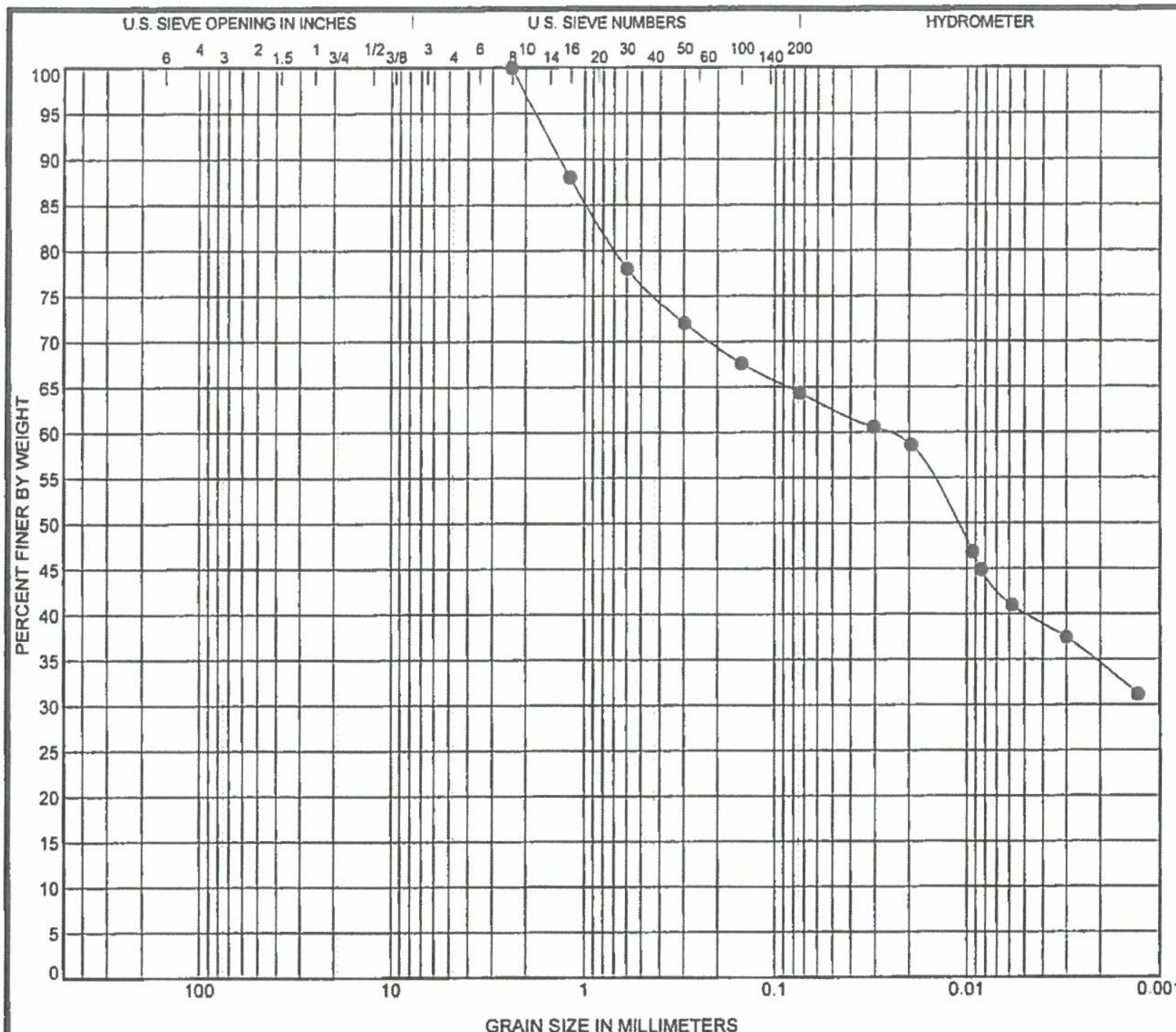
CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES

(ASTM: D 2487 and 2488)

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 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 per cent ..... GW, GP, SW, SP; More than 12 per cent ..... GM, GC, SM, SC; 5 to 12 per cent ..... *Borderline* cases requiring dual symbols	$C_u = \frac{D_{60}}{D_{10}}$ greater than 4; $C_c = \frac{(D_{30})^2}{D_{10} X D_{60}}$ between 1 and 3	
			GP		Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for GW	
		Gravels with fines (Appreciable amount of fines)	GM	d / u	Silty gravels, gravel-sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 *are border-line* cases requiring use of dual symbols
			GC		Clayey gravels, gravel-sand-clay mixtures		Atterberg limits below "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; $C_c = \frac{(D_{30})^2}{D_{10} X D_{60}}$ between 1 and 3	
			SP		Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for SW	
		Sands with fines (Appreciable amount of fines)	SM	d / u	Silty sands, sand-silt mixtures		Atterberg limits below "A" line or P.I. less than 4	Limits plotting in hatched zone with P.I. between 4 and 7 *are borderline* cases requiring use of dual symbols.
			SC		Clayey sands, sand-clay mixtures		Atterberg limits below "A" line or P.I. greater than 7	
Fine-grained soils More than half of material is *smaller* than No. 200 sieve)	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	See Plasticity Chart		
			CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean 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			
Highl organic soils			Pt		Peat and other highly organic soil			



Plasticity Chart



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Specimen Identification		Classification	LL	PL	PI	Cc	Cu
●	B-4	4.5	SANDY LEAN CLAY(CL)	48	20	28		

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-4	4.5	2.38	0.027			0.0	35.6	24.2	40.2

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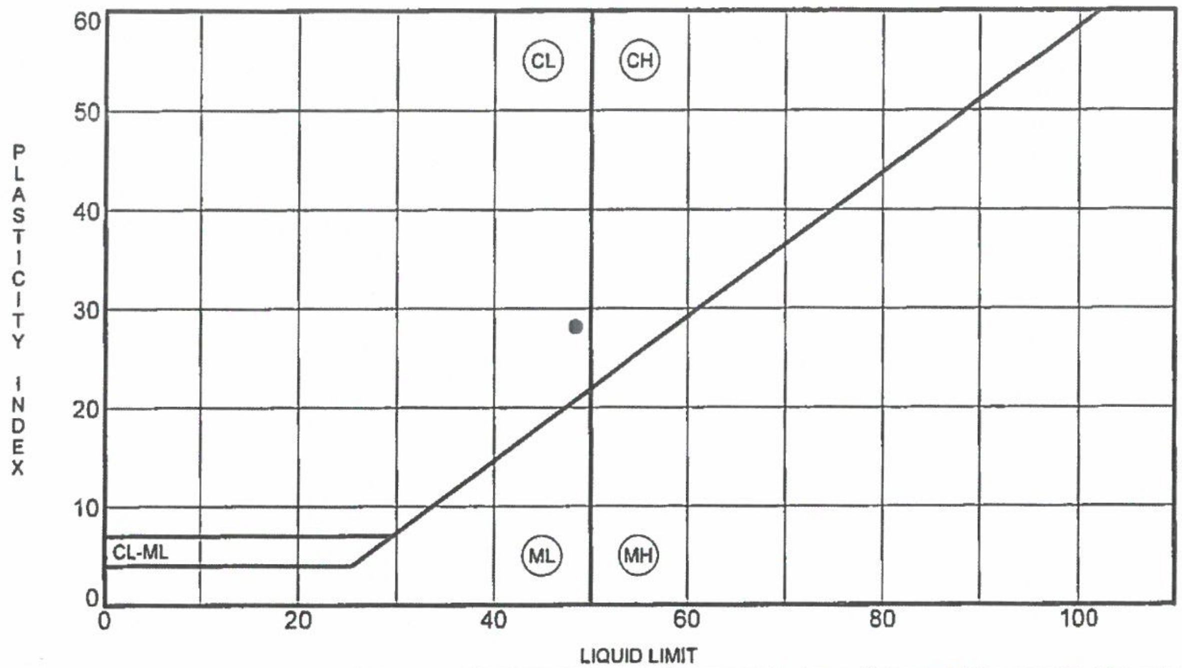
GRAIN SIZE DISTRIBUTION

Project: Six Mausoleums

Location: Louisville Memorial Gardens East

Number: 12-199



	Specimen Identification		LL	PL	PI	Fines	Classification
●	B-4	4.5	48	20	28	64	SANDY LEAN CLAY(CL)

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ATTERBERG LIMITS' RESULTS

Project: Six Mausoleums

Location: Louisville Memorial Gardens East

Number: 12-199