

REPORT OF  
GEOTECHNICAL EXPLORATION  
FOR  
CAPITAL PLAZA PARKING GARAGE  
AND OFFICE BUILDING  
FRANKFORT, KENTUCKY  
PROJECT NO. 1831-11-430  
March 2, 2012

Prepared For

Commonwealth of Kentucky Finance and Administration Cabinet  
Department for Facilities and Support Services  
Division of Engineering and Contract Administration  
403 Wapping Street, 1<sup>st</sup> Floor  
Frankfort, Kentucky 40601

Prepared by  
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March 2, 2012

Commonwealth of Kentucky Finance and Administration Cabinet  
Department for Facilities and Support Services  
Division of Engineering and Contract Administration  
403 Wapping Street, 1<sup>st</sup> Floor  
Frankfort, Kentucky 40601

Attention: Mr. Butch Hatcher, Project Manager

Reference: **Final Report of Geotechnical Exploration**  
**Capital Plaza Parking Garage and Office Building**  
Frankfort, Kentucky  
S&ME Project No. 1831-11-430

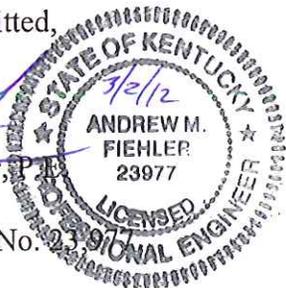
Dear Mr. Hatcher:

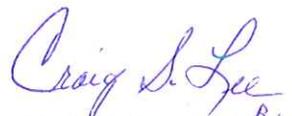
S&ME, Inc. has completed the geotechnical exploration for the proposed office building and parking garage in downtown Frankfort, Kentucky. The purpose of this exploration was to obtain subsurface data at the site pursuant to developing site preparation and foundation recommendations for the proposed project. S&ME previously issued a preliminary geotechnical report dated December 22, 2011. We conducted this project in general accordance with S&ME Proposal No. KY5697Revised, dated September 13, 2011 which was authorized by you. This report explains our understanding of the project, documents our findings, and presents our conclusions and engineering recommendations. Additional design details are still being developed, particularly for the office building. Once known and provided to S&ME, we will issue addendums as needed to address the new information.

S&ME appreciates the opportunity to be of service to the Finance and Administration Cabinet. We look forward to serving as your geotechnical engineering consultant throughout project completion. If you have any questions, please call.

Respectfully submitted,  
S&ME, Inc.

  
Andrew M. Fiehler, P.E.  
Project Engineer  
Kentucky License No. 23977



  
Craig S. Lee, P.E. *By: KSL*  
Senior Engineer

cc: Mr. Darrell Douglas – Sherman Carter Barnhart Architects

Attachments: Report of Geotechnical Exploration

2011 Projects / 1831-11-430 Capital Plaza Report

**FINAL REPORT OF GEOTECHNICAL EXPLORATION  
CAPITAL PLAZA PARKING GARAGE AND OFFICE BUILDING  
Frankfort, Kentucky  
S&ME Project No. 1831-11-430**

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Important Information About Your Geotechnical Engineering Report (ASF E)

Appendix A	Site Location/Topographic Map Boring Location Plan Geophysical Test Location Plan
Appendix B	Test Boring Record Legend Test Boring Records Field Testing Procedures
Appendix C	Summary of Laboratory Test Data Laboratory Testing Procedures
Appendix D	Guide for Concrete Floor and Slab Construction - ACI 302.1R-96

**FINAL REPORT OF GEOTECHNICAL EXPLORATION  
CAPITAL PLAZA PARKING GARAGE AND OFFICE BUILDING  
Frankfort, Kentucky  
S&ME Project No. 1831-11-430**

## **1.0 INTRODUCTION**

S&ME, Inc. has completed the geotechnical exploration for the proposed office building and parking garage in downtown Frankfort, Kentucky. The purpose of this exploration was to obtain subsurface data at the site pursuant to developing site preparation and foundation recommendations for the proposed project. S&ME previously issued a preliminary geotechnical report dated December 22, 2011. We conducted this project in general accordance with S&ME Proposal No. KY5697Revised, dated September 13, 2011 which was authorized by Mr. Butch Hatcher with the Finance and Administration Cabinet. This report explains our understanding of the project, documents our findings, and presents our conclusions and engineering recommendations.

Additional design details are still being developed, particularly for the office building. At present, S&ME has not been provided with detailed structural loads and tolerances for the office building, and anticipated traffic loads as well as several other details. Once these details are known/developed and provided to S&ME, we will issue addendums and revise our recommendations as needed.

## **2.0 PROJECT INFORMATION**

**Proposed Construction** - The project will consist of two phases. The initial phase will be a new six-story, 520 space parking garage adjacent to the Kentucky Transportation Cabinet parking garage. The second phase will be a new 270,000 square foot, six-story office building.

The garage structure will have a footprint of about 30,000 square feet (120 ft x 250 ft) while the office building will have a footprint of about 45,000 square feet (115 ft x 390 ft). The garage will have maximum column loads on the order of 1,700 kips with settlement tolerances of 1½ inches total and ¾ of an inch differential. The maximum column loads of the office building were estimated by Mr. Curtis Byers to be about 1,500 kips. Settlement tolerances of the office building were not yet available.

**Existing Site** – The project site is currently occupied by two city streets (Hill Street and Saint Clair Street) and an underground parking garage. The proposed garage will be situated over the

existing Saint Clair Street while the new office building will be situated, at least partially, over the underground parking garage.

### **3.0 SITE GEOLOGY**

A review of the USGS geologic map of the Frankfort East and West Quadrangles (1968/1975) indicates the project site is underlain by alluvium. The Tyrone Formation is mapped below the alluvium.

Alluvium is general term used for soils deposited by rivers. The alluvium is comprised of stratified clay, silt, sand and pebbles. Specifically, the site is located on an inside bend of the Kentucky River. Interior bends of rivers tend to contain greater thicknesses of the finer grained sediments (i.e. – fine sand, silt and clay) while the coarser materials (i.e. – coarse sands and gravels) are deposited closer to the river channel. Our soil test borings agree with the mapping, encountering clay and silt near the ground surface that transitions to coarser grained deposits nearer to bedrock.

The Tyrone Formation is described as light gray limestone that occurs in mostly thick beds. The Tyrone also contains calcite grains and to a lesser degree chert inclusions. The Tyrone is quarried for use as aggregate in several quarries in the area. The recovered rock cores were classified as limestone consistent with the mapped geology.

The refusal materials at this site were explored by coring rock from three of the soil test borings. For more detailed descriptions of the data obtained from our borings, please refer to our Test Boring Records in Appendix B and the Laboratory Test Data in Appendix C.

### **4.0 EXPLORATION METHODS**

The procedures used by S&ME for field and laboratory sampling and testing are in general accordance with ASTM procedures and established engineering practice. Appendix B contains brief descriptions of the procedures used in this exploration.

#### **4.1 Field Exploration – Soil Test Borings**

A total of 15 soil test borings were performed across the site and were labeled as B-1 through B-15. S&ME engineer Andrew Fiehler, P.E., visited the site to observe pertinent site features, surface indications of the site geology, and to direct drilling operations. Figure 2 in Appendix A shows the locations of the borings.

Two drill rigs, a track mounted Diedrich D-50 and a truck mounted Mobile B-80, were used to perform the borings. The drill rig used for each boring is noted on the Test Boring Records in Appendix B. The drillers obtained soil samples in the soil test borings using a split-barrel sampler driven by an automatic hammer system or rope and cathead hammer system in general accordance with ASTM D1586. The 15 borings were extended to auger refusal. Upon encountering auger refusal, borings B-3, B-9 and B-15 were advanced 10 feet into the bedrock using rock coring techniques. The stratification lines shown on the Test Boring Records represent the approximate boundaries between soil and/or rock surfaces. The transitions may be more gradual than shown.

#### **4.2 Field Exploration – Shear Wave Velocity Measurements**

Shear wave velocity measurements can be obtained using either shear wave surveys such as crosshole and downhole tests or surface wave surveys such as SASW, MASW, MAM, or ReMi™. Analysis of surface waves (R-waves) can be used to determine shear-wave velocities ( $V_s$ ) as surface waves are fundamentally similar in behavior to shear waves (S-waves). In addition, the surface waves propagate to depths that are proportional to their frequencies (i.e., dispersion). The surface waves are recorded at the ground surface along a spread of low-frequency geophones. Recorded surface waves are transformed from time domain into frequency domain, from which the phase characteristics of the surface waves can be determined. A dispersion curve (a.k.a., phase velocity curve, slowness curve) is developed allowing the phase velocity ( $C_f$ ) of particular frequency waves to be calculated. The dispersion curve is then transformed into the shear-wave velocity profile through a complex inversion and iterative processing.

To measure shear-wave velocities, S&ME performed MASW (Multi-Channel Analysis of Surface Waves) and MAM (Microtremor Array Measurements) with non-linear array geometry, combining the dispersion curves from both tests prior to the inversion process. Performing both MASW and MAM provides the greater depth of penetration associated with microtremor analyses (low frequency surface waves) without sacrificing resolution at shallower depths from MASW (higher frequency surface waves). In addition, our experience indicates using a combination of both methods to develop a shear wave velocity profile is more accurate than using Refraction Microtremor (ReMi™) exclusively, particularly when the ReMi™ array geometry is linear.

MASW and MAM tests were performed at two locations (SW-1 and SW-2) to produce two separate shear wave velocity profiles at the site. The MASW and MAM testing was conducted

using the 16-channel Geometrics ES3000 seismograph and 4.5 Hz vertical geophones. For the MASW testing, the geophones were spaced in a linear geometry at intervals of 5- to 10-feet and surface waves generated by a 10-pound sledgehammer striking a metal plate. MAM testing was conducted using an “L-shaped” array geometry with geophone spacing of 30 feet. Because the source locations of the microtremors are not known, the 2-dimensional array geometry is used for the MAM. The analysis was conducted using the OYO Corporation’s SeisImager/SW software (*Pickwin v. 3.14* and *WaveEq*). The analysis of these measurements indicates the  $V_{s100}$  at the site to be 846 ft/sec which places the site in a site seismic classification of “D”. Figure 3 in Appendix A shows the locations of the two arrays.

### **4.3 Laboratory Testing**

Mr. Fiehler sealed and returned the soil samples to our laboratory where he assigned the applicable laboratory tests. These tests are used to determine the engineering properties of the soil. All soil samples were visually classified by the geotechnical engineer in general accordance with the Unified Soil Classification System (ASTM D2487). We conducted natural moisture content determinations, Atterberg limits tests, and grain size determinations on selected soil samples to aid in classification. Representative rock core samples were selected for compressive strength testing to help assess the allowable bearing pressure of the bedrock. The obtained laboratory data and descriptions of these tests are included in Appendix C.

## **5.0 SUBSURFACE CONDITIONS**

### **5.1 General Soil Profile**

Twelve of the 15 borings were advanced in existing lawn areas and encountered between two and six inches of topsoil at the ground surface. Borings B-1, B-12 and B-15 were advanced through the existing concrete roadways. The concrete was measured to be eight inches thick at B-1 and five inches thick at borings B-12 and B-15. Beneath the concrete these borings encountered a layer of base gravel about six to eight inches thick.

Beneath the surficial materials, 11 borings encountered a horizon of previously placed fill that extended to depths of four to 12 feet below the ground surface. The fill classified low plasticity, lean clay (CL) under the Unified Soil Classification System (USCS) with liquid limits of 28 to 35 percent and plasticity indices of 11 to 14 percent. Beneath the fill horizon, our borings lean clay that extended to between 15 and 35 feet below the ground surface. Atterberg limits of the lean clay had liquid limits ranging from 29 to 41 percent with plasticity indices ranging from 10 percent to 18 percent.

Beneath the lean clay horizon, our borings encountered alluvial silt and sand extending to a depth of about 40 to 50 feet. About 40 to 50 feet below the ground surface, the borings encountered medium to coarse grained sand with pea gravel pieces. The grain size of the alluvial soil generally increased with depth. The sand and gravel horizon continued to auger refusal which ranged from about 57 to 74 feet below the ground surface. Refusal elevations varied from 446.7 feet in boring B-7 (on the northern side of the proposed garage) to 428.1 feet in boring B-15 (on the southern side of the proposed garage).

The refusal materials were explored by coring 10 feet of rock from borings B-3, B-9 and B-14. The core runs initially penetrated about 3 feet of weathered rock with clay seams. The recovered rock core samples consisted of limestone that is gray and fine to medium crystalline with interbedded shale partings and laminations. The upper one foot to three feet of the recovered core showed indications of water staining with occasional clay lenses. Unconfined compressive strength testing was performed on selected rock core samples on the interbedded limestone and shale. The tested samples had unconfined compressive strengths ranging from 267 ksf to 713 ksf.

Please refer to the Test Boring Records in Appendix B and the Laboratory Data Summary in Appendix C for additional details of the borings and lab tests.

## **5.2 Groundwater**

The groundwater elevation measured during drilling varied in elevation from about 464 feet to about 482 feet. The depth of the water and duration of flow is directly dependent on recent rainfall activities and site specific drainage characteristics. Based on an average finished floor and site elevation of between 498 and 499 feet, we do not anticipate encountering static groundwater during the surface portions of the project construction. Pockets or zones of trapped water may be encountered; however, we expect that these zones (if encountered) can be handled with excavation of temporary sumps and pumping the water from the excavation. The deep foundation construction will encounter groundwater during installation and the contractor should be prepared accordingly. Additional discussion regarding the recommended foundation type and groundwater are included in the following sections.

For safety purposes the borings were backfilled with auger cuttings after the completion of drilling, therefore, 24-hour water levels were not measured. Management of both surface and subsurface water will be a key issue to development of this site.

## **6.0 CONCLUSIONS AND RECOMMENDATIONS**

### **General Discussion**

Based on our understanding of the project, the project will consist of the following three distinct separate but related phases:

- Construction of the new garage
- Demolition of the existing below grade parking garage
- Construction of the office building

The new garage will be constructed first to create parking spaces that will be lost by demolition of the below grade parking garage. After the new garage is constructed, the existing below grade garage will be demolished and the site prepared for the construction of the new office building. We understand that the design team would like to use on-site demolition materials, such as crushed concrete, for some of the site grading, where possible to reduce debris disposal costs. Once the garage is demolished and the site is re-graded, construction of the new office building will take place.

### **6.1 Foundation Recommendations**

#### **6.1.1 Foundation Discussion**

Based on the provided structural loading, we recommend the use of deep foundations for the garage and office building. There are numerous deep foundations options including: drilled shafts, driven piles, auger-cast piles and micro-piles. Each method has advantages and disadvantages. While these options would provide the required structural support for the foundation loads, it is our opinion that the constructability of a deep foundation system will be one of the main challenges of developing this site.

We have discussed the above mentioned options with several deep foundation contractors and the general consensus of the contractors is that end bearing auger-cast piles (ACP) appear to be the most economical option. The adjacent KYTC parking garage and office building are supported by end bearing ACP. Based on the apparent economic advantage, we recommend considering auger-cast piles (ACP) for the proposed building foundation system. The following sections of the report are tailored for using ACP as the foundation system for the garage and include our recommendations for design and installation of ACP.

The plans for the parking garage include construction of two “express ramps” on the northwest side of the garage. There will be two ramps (one “up” and one “down”) consisting of two triangular shaped parallel concrete walls about 20 feet apart that will retain crushed stone infill for the pavement surface. The pairs of walls for each ramp will be supported by a single mat type foundation with a net maximum bearing pressure of about 2,500 psf. The ramps will reportedly be able to tolerate about 1 ½ inches of total settlement and about ¾ of an inch of differential settlement.

### 6.1.2 Auger Cast Piles

Bedrock at the project site varies from about 55 to 70 feet (elevations of about 443 feet to about 428 feet) below the proposed office building finished floor elevation of 498.5 feet. Generally, the bedrock slopes downhill to the south. Based on the depth to bedrock and the required length to establish adequate friction, we recommend that the piles be designed as end-bearing piles rather than friction piles. We recommend that the piles be designed with a minimum diameter of 16 inches and a maximum allowable axial capacity of 140 kips. We recommend that a maximum uplift capacity of 70 kips per ACP be used for the design. We recommend a minimum ACP spacing of three diameters, measured from center to center of the piles. The design team should include the weight of the ACP in the uplift analysis.

Our recommended soil parameters for the L-PILE analysis of the ACP are shown in the following table. Lateral analysis and the parameters presented assume that the allowable deflection is sufficient to mobilize the soil strength. The recommended parameters were based on the soil conditions observed in our borings, laboratory test results, and published correlations of properties with soil type and consistency. We recommend that the upper three feet of the subsurface profile be ignored in the lateral analysis.

Depth Below Ground Surface (ft)	Elevation	KSOIL (soil type)	K <sub>static</sub> (psi/in)	γ <sub>wet</sub> , (pci) <sup>1</sup>	c, (psi)	e <sub>50</sub>	Φ <sup>2</sup>
3 to 18	495 to 480	1	100	0.069	3.4	0.010	30
18 to 53	480 to 445	4	20	0.053	0	N/A	35
53 to Rock	445 to Rock	4	35	0.053	0	N/A	35

1. USDOT – FHWA Geotechnical Engineering Circular No. 5 – April 2002

2. CERM – Lindeburg – 9<sup>th</sup> Edition

### **Auger Cast Pile Construction Considerations**

Auger-cast piles are constructed by first rotating a continuous flight hollow shaft auger into the ground to a pre-determined depth. In this case, we recommend the augers penetrate at least one foot into the weathered bedrock. Cement grout is then pumped through the auger shaft as the auger is gradually withdrawn, leaving a continuous grout column in the ground.

The grout properties are critical in achieving a well-constructed pile which performs as designed. The grout should include additives which control setting shrinkage. The grout must be fluid enough to be pumped easily and must flow without excessive pressure losses. The grout strength and structural adequacy of the pile section should be reviewed in conjunction with the most recent edition of the Kentucky Building Code and the expected load conditions.

Auger-cast piles may be reinforced with single or bundled reinforcing rods, rolled steel sections, or reinforcing bar cages. All reinforcing should be inserted before the grout sets up, normally within ten minutes after the augers are withdrawn. The reinforcing should be placed in the center of the pile and plumb to avoid having it protrude from the grout into the soil. Because flexible reinforcing rods are difficult to center, they should be installed with a centering device or devices.

Improper grout injection and auger withdrawal techniques can result in low capacity auger-cast piles. Because piles cannot be inspected after construction, the use of proper procedures is extremely important. It is critical that a sufficient volume of grout be continuously pumped at sufficient pressure to prevent suction from developing as the augers are withdrawn. Such suction can cause the soil to mix with the grout, the soils to be disturbed, and the drilled hole to collapse. This action results in a low capacity pile and a reduced cross-sectional area.

The grout should be pumped with sufficient pressure and the auger withdrawn slowly enough to keep the hole filled, to prevent hole collapse, and to cause lateral penetration of the grout into soft or porous zones of surrounding soil. A pressure head of at least 10 feet of grout either above the injection point or above the ground water level, whichever is higher, should be maintained at all times during auger pulls so that the grout has a displacing action and resists the movement of loose material into the hole. The following minimum grout heads are recommended:

**Table 1 - Minimum Grout Heads**

<b>Location of Injection Point</b>	<b>Minimum Grout Head (feet)</b>
Above water table	10
Below water table	20

These pressure heads should be maintained at all times during auger pulls/grout placement so that the grout has a displacing action and resists the movement of loose material into the hole. The auger withdrawal rate should not exceed 10 feet per minute, unless a faster rate can be demonstrated to be acceptable. This method of placement should be used at all times regardless of whether the hole is sufficiently stable to retain its shape without support from the earth-filled auger flights. Please note that the water levels measured during our exploration may not be representative of the groundwater conditions during the time of construction. Groundwater level measurements should be taken daily during auger cast pile construction to determine the minimum grout head requirement.

During pile installation, the following quality control observations should be performed by qualified geotechnical personnel:

- Monitor installation procedures to check that the tip depths are properly achieved and that auger withdrawal techniques are sufficient to remove loose cuttings from the pile.
- Monitor and record the rate of auger penetration and withdrawal.
- Check and calibrate the equipment for controlling and measuring the flow rate of grout into the pile.
- Calculate the ratio of actual grout take to the theoretical hole volume.
- Monitor installation of steel reinforcement.

Our soil test borings did not encounter obstructions within the soil horizons which were difficult for our auger to penetrate. Therefore, we do not anticipate the ACP drills will require special tooling to drill through obstructions such as debris fill, hardpan, etc.

### **6.1.2 Seismic Information**

The current seismic design procedures outlined in the NEHRP (National Earthquake Hazard Reduction Program) guidelines mandate structural design loads be based on the seismic coefficients of the site. To measure shear-wave velocities, S&ME performed MASW (Multi-Channel Analysis of Surface Waves) and MAM (Microtremor Array Measurements) with non-

linear array geometry, combining the dispersion curves from both tests prior to the inversion process. The analysis of these measurements indicates the  $V_{s100}$  at the site to be 846 ft/sec which places the site in a site seismic classification of “D”. This classification is further defined in Table 1613.5.2 in the 2007 Kentucky Building Code.

### **6.1.3 Liquefaction Discussion**

The Mid-American Earthquake Engineering Research Center has established guidelines to evaluate the potential for liquefaction. Table 4-1 in Technical Report MCEER-98-0005 (p.21) stipulates that, in order for a soil to be liquefiable, all of the following criteria must be met:

1. Clay Fraction (% finer than 0.005mm) < 15%
2. Liquid Limit (LL) < 35%
3. Moisture Content (MC) > 0.9LL
4. Depth < 15m
5.  $(N_1)_{60cs} < 30$
6. Soil must be saturated

Our laboratory testing and field observations indicate that several zones meet these six of the criteria. To evaluate if the observed subsurface profile is potentially liquefiable, we performed a liquefaction potential analysis based on the “Simplified Method” for evaluating the liquefaction resistance of soil as described by the paper presented by Youd et al in the October 2001 issue of *Journal of Geotechnical and Geoenvironmental Engineering*. This analysis identified a zone of soil approximately 10 to 15 feet thick, between approximate elevations of 455 feet and 440 feet, with a factor of safety of less than 1 against liquefaction during a seismic event. This zone appears to cover the central portion of the site. Cone Penetrometer (CPT) testing would be helpful to more accurately assess the seismic characteristics of these layers as compared to the SPT method.

## **6.2 Retaining Walls / Express Ramps – Parking Garage**

Portions of the first floor parking garage ramp system and the express ramps will be supported by soil or crushed stone fill. We understand that these retaining walls are typically supported by shallow soil supported foundations even when the main structure is supported by deep foundations. We understand that the express ramp walls will be supported by a mat foundation.

We recommend a maximum allowable soil bearing pressure of 2,500 psf be used for the ramp foundations. Expect that some improvement of portions of the foundation excavations may be required as several of our borings did encounter several three to five feet thick horizons of softer clay at the approximate foundation bearing elevations of 495 feet. The foundations should be

embedded at least 24 inches below the exterior grade for frost protection. Based on empirical correlations of soil types, the provided loading and provided foundation dimensions, we estimate the total settlement of the ramps using the above bearing pressure will be less than the maximum settlement tolerances provided.

The retaining wall and foundation design will depend on the actual materials used to construct the garage ramp subgrade. Below are a list of presumed soil properties for the lean clay we encountered near the ground surface across the majority of the site and #57 crushed limestone aggregate. If soil or another gradation of crushed limestone is used for the actual ramp subgrade, evaluation of the material properties and their affect on the wall will be required.

<b>Material</b>	<b>Compacted Unit Weight - <math>\gamma</math></b>	<b>Phi Angle - <math>\phi</math></b>	<b>Active Pressure – <b>Ka</b></b>	<b>At-Rest Pressure – <b>Ko</b></b>	<b>Passive Pressure – <b>Kp</b></b>
Lean Clay (CL)	125 pcf	26° *	0.4	0.6	2.6
#57 Crushed Limestone	110 pcf	35° *	0.3	0.4	3.7

\* presumed values for internal angle of friction based on typical values for the encountered soil

### 6.3 Floor Slab Recommendations

The lower level of the parking garage will consist of a soil supported slab. Our borings indicate that the soil horizon immediately below the existing roadway and adjacent lawn area is lean clay. The lower level of the proposed office building, which will be a combination of basement and first floor, will also be a soil supported slab. However, the office building will bear on a combination of the existing lean clay on the eastern end and newly placed fill on the western end.

We recommend that control joints be placed in the slab around columns and along footing supported walls to reduce cracking due to minor differential settlements. We suggest a layer of compacted dense graded aggregate (DGA) directly beneath the slab to enhance support and provide a working base for construction of the floor slab. The actual DGA thickness should be based on the floor slab design, but our experience suggests a minimum depth of 6 inches. The DGA should be moist, but not wet, as the concrete is placed to reduce curling of the slab as the concrete cures. We recommend that ACI 302.1R-96 “GUIDE FOR CONCRETE FLOOR AND SLAB CONSTRUCTION” be followed for design and placement of concrete floor slabs. A copy of ACI 302.1R-96 is included in Appendix D of this report for your use.

Between completion of grading/foundation construction and slab construction, the floor slab subgrade is often disturbed by weather, foundation and utility line installation, and other construction activities. For this reason, the subgrade should be evaluated by a geotechnical engineer immediately prior to constructing the slab.

#### **6.4 Pavement**

At the time of the exploration, a site development plan was not yet complete. As such, S&ME did not obtain soil samples for laboratory CBR testing as the potential pavement subgrade materials were not known. In order to allow project design to move forward, we have assumed a CBR value of 3 percent for the on-site lean clay. Once the plan subgrade materials have been determined, confirming CBR testing of the soil should be performed to verify that the soil has a CBR value of at least 3 percent.

The recommended pavement section given below is based on the assumption that any newly placed fill soils for the pavement subgrade have been compacted to at least 95 percent of the standard Proctor maximum dry density at moisture contents ranging from  $\pm 3$  percent of the soil's optimum moisture content as determined by the standard Proctor test.

Minimizing infiltration of water into the subgrade and rapid removal of subsurface water are essential for the successful long-term performance of the pavement. Both the subgrade and the pavement surface should have a minimum slope of one-quarter inch per foot to promote surface drainage. Edges of the pavement should provide a means of water outlet by extending the aggregate base course through to side ditches. Side ditches should be at least 2 feet below the pavement surface.

The materials should conform and be placed and compacted in accordance with the applicable sections of the Kentucky Transportation Cabinet (KTC) Standard Specifications for Road and Bridge Construction, latest edition.

We used the American Association of State Highway and Transportation Officials (AASHTO) Guide for Design of Pavement Structures (1993) as a basis for our flexible pavement thickness analysis. The total pavement thickness requirement is a function of the California bearing ratio (CBR). We have based our design on an **assumed CBR value of 3 percent**. S&ME should be retained to test any soils to be placed as fill to determine if it meets the criteria set forth in this report. If testing of the soils indicates that the actual CBR value is less than 3 percent, S&ME must re-evaluate the following pavement thickness recommendations, and acknowledge any changes in writing.

Anticipated traffic volumes were not available or provided for our use in determining the recommended pavement thickness. The following pavement design recommendations are based on the assumptions of a 20 year service life, a CBR value of 3 percent, 20,000 ESAL’s for light duty pavement, and 50,000 ESAL’s for heavy duty pavement. If actual or anticipated traffic volumes exceed the 50,000 ESAL value used for this design, S&ME must re-evaluate the pavement thickness recommendations. The total pavement thickness requirement is obtained from the AASHTO nomograph in terms of a structural number (SN), a weighted sum of the pavement layer thicknesses accounting for their structural and drainage properties.

S&ME recommends that the pavement section (base stone and asphalt) be placed after the majority of the new building construction has been completed. S&ME recommends that both binder and surface mix asphalt be placed sequentially before traffic is allowed on the new pavement. **S&ME recommends that the light duty pavement section be used for light automobile parking, and that the heavy duty pavement section be used for drive lanes and access lanes.** S&ME recommends the following flexible asphalt pavement sections for this project:

**Flexible Asphalt Pavement Bearing on Soil**

<b>MATERIAL</b>	<b>LIGHT DUTY</b>	<b>HEAVY DUTY</b>	<b>KY TRANSPORTATION CABINET SPECIFICATION</b>
Asphalt Surface Coarse	1-½ Inches	1-½ Inches	Section 400
Asphalt Binder Coarse	3 Inches	4 Inches	Section 400
Crushed Stone Base	8 Inches	10 inches	Section 303

Our pavement recommendations are based on the assumption that S&ME is retained to monitor the installation of the asphalt and base, check the installed thickness of the aggregate materials, and perform in-place density tests. Asphalt placement should be monitored full-time to observe placement and compaction procedures. Asphalt samples should be collected periodically and tested for asphalt cement content, aggregate gradation, and Marshall Density.

**Impervious Concrete Pavement** - We recommend that in areas where heavy, concentrated loads (i.e. - dumpster area, entrances, etc.) are expected or in desired areas, a rigid (concrete) pavement section will be used. For dumpster areas, we recommend that rigid pavement be extended beyond the dumpster pad for the entire length of the garbage truck. The pavement subgrade should consist of soil fill placed in accordance with the recommendations in this report. We recommend that the concrete pavement be supported by at least a 6 inch layer of compacted DGA. The DGA should be compacted to a minimum of 95 percent of the standard Proctor maximum dry density. We recommend a minimum concrete section of 6 inches for this site.

The concrete should be air-entrained and have a 28-day compressive strength of 4,000 psi. Joint spacing should be at a maximum spacing of 20 feet each way.

### **6.5 Using construction debris for backfill**

The demolition of the underground parking garage will generate a significant volume of debris which is anticipated to be primarily concrete. We understand that the design team desires to re-use as much of the demolished concrete as possible on-site to help reduce waste cost. The concrete should be crushed to generate a uniform gradation and aid in removal of reinforcing steel. The final use of the crushed concrete will determine the required gradation or screen size for the crushing operations. Once the crushing operation has commenced, laboratory testing should be performed on samples of the crushed concrete to verify the gradation prior to placing the crushed concrete.

Crushed concrete can be used in just about any situation where crushed limestone aggregate is used, provided it meets gradation requirements if used as free draining aggregate. To be considered free draining, the percent fines must be low. The crushed concrete can be used to fill the underground parking garage to the plan subgrade elevations, provided the total crushed concrete depth does not exceed about three feet. If the crushed concrete depth exceeds about three feet, the ACP installation process may require removal of some of the crushed concrete. Additional discussion about using crushed concrete and other demolition debris is included in the following sections.

### **6.6 General Earthwork Recommendations**

#### *Site Preparation*

Remove the topsoil and trees in the proposed construction areas to prepare the area for construction. We recommend that the root mass of the trees also be removed. If the bottom of the resulting hole is above plan subgrade elevation, the hole should be backfilled with structural fill according to our recommendations presented later in this report. Deleterious materials should be wasted off-site or used in landscape areas that are not proposed for future development.

There were several underground utilities marked by the utility location services within the proposed garage and office building footprint that will likely have to be relocated. We recommend leaving as much of the existing pavement and/or base stone in-place as long as possible to provide a working platform for the ACP drill rigs and other equipment.

#### *Structural Fill Placement*

It appears that the majority of the existing site elevations are above the planned finished floor elevations. However, the existing underground parking garage extends into the proposed office

building footprint. Backfilling of the demolished underground parking garage will be required to achieve the anticipated site grades.

We understand the design team would like to utilize as much of the demolition debris as possible for the new construction. We expect that the majority of the on-site filling activities will take place in the demolished underground parking garage area. This area can be backfilled to the design grade with crushed concrete, crushed CMU blocks, crushed bricks, or structural soil fill. However, we recommend that the office building footprint be backfilled with soil fill as the ACP will likely have difficulty penetrating more than about three feet of crushed debris. The debris fill should be concentrated to the proposed parking lot area or the upper three feet of the building pad.

The crushed debris should have a maximum particle size of 4 inches and be placed in 10 to 12 inch thick lifts and compacted. For quality control, the crushed debris should be tested to determine the gradation at the beginning of the crushing process to verify the crusher set-up. Once verified, production crushing can continue. We recommend that concrete and CMU be crushed separately and mixed during placing. When CMU and concrete are crushed at the same time, the concrete pieces pulverize the brick pieces into fine grained material in the crushing process rather than the desired granular gradation. Care should be taken to remove wood, steel, drywall and other deleterious material prior to crushing as separating after crushing is very difficult.

Ideally, structural soil fill is defined as inorganic natural soil with maximum particles sizes of 3 inches, plasticity index of 30 or less, and maximum dry density of at least 100 pounds per cubic foot (pcf) when tested by the standard Proctor method (ASTM D698). During construction, standard Proctor testing and additional Atterberg limits testing of fill soils (on-site and/or off-site) should be performed by S&ME for compliance with the project specifications before they are used as fill material. If off-site fill is imported, we recommend that the proposed borrow soil be tested prior to transporting it to the site. Please realize that the laboratory conformance testing usually takes three to four business days to complete. Therefore, the contractor should plan accordingly.

Structural fill should be placed in relatively thin (6- to 8-inch thick) layers and compacted to at least 98 percent of the standard Proctor maximum dry density for the building pads and 95 percent of the standard Proctor maximum dry density for pavement areas. Typically, the moisture content of the fill material should be maintained within -2 percent to +3 percent of optimum in order to obtain proper compaction.

In-place density testing must be performed on structural soil fill as a check that the previously recommended compaction criteria have been achieved. This allows our project engineer to monitor the quality of the fill construction and verify that his design criterion is being achieved in the field. We further recommend that these tests be performed on a full-time basis by S&ME. The testing frequency for density tests performed on a full-time basis can be determined by our personnel based on the area to be tested, the grading equipment used, and construction schedule. Tests should be performed at vertical intervals of 8-inches or less (the recommended lift thickness) as the fill is being placed.

#### *Monitoring of Fill Placement*

In-place density testing of structural soil fill must be performed as a check that the previously recommended compaction criteria have been achieved. This allows our project engineer to monitor the quality of the fill construction and verify that his design criterion is being achieved in the field. Performance of slabs-on-grade and foundations will depend directly on the quality of the fill construction. We further recommend that these tests be performed on a full-time basis by S&ME. The testing frequency for density tests performed on a full-time basis can be determined by our personnel based on the area to be tested, the grading equipment used, and construction schedule. Tests should be performed at vertical intervals of 8-inches or less (i.e. - each lift) as the fill is being placed. We recommend that an engineering special inspector working under the direction of our project geotechnical engineer perform the density tests.

Monitoring of crushed debris fill must be done visually by an experienced geotechnical Special Inspector working under strict supervision by one of S&ME's senior geotechnical engineers. The experience of the equipment operator and geo-technician are crucial to achieving the desired performance from the fill. Key indicators include material type, gradation, percentage of fines, and moisture content, equipment used to place the material, uniformity of compactive effort, reduction of voids between concrete pieces, and how the fill material reacts under the equipment. The placement criteria will vary somewhat as the material varies. For example, as the fines content increases, the lift thickness should be decreased.

#### *Site Degradation During Construction*

The on-site soils are sensitive to changes in moisture content. If grading operations are performed during periods of wet weather, these materials will not perform satisfactorily during proofrolling. If soft or wet soils are encountered during the proofrolling observations, we recommend that the area be undercut to stiff native soils or stabilized in-place. An alternative to wasting the wet clay soils is to temporarily stockpile this material for aeration and proper

placement during dryer conditions. We recommend that earthwork for this project be performed during the warm, dry summer months.

## **7.0 FOLLOW UP SERVICES**

Our services should not end with the submission of this report. Field observations, monitoring, and testing during earthwork, foundation and building construction are an extension of the geotechnical design. We recommend that the owner retain S&ME for these services, and that we be allowed to continue our involvement through these phases of the construction. S&ME is not responsible for the conclusions, opinions, or recommendations of others based on the data in this report.

## **8.0 LIMITATIONS OF CONCLUSIONS AND RECOMMENDATIONS**

This report has been prepared for the exclusive use of the Finance and Administration Cabinet for specific application to this project site. Our conclusions and recommendations have been prepared using generally accepted standards of geotechnical engineering practice in the Commonwealth of Kentucky. No other warranty is expressed or implied. This company is not responsible for the conclusions, opinions, or recommendations of others based on these data.

Our conclusions and recommendations are based on the design information furnished to us, the data obtained from the previously described geotechnical exploration, and our past experience. They do not reflect variations in the subsurface conditions that are likely to exist between our borings and in unexplored areas of the site. These variations result from the inherent variability of the general subsurface conditions in this geologic region. If such variations become apparent during construction, it will be necessary for us to re-evaluate our conclusions and recommendations based upon on-site observation of the conditions.

If the overall design or location of the new site improvements is changed, the recommendations contained in this report must not be considered valid unless S&ME reviews the changes and our recommendations are modified and verified in writing. When the design is finalized, retain S&ME to review the foundation plan, grading plan, and applicable portions of the project specifications. This review will allow us to check whether these documents are consistent with the intent of our recommendations.

We may recommend that a supplementary exploration be performed when significant design changes (such as movement of the building or pavement areas) are incorporated in the final design after the geotechnical exploration has been completed. This supplementary exploration may

include obtaining additional soil data at new building corners to provide specific recommendations for foundations.

Field observations, monitoring, and quality assurance testing during earthwork and foundation installation are an extension of the geotechnical design. At that time we can evaluate if the actual conditions are consistent with our design assumptions. We can then modify our recommendations if needed. We recommend that the Owner retain these services and that S&ME be allowed to continue our involvement in the project through these phases of construction. Our firm is not responsible for interpretation of the data contained in this report by others, nor do we accept any responsibility for job site safety, which is the sole responsibility of the contractor.

**FINAL REPORT OF GEOTECHNICAL EXPLORATION  
CAPITAL PLAZA PARKING GARAGE AND OFFICE BUILDING  
Frankfort, Kentucky  
S&ME Project No. 1831-11-430**

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**FINAL REPORT OF GEOTECHNICAL EXPLORATION  
CAPITAL PLAZA PARKING GARAGE AND OFFICE BUILDING  
Frankfort, Kentucky  
S&ME Project No. 1831-11-430**

**1.0 INTRODUCTION**

S&ME, Inc. has completed the geotechnical exploration for the proposed office building and parking garage in downtown Frankfort, Kentucky. The purpose of this exploration was to obtain subsurface data at the site pursuant to developing site preparation and foundation recommendations for the proposed project. S&ME previously issued a preliminary geotechnical report dated December 22, 2011. We conducted this project in general accordance with S&ME Proposal No. KY5697Revised, dated September 13, 2011 which was authorized by Mr. Butch Hatcher with the Finance and Administration Cabinet. This report explains our understanding of the project, documents our findings, and presents our conclusions and engineering recommendations.

Additional design details are still being developed, particularly for the office building. At present, S&ME has not been provided with detailed structural loads and tolerances for the office building, and anticipated traffic loads as well as several other details. Once these details are known/developed and provided to S&ME, we will issue addendums and revise our recommendations as needed.

**2.0 PROJECT INFORMATION**

**Proposed Construction** - The project will consist of two phases. The initial phase will be a new six-story, 520 space parking garage adjacent to the Kentucky Transportation Cabinet parking garage. The second phase will be a new 270,000 square foot, six-story office building.

The garage structure will have a footprint of about 30,000 square feet (120 ft x 250 ft) while the office building will have a footprint of about 45,000 square feet (115 ft x 390 ft). The garage will have maximum column loads on the order of 1,700 kips with settlement tolerances of 1½ inches total and ¾ of an inch differential. The maximum column loads of the office building were estimated by Mr. Curtis Byers to be about 1,500 kips. Settlement tolerances of the office building were not yet available.

**Existing Site** – The project site is currently occupied by two city streets (Hill Street and Saint Clair Street) and an underground parking garage. The proposed garage will be situated over the

existing Saint Clair Street while the new office building will be situated, at least partially, over the underground parking garage.

### **3.0 SITE GEOLOGY**

A review of the USGS geologic map of the Frankfort East and West Quadrangles (1968/1975) indicates the project site is underlain by alluvium. The Tyrone Formation is mapped below the alluvium.

Alluvium is general term used for soils deposited by rivers. The alluvium is comprised of stratified clay, silt, sand and pebbles. Specifically, the site is located on an inside bend of the Kentucky River. Interior bends of rivers tend to contain greater thicknesses of the finer grained sediments (i.e. – fine sand, silt and clay) while the coarser materials (i.e. – coarse sands and gravels) are deposited closer to the river channel. Our soil test borings agree with the mapping, encountering clay and silt near the ground surface that transitions to coarser grained deposits nearer to bedrock.

The Tyrone Formation is described as light gray limestone that occurs in mostly thick beds. The Tyrone also contains calcite grains and to a lesser degree chert inclusions. The Tyrone is quarried for use as aggregate in several quarries in the area. The recovered rock cores were classified as limestone consistent with the mapped geology.

The refusal materials at this site were explored by coring rock from three of the soil test borings. For more detailed descriptions of the data obtained from our borings, please refer to our Test Boring Records in Appendix B and the Laboratory Test Data in Appendix C.

### **4.0 EXPLORATION METHODS**

The procedures used by S&ME for field and laboratory sampling and testing are in general accordance with ASTM procedures and established engineering practice. Appendix B contains brief descriptions of the procedures used in this exploration.

#### **4.1 Field Exploration – Soil Test Borings**

A total of 15 soil test borings were performed across the site and were labeled as B-1 through B-15. S&ME engineer Andrew Fiehler, P.E., visited the site to observe pertinent site features, surface indications of the site geology, and to direct drilling operations. Figure 2 in Appendix A shows the locations of the borings.

Two drill rigs, a track mounted Diedrich D-50 and a truck mounted Mobile B-80, were used to perform the borings. The drill rig used for each boring is noted on the Test Boring Records in Appendix B. The drillers obtained soil samples in the soil test borings using a split-barrel sampler driven by an automatic hammer system or rope and cathead hammer system in general accordance with ASTM D1586. The 15 borings were extended to auger refusal. Upon encountering auger refusal, borings B-3, B-9 and B-15 were advanced 10 feet into the bedrock using rock coring techniques. The stratification lines shown on the Test Boring Records represent the approximate boundaries between soil and/or rock surfaces. The transitions may be more gradual than shown.

#### **4.2 Field Exploration – Shear Wave Velocity Measurements**

Shear wave velocity measurements can be obtained using either shear wave surveys such as crosshole and downhole tests or surface wave surveys such as SASW, MASW, MAM, or ReMi<sup>TM</sup>. Analysis of surface waves (R-waves) can be used to determine shear-wave velocities ( $V_s$ ) as surface waves are fundamentally similar in behavior to shear waves (S-waves). In addition, the surface waves propagate to depths that are proportional to their frequencies (i.e., dispersion). The surface waves are recorded at the ground surface along a spread of low-frequency geophones. Recorded surface waves are transformed from time domain into frequency domain, from which the phase characteristics of the surface waves can be determined. A dispersion curve (a.k.a., phase velocity curve, slowness curve) is developed allowing the phase velocity ( $C_f$ ) of particular frequency waves to be calculated. The dispersion curve is then transformed into the shear-wave velocity profile through a complex inversion and iterative processing.

To measure shear-wave velocities, S&ME performed MASW (Multi-Channel Analysis of Surface Waves) and MAM (Microtremor Array Measurements) with non-linear array geometry, combining the dispersion curves from both tests prior to the inversion process. Performing both MASW and MAM provides the greater depth of penetration associated with microtremor analyses (low frequency surface waves) without sacrificing resolution at shallower depths from MASW (higher frequency surface waves). In addition, our experience indicates using a combination of both methods to develop a shear wave velocity profile is more accurate than using Refraction Microtremor (ReMi<sup>TM</sup>) exclusively, particularly when the ReMi<sup>TM</sup> array geometry is linear.

MASW and MAM tests were performed at two locations (SW-1 and SW-2) to produce two separate shear wave velocity profiles at the site. The MASW and MAM testing was conducted

using the 16-channel Geometrics ES3000 seismograph and 4.5 Hz vertical geophones. For the MASW testing, the geophones were spaced in a linear geometry at intervals of 5- to 10-feet and surface waves generated by a 10-pound sledgehammer striking a metal plate. MAM testing was conducted using an “L-shaped” array geometry with geophone spacing of 30 feet. Because the source locations of the microtremors are not known, the 2-dimensional array geometry is used for the MAM. The analysis was conducted using the OYO Corporation’s SeisImager/SW software (*Pickwin v. 3.14* and *WaveEq*). The analysis of these measurements indicates the  $V_{s100}$  at the site to be 846 ft/sec which places the site in a site seismic classification of “D”. Figure 3 in Appendix A shows the locations of the two arrays.

### **4.3 Laboratory Testing**

Mr. Fiehler sealed and returned the soil samples to our laboratory where he assigned the applicable laboratory tests. These tests are used to determine the engineering properties of the soil. All soil samples were visually classified by the geotechnical engineer in general accordance with the Unified Soil Classification System (ASTM D2487). We conducted natural moisture content determinations, Atterberg limits tests, and grain size determinations on selected soil samples to aid in classification. Representative rock core samples were selected for compressive strength testing to help assess the allowable bearing pressure of the bedrock. The obtained laboratory data and descriptions of these tests are included in Appendix C.

## **5.0 SUBSURFACE CONDITIONS**

### **5.1 General Soil Profile**

Twelve of the 15 borings were advanced in existing lawn areas and encountered between two and six inches of topsoil at the ground surface. Borings B-1, B-12 and B-15 were advanced through the existing concrete roadways. The concrete was measured to be eight inches thick at B-1 and five inches thick at borings B-12 and B-15. Beneath the concrete these borings encountered a layer of base gravel about six to eight inches thick.

Beneath the surficial materials, 11 borings encountered a horizon of previously placed fill that extended to depths of four to 12 feet below the ground surface. The fill classified low plasticity, lean clay (CL) under the Unified Soil Classification System (USCS) with liquid limits of 28 to 35 percent and plasticity indices of 11 to 14 percent. Beneath the fill horizon, our borings lean clay that extended to between 15 and 35 feet below the ground surface. Atterberg limits of the lean clay had liquid limits ranging from 29 to 41 percent with plasticity indices ranging from 10 percent to 18 percent.

Beneath the lean clay horizon, our borings encountered alluvial silt and sand extending to a depth of about 40 to 50 feet. About 40 to 50 feet below the ground surface, the borings encountered medium to coarse grained sand with pea gravel pieces. The grain size of the alluvial soil generally increased with depth. The sand and gravel horizon continued to auger refusal which ranged from about 57 to 74 feet below the ground surface. Refusal elevations varied from 446.7 feet in boring B-7 (on the northern side of the proposed garage) to 428.1 feet in boring B-15 (on the southern side of the proposed garage).

The refusal materials were explored by coring 10 feet of rock from borings B-3, B-9 and B-14. The core runs initially penetrated about 3 feet of weathered rock with clay seams. The recovered rock core samples consisted of limestone that is gray and fine to medium crystalline with interbedded shale partings and laminations. The upper one foot to three feet of the recovered core showed indications of water staining with occasional clay lenses. Unconfined compressive strength testing was performed on selected rock core samples on the interbedded limestone and shale. The tested samples had unconfined compressive strengths ranging from 267 ksf to 713 ksf.

Please refer to the Test Boring Records in Appendix B and the Laboratory Data Summary in Appendix C for additional details of the borings and lab tests.

## **5.2 Groundwater**

The groundwater elevation measured during drilling varied in elevation from about 464 feet to about 482 feet. The depth of the water and duration of flow is directly dependent on recent rainfall activities and site specific drainage characteristics. Based on an average finished floor and site elevation of between 498 and 499 feet, we do not anticipate encountering static groundwater during the surface portions of the project construction. Pockets or zones of trapped water may be encountered; however, we expect that these zones (if encountered) can be handled with excavation of temporary sumps and pumping the water from the excavation. The deep foundation construction will encounter groundwater during installation and the contractor should be prepared accordingly. Additional discussion regarding the recommended foundation type and groundwater are included in the following sections.

For safety purposes the borings were backfilled with auger cuttings after the completion of drilling, therefore, 24-hour water levels were not measured. Management of both surface and subsurface water will be a key issue to development of this site.

## **6.0 CONCLUSIONS AND RECOMMENDATIONS**

### **General Discussion**

Based on our understanding of the project, the project will consist of the following three distinct separate but related phases:

- Construction of the new garage
- Demolition of the existing below grade parking garage
- Construction of the office building

The new garage will be constructed first to create parking spaces that will be lost by demolition of the below grade parking garage. After the new garage is constructed, the existing below grade garage will be demolished and the site prepared for the construction of the new office building. We understand that the design team would like to use on-site demolition materials, such as crushed concrete, for some of the site grading, where possible to reduce debris disposal costs. Once the garage is demolished and the site is re-graded, construction of the new office building will take place.

### **6.1 Foundation Recommendations**

#### **6.1.1 Foundation Discussion**

Based on the provided structural loading, we recommend the use of deep foundations for the garage and office building. There are numerous deep foundations options including: drilled shafts, driven piles, auger-cast piles and micro-piles. Each method has advantages and disadvantages. While these options would provide the required structural support for the foundation loads, it is our opinion that the constructability of a deep foundation system will be one of the main challenges of developing this site.

We have discussed the above mentioned options with several deep foundation contractors and the general consensus of the contractors is that end bearing auger-cast piles (ACP) appear to be the most economical option. The adjacent KYTC parking garage and office building are supported by end bearing ACP. Based on the apparent economic advantage, we recommend considering auger-cast piles (ACP) for the proposed building foundation system. The following sections of the report are tailored for using ACP as the foundation system for the garage and include our recommendations for design and installation of ACP.

The plans for the parking garage include construction of two “express ramps” on the northwest side of the garage. There will be two ramps (one “up” and one “down”) consisting of two triangular shaped parallel concrete walls about 20 feet apart that will retain crushed stone infill for the pavement surface. The pairs of walls for each ramp will be supported by a single mat type foundation with a net maximum bearing pressure of about 2,500 psf. The ramps will reportedly be able to tolerate about 1 ½ inches of total settlement and about ¾ of an inch of differential settlement.

### 6.1.2 Auger Cast Piles

Bedrock at the project site varies from about 55 to 70 feet (elevations of about 443 feet to about 428 feet) below the proposed office building finished floor elevation of 498.5 feet. Generally, the bedrock slopes downhill to the south. Based on the depth to bedrock and the required length to establish adequate friction, we recommend that the piles be designed as end-bearing piles rather than friction piles. We recommend that the piles be designed with a minimum diameter of 16 inches and a maximum allowable axial capacity of 140 kips. We recommend that a maximum uplift capacity of 70 kips per ACP be used for the design. We recommend a minimum ACP spacing of three diameters, measured from center to center of the piles. The design team should include the weight of the ACP in the uplift analysis.

Our recommended soil parameters for the L-PILE analysis of the ACP are shown in the following table. Lateral analysis and the parameters presented assume that the allowable deflection is sufficient to mobilize the soil strength. The recommended parameters were based on the soil conditions observed in our borings, laboratory test results, and published correlations of properties with soil type and consistency. We recommend that the upper three feet of the subsurface profile be ignored in the lateral analysis.

Depth Below Ground Surface (ft)	Elevation	KSOIL (soil type)	K <sub>static</sub> (psi/in)	γ <sub>wet</sub> , (pci) <sup>1</sup>	c, (psi)	e <sub>50</sub>	Φ <sup>2</sup>
3 to 18	495 to 480	1	100	0.069	3.4	0.010	30
18 to 53	480 to 445	4	20	0.053	0	N/A	35
53 to Rock	445 to Rock	4	35	0.053	0	N/A	35

1. USDOT – FHWA Geotechnical Engineering Circular No. 5 – April 2002
2. CERM – Lindeburg – 9<sup>th</sup> Edition

### **Auger Cast Pile Construction Considerations**

Auger-cast piles are constructed by first rotating a continuous flight hollow shaft auger into the ground to a pre-determined depth. In this case, we recommend the augers penetrate at least one foot into the weathered bedrock. Cement grout is then pumped through the auger shaft as the auger is gradually withdrawn, leaving a continuous grout column in the ground.

The grout properties are critical in achieving a well-constructed pile which performs as designed. The grout should include additives which control setting shrinkage. The grout must be fluid enough to be pumped easily and must flow without excessive pressure losses. The grout strength and structural adequacy of the pile section should be reviewed in conjunction with the most recent edition of the Kentucky Building Code and the expected load conditions.

Auger-cast piles may be reinforced with single or bundled reinforcing rods, rolled steel sections, or reinforcing bar cages. All reinforcing should be inserted before the grout sets up, normally within ten minutes after the augers are withdrawn. The reinforcing should be placed in the center of the pile and plumb to avoid having it protrude from the grout into the soil. Because flexible reinforcing rods are difficult to center, they should be installed with a centering device or devices.

Improper grout injection and auger withdrawal techniques can result in low capacity auger-cast piles. Because piles cannot be inspected after construction, the use of proper procedures is extremely important. It is critical that a sufficient volume of grout be continuously pumped at sufficient pressure to prevent suction from developing as the augers are withdrawn. Such suction can cause the soil to mix with the grout, the soils to be disturbed, and the drilled hole to collapse. This action results in a low capacity pile and a reduced cross-sectional area.

The grout should be pumped with sufficient pressure and the auger withdrawn slowly enough to keep the hole filled, to prevent hole collapse, and to cause lateral penetration of the grout into soft or porous zones of surrounding soil. A pressure head of at least 10 feet of grout either above the injection point or above the ground water level, whichever is higher, should be maintained at all times during auger pulls so that the grout has a displacing action and resists the movement of loose material into the hole. The following minimum grout heads are recommended:

**Table 1 - Minimum Grout Heads**

<b>Location of Injection Point</b>	<b>Minimum Grout Head (feet)</b>
Above water table	10
Below water table	20

These pressure heads should be maintained at all times during auger pulls/grout placement so that the grout has a displacing action and resists the movement of loose material into the hole. The auger withdrawal rate should not exceed 10 feet per minute, unless a faster rate can be demonstrated to be acceptable. This method of placement should be used at all times regardless of whether the hole is sufficiently stable to retain its shape without support from the earth-filled auger flights. Please note that the water levels measured during our exploration may not be representative of the groundwater conditions during the time of construction. Groundwater level measurements should be taken daily during auger cast pile construction to determine the minimum grout head requirement.

During pile installation, the following quality control observations should be performed by qualified geotechnical personnel:

- Monitor installation procedures to check that the tip depths are properly achieved and that auger withdrawal techniques are sufficient to remove loose cuttings from the pile.
- Monitor and record the rate of auger penetration and withdrawal.
- Check and calibrate the equipment for controlling and measuring the flow rate of grout into the pile.
- Calculate the ratio of actual grout take to the theoretical hole volume.
- Monitor installation of steel reinforcement.

Our soil test borings did not encounter obstructions within the soil horizons which were difficult for our auger to penetrate. Therefore, we do not anticipate the ACP drills will require special tooling to drill through obstructions such as debris fill, hardpan, etc.

### **6.1.2 Seismic Information**

The current seismic design procedures outlined in the NEHRP (National Earthquake Hazard Reduction Program) guidelines mandate structural design loads be based on the seismic coefficients of the site. To measure shear-wave velocities, S&ME performed MASW (Multi-Channel Analysis of Surface Waves) and MAM (Microtremor Array Measurements) with non-

linear array geometry, combining the dispersion curves from both tests prior to the inversion process. The analysis of these measurements indicates the  $V_{s100}$  at the site to be 846 ft/sec which places the site in a site seismic classification of “D”. This classification is further defined in Table 1613.5.2 in the 2007 Kentucky Building Code.

### **6.1.3 Liquefaction Discussion**

The Mid-American Earthquake Engineering Research Center has established guidelines to evaluate the potential for liquefaction. Table 4-1 in Technical Report MCEER-98-0005 (p.21) stipulates that, in order for a soil to be liquefiable, all of the following criteria must be met:

1. Clay Fraction (% finer than 0.005mm) < 15%
2. Liquid Limit (LL) < 35%
3. Moisture Content (MC) > 0.9LL
4. Depth < 15m
5.  $(N_1)_{60cs} < 30$
6. Soil must be saturated

Our laboratory testing and field observations indicate that several zones meet these six of the criteria. To evaluate if the observed subsurface profile is potentially liquefiable, we performed a liquefaction potential analysis based on the “Simplified Method” for evaluating the liquefaction resistance of soil as described by the paper presented by Youd et al in the October 2001 issue of *Journal of Geotechnical and Geoenvironmental Engineering*. This analysis identified a zone of soil approximately 10 to 15 feet thick, between approximate elevations of 455 feet and 440 feet, with a factor of safety of less than 1 against liquefaction during a seismic event. This zone appears to cover the central portion of the site. Cone Penetrometer (CPT) testing would be helpful to more accurately assess the seismic characteristics of these layers as compared to the SPT method.

## **6.2 Retaining Walls / Express Ramps – Parking Garage**

Portions of the first floor parking garage ramp system and the express ramps will be supported by soil or crushed stone fill. We understand that these retaining walls are typically supported by shallow soil supported foundations even when the main structure is supported by deep foundations. We understand that the express ramp walls will be supported by a mat foundation.

We recommend a maximum allowable soil bearing pressure of 2,500 psf be used for the ramp foundations. Expect that some improvement of portions of the foundation excavations may be required as several of our borings did encounter several three to five feet thick horizons of softer clay at the approximate foundation bearing elevations of 495 feet. The foundations should be

embedded at least 24 inches below the exterior grade for frost protection. Based on empirical correlations of soil types, the provided loading and provided foundation dimensions, we estimate the total settlement of the ramps using the above bearing pressure will be less than the maximum settlement tolerances provided.

The retaining wall and foundation design will depend on the actual materials used to construct the garage ramp subgrade. Below are a list of presumed soil properties for the lean clay we encountered near the ground surface across the majority of the site and #57 crushed limestone aggregate. If soil or another gradation of crushed limestone is used for the actual ramp subgrade, evaluation of the material properties and their affect on the wall will be required.

<b>Material</b>	<b>Compacted Unit Weight - <math>\gamma</math></b>	<b>Phi Angle - <math>\phi</math></b>	<b>Active Pressure – <b>Ka</b></b>	<b>At-Rest Pressure – <b>Ko</b></b>	<b>Passive Pressure – <b>Kp</b></b>
Lean Clay (CL)	125 pcf	26° *	0.4	0.6	2.6
#57 Crushed Limestone	110 pcf	35° *	0.3	0.4	3.7

\* presumed values for internal angle of friction based on typical values for the encountered soil

### 6.3 Floor Slab Recommendations

The lower level of the parking garage will consist of a soil supported slab. Our borings indicate that the soil horizon immediately below the existing roadway and adjacent lawn area is lean clay. The lower level of the proposed office building, which will be a combination of basement and first floor, will also be a soil supported slab. However, the office building will bear on a combination of the existing lean clay on the eastern end and newly placed fill on the western end.

We recommend that control joints be placed in the slab around columns and along footing supported walls to reduce cracking due to minor differential settlements. We suggest a layer of compacted dense graded aggregate (DGA) directly beneath the slab to enhance support and provide a working base for construction of the floor slab. The actual DGA thickness should be based on the floor slab design, but our experience suggests a minimum depth of 6 inches. The DGA should be moist, but not wet, as the concrete is placed to reduce curling of the slab as the concrete cures. We recommend that ACI 302.1R-96 “GUIDE FOR CONCRETE FLOOR AND SLAB CONSTRUCTION” be followed for design and placement of concrete floor slabs. A copy of ACI 302.1R-96 is included in Appendix D of this report for your use.

Between completion of grading/foundation construction and slab construction, the floor slab subgrade is often disturbed by weather, foundation and utility line installation, and other construction activities. For this reason, the subgrade should be evaluated by a geotechnical engineer immediately prior to constructing the slab.

#### **6.4 Pavement**

At the time of the exploration, a site development plan was not yet complete. As such, S&ME did not obtain soil samples for laboratory CBR testing as the potential pavement subgrade materials were not known. In order to allow project design to move forward, we have assumed a CBR value of 3 percent for the on-site lean clay. Once the plan subgrade materials have been determined, confirming CBR testing of the soil should be performed to verify that the soil has a CBR value of at least 3 percent.

The recommended pavement section given below is based on the assumption that any newly placed fill soils for the pavement subgrade have been compacted to at least 95 percent of the standard Proctor maximum dry density at moisture contents ranging from  $\pm 3$  percent of the soil's optimum moisture content as determined by the standard Proctor test.

Minimizing infiltration of water into the subgrade and rapid removal of subsurface water are essential for the successful long-term performance of the pavement. Both the subgrade and the pavement surface should have a minimum slope of one-quarter inch per foot to promote surface drainage. Edges of the pavement should provide a means of water outlet by extending the aggregate base course through to side ditches. Side ditches should be at least 2 feet below the pavement surface.

The materials should conform and be placed and compacted in accordance with the applicable sections of the Kentucky Transportation Cabinet (KTC) Standard Specifications for Road and Bridge Construction, latest edition.

We used the American Association of State Highway and Transportation Officials (AASHTO) Guide for Design of Pavement Structures (1993) as a basis for our flexible pavement thickness analysis. The total pavement thickness requirement is a function of the California bearing ratio (CBR). We have based our design on an **assumed CBR value of 3 percent**. S&ME should be retained to test any soils to be placed as fill to determine if it meets the criteria set forth in this report. If testing of the soils indicates that the actual CBR value is less than 3 percent, S&ME must re-evaluate the following pavement thickness recommendations, and acknowledge any changes in writing.

Anticipated traffic volumes were not available or provided for our use in determining the recommended pavement thickness. The following pavement design recommendations are based on the assumptions of a 20 year service life, a CBR value of 3 percent, 20,000 ESAL’s for light duty pavement, and 50,000 ESAL’s for heavy duty pavement. If actual or anticipated traffic volumes exceed the 50,000 ESAL value used for this design, S&ME must re-evaluate the pavement thickness recommendations. The total pavement thickness requirement is obtained from the AASHTO nomograph in terms of a structural number (SN), a weighted sum of the pavement layer thicknesses accounting for their structural and drainage properties.

S&ME recommends that the pavement section (base stone and asphalt) be placed after the majority of the new building construction has been completed. S&ME recommends that both binder and surface mix asphalt be placed sequentially before traffic is allowed on the new pavement. **S&ME recommends that the light duty pavement section be used for light automobile parking, and that the heavy duty pavement section be used for drive lanes and access lanes.** S&ME recommends the following flexible asphalt pavement sections for this project:

**Flexible Asphalt Pavement Bearing on Soil**

<b>MATERIAL</b>	<b>LIGHT DUTY</b>	<b>HEAVY DUTY</b>	<b>KY TRANSPORTATION CABINET SPECIFICATION</b>
Asphalt Surface Coarse	1-½ Inches	1-½ Inches	Section 400
Asphalt Binder Coarse	3 Inches	4 Inches	Section 400
Crushed Stone Base	8 Inches	10 inches	Section 303

Our pavement recommendations are based on the assumption that S&ME is retained to monitor the installation of the asphalt and base, check the installed thickness of the aggregate materials, and perform in-place density tests. Asphalt placement should be monitored full-time to observe placement and compaction procedures. Asphalt samples should be collected periodically and tested for asphalt cement content, aggregate gradation, and Marshall Density.

**Impervious Concrete Pavement** - We recommend that in areas where heavy, concentrated loads (i.e. - dumpster area, entrances, etc.) are expected or in desired areas, a rigid (concrete) pavement section will be used. For dumpster areas, we recommend that rigid pavement be extended beyond the dumpster pad for the entire length of the garbage truck. The pavement subgrade should consist of soil fill placed in accordance with the recommendations in this report. We recommend that the concrete pavement be supported by at least a 6 inch layer of compacted DGA. The DGA should be compacted to a minimum of 95 percent of the standard Proctor maximum dry density. We recommend a minimum concrete section of 6 inches for this site.

The concrete should be air-entrained and have a 28-day compressive strength of 4,000 psi. Joint spacing should be at a maximum spacing of 20 feet each way.

### **6.5 Using construction debris for backfill**

The demolition of the underground parking garage will generate a significant volume of debris which is anticipated to be primarily concrete. We understand that the design team desires to re-use as much of the demolished concrete as possible on-site to help reduce waste cost. The concrete should be crushed to generate a uniform gradation and aid in removal of reinforcing steel. The final use of the crushed concrete will determine the required gradation or screen size for the crushing operations. Once the crushing operation has commenced, laboratory testing should be performed on samples of the crushed concrete to verify the gradation prior to placing the crushed concrete.

Crushed concrete can be used in just about any situation where crushed limestone aggregate is used, provided it meets gradation requirements if used as free draining aggregate. To be considered free draining, the percent fines must be low. The crushed concrete can be used to fill the underground parking garage to the plan subgrade elevations, provided the total crushed concrete depth does not exceed about three feet. If the crushed concrete depth exceeds about three feet, the ACP installation process may require removal of some of the crushed concrete. Additional discussion about using crushed concrete and other demolition debris is included in the following sections.

### **6.6 General Earthwork Recommendations**

#### *Site Preparation*

Remove the topsoil and trees in the proposed construction areas to prepare the area for construction. We recommend that the root mass of the trees also be removed. If the bottom of the resulting hole is above plan subgrade elevation, the hole should be backfilled with structural fill according to our recommendations presented later in this report. Deleterious materials should be wasted off-site or used in landscape areas that are not proposed for future development.

There were several underground utilities marked by the utility location services within the proposed garage and office building footprint that will likely have to be relocated. We recommend leaving as much of the existing pavement and/or base stone in-place as long as possible to provide a working platform for the ACP drill rigs and other equipment.

#### *Structural Fill Placement*

It appears that the majority of the existing site elevations are above the planned finished floor elevations. However, the existing underground parking garage extends into the proposed office

building footprint. Backfilling of the demolished underground parking garage will be required to achieve the anticipated site grades.

We understand the design team would like to utilize as much of the demolition debris as possible for the new construction. We expect that the majority of the on-site filling activities will take place in the demolished underground parking garage area. This area can be backfilled to the design grade with crushed concrete, crushed CMU blocks, crushed bricks, or structural soil fill. However, we recommend that the office building footprint be backfilled with soil fill as the ACP will likely have difficulty penetrating more than about three feet of crushed debris. The debris fill should be concentrated to the proposed parking lot area or the upper three feet of the building pad.

The crushed debris should have a maximum particle size of 4 inches and be placed in 10 to 12 inch thick lifts and compacted. For quality control, the crushed debris should be tested to determine the gradation at the beginning of the crushing process to verify the crusher set-up. Once verified, production crushing can continue. We recommend that concrete and CMU be crushed separately and mixed during placing. When CMU and concrete are crushed at the same time, the concrete pieces pulverize the brick pieces into fine grained material in the crushing process rather than the desired granular gradation. Care should be taken to remove wood, steel, drywall and other deleterious material prior to crushing as separating after crushing is very difficult.

Ideally, structural soil fill is defined as inorganic natural soil with maximum particles sizes of 3 inches, plasticity index of 30 or less, and maximum dry density of at least 100 pounds per cubic foot (pcf) when tested by the standard Proctor method (ASTM D698). During construction, standard Proctor testing and additional Atterberg limits testing of fill soils (on-site and/or off-site) should be performed by S&ME for compliance with the project specifications before they are used as fill material. If off-site fill is imported, we recommend that the proposed borrow soil be tested prior to transporting it to the site. Please realize that the laboratory conformance testing usually takes three to four business days to complete. Therefore, the contractor should plan accordingly.

Structural fill should be placed in relatively thin (6- to 8-inch thick) layers and compacted to at least 98 percent of the standard Proctor maximum dry density for the building pads and 95 percent of the standard Proctor maximum dry density for pavement areas. Typically, the moisture content of the fill material should be maintained within -2 percent to +3 percent of optimum in order to obtain proper compaction.

In-place density testing must be performed on structural soil fill as a check that the previously recommended compaction criteria have been achieved. This allows our project engineer to monitor the quality of the fill construction and verify that his design criterion is being achieved in the field. We further recommend that these tests be performed on a full-time basis by S&ME. The testing frequency for density tests performed on a full-time basis can be determined by our personnel based on the area to be tested, the grading equipment used, and construction schedule. Tests should be performed at vertical intervals of 8-inches or less (the recommended lift thickness) as the fill is being placed.

#### *Monitoring of Fill Placement*

In-place density testing of structural soil fill must be performed as a check that the previously recommended compaction criteria have been achieved. This allows our project engineer to monitor the quality of the fill construction and verify that his design criterion is being achieved in the field. Performance of slabs-on-grade and foundations will depend directly on the quality of the fill construction. We further recommend that these tests be performed on a full-time basis by S&ME. The testing frequency for density tests performed on a full-time basis can be determined by our personnel based on the area to be tested, the grading equipment used, and construction schedule. Tests should be performed at vertical intervals of 8-inches or less (i.e. - each lift) as the fill is being placed. We recommend that an engineering special inspector working under the direction of our project geotechnical engineer perform the density tests.

Monitoring of crushed debris fill must be done visually by an experienced geotechnical Special Inspector working under strict supervision by one of S&ME's senior geotechnical engineers. The experience of the equipment operator and geo-technician are crucial to achieving the desired performance from the fill. Key indicators include material type, gradation, percentage of fines, and moisture content, equipment used to place the material, uniformity of compactive effort, reduction of voids between concrete pieces, and how the fill material reacts under the equipment. The placement criteria will vary somewhat as the material varies. For example, as the fines content increases, the lift thickness should be decreased.

#### *Site Degradation During Construction*

The on-site soils are sensitive to changes in moisture content. If grading operations are performed during periods of wet weather, these materials will not perform satisfactorily during proofrolling. If soft or wet soils are encountered during the proofrolling observations, we recommend that the area be undercut to stiff native soils or stabilized in-place. An alternative to wasting the wet clay soils is to temporarily stockpile this material for aeration and proper

placement during dryer conditions. We recommend that earthwork for this project be performed during the warm, dry summer months.

## **7.0 FOLLOW UP SERVICES**

Our services should not end with the submission of this report. Field observations, monitoring, and testing during earthwork, foundation and building construction are an extension of the geotechnical design. We recommend that the owner retain S&ME for these services, and that we be allowed to continue our involvement through these phases of the construction. S&ME is not responsible for the conclusions, opinions, or recommendations of others based on the data in this report.

## **8.0 LIMITATIONS OF CONCLUSIONS AND RECOMMENDATIONS**

This report has been prepared for the exclusive use of the Finance and Administration Cabinet for specific application to this project site. Our conclusions and recommendations have been prepared using generally accepted standards of geotechnical engineering practice in the Commonwealth of Kentucky. No other warranty is expressed or implied. This company is not responsible for the conclusions, opinions, or recommendations of others based on these data.

Our conclusions and recommendations are based on the design information furnished to us, the data obtained from the previously described geotechnical exploration, and our past experience. They do not reflect variations in the subsurface conditions that are likely to exist between our borings and in unexplored areas of the site. These variations result from the inherent variability of the general subsurface conditions in this geologic region. If such variations become apparent during construction, it will be necessary for us to re-evaluate our conclusions and recommendations based upon on-site observation of the conditions.

If the overall design or location of the new site improvements is changed, the recommendations contained in this report must not be considered valid unless S&ME reviews the changes and our recommendations are modified and verified in writing. When the design is finalized, retain S&ME to review the foundation plan, grading plan, and applicable portions of the project specifications. This review will allow us to check whether these documents are consistent with the intent of our recommendations.

We may recommend that a supplementary exploration be performed when significant design changes (such as movement of the building or pavement areas) are incorporated in the final design after the geotechnical exploration has been completed. This supplementary exploration may

include obtaining additional soil data at new building corners to provide specific recommendations for foundations.

Field observations, monitoring, and quality assurance testing during earthwork and foundation installation are an extension of the geotechnical design. At that time we can evaluate if the actual conditions are consistent with our design assumptions. We can then modify our recommendations if needed. We recommend that the Owner retain these services and that S&ME be allowed to continue our involvement in the project through these phases of construction. Our firm is not responsible for interpretation of the data contained in this report by others, nor do we accept any responsibility for job site safety, which is the sole responsibility of the contractor.

# 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 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 your ASFE-member geotechnical engineer for more information.



8811 Colesville Road/Suite G106, Silver Spring, MD 20910

Telephone: 301/565-2733 Facsimile: 301/589-2017

e-mail: [info@asfe.org](mailto:info@asfe.org) [www.asfe.org](http://www.asfe.org)

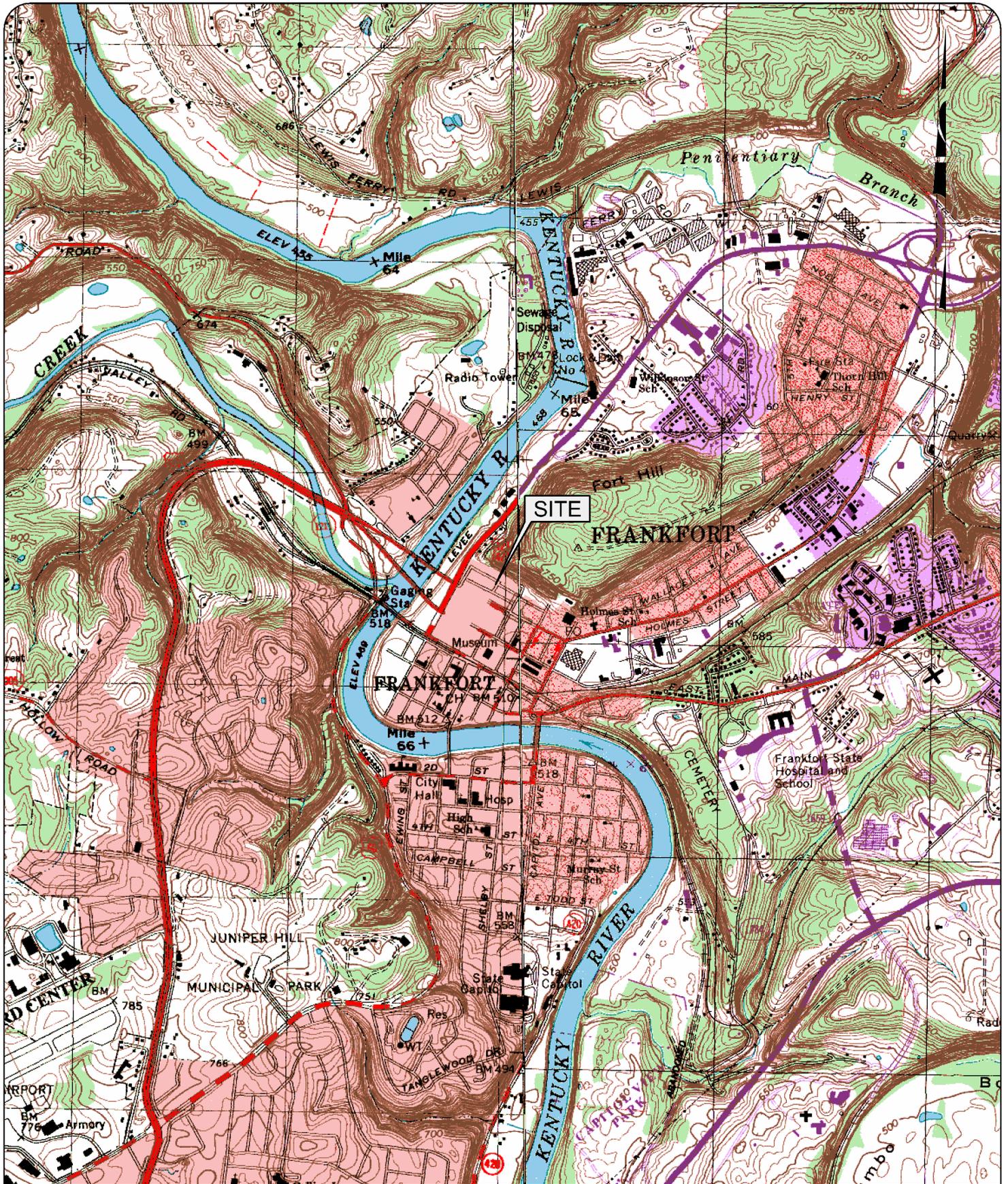
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## **APPENDIX A**

**SITE LOCATION/TOPOGRAPHIC MAP**

**BORING LOCATION PLAN**

**GEOPHYSICAL TEST LOCATION PLAN**



SCALE: 1" = 2000'  
 DATE: 10/27/11  
 DRAWN BY: CAC  
 PROJECT NO: 1831-11-430



**S&ME**  
[WWW.SMEINC.COM](http://WWW.SMEINC.COM)  
 422 CODELL DRIVE, LEXINGTON, KY 40509  
 PHONE: 859.293.5518

**CAPITAL PLAZA  
 FRANKFORT, KENTUCKY**

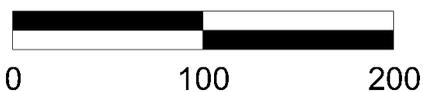
FIGURE NO.

1



AERIAL IMAGE  
GOOGLE MAPS  
DATED 2009

GRAPHIC SCALE IN FEET



WWW.SMEINC.COM  
422 CODELL DRIVE, LEXINGTON, KY 40509  
PHONE: 859.293.5518

SCALE:

1" = 100'

DATE:

12/06/11

PROJECT NUMBER:

1831-11-430

DRAWN BY:

CAC

DRAWING NUMBER:

B-1

CHECKED BY:

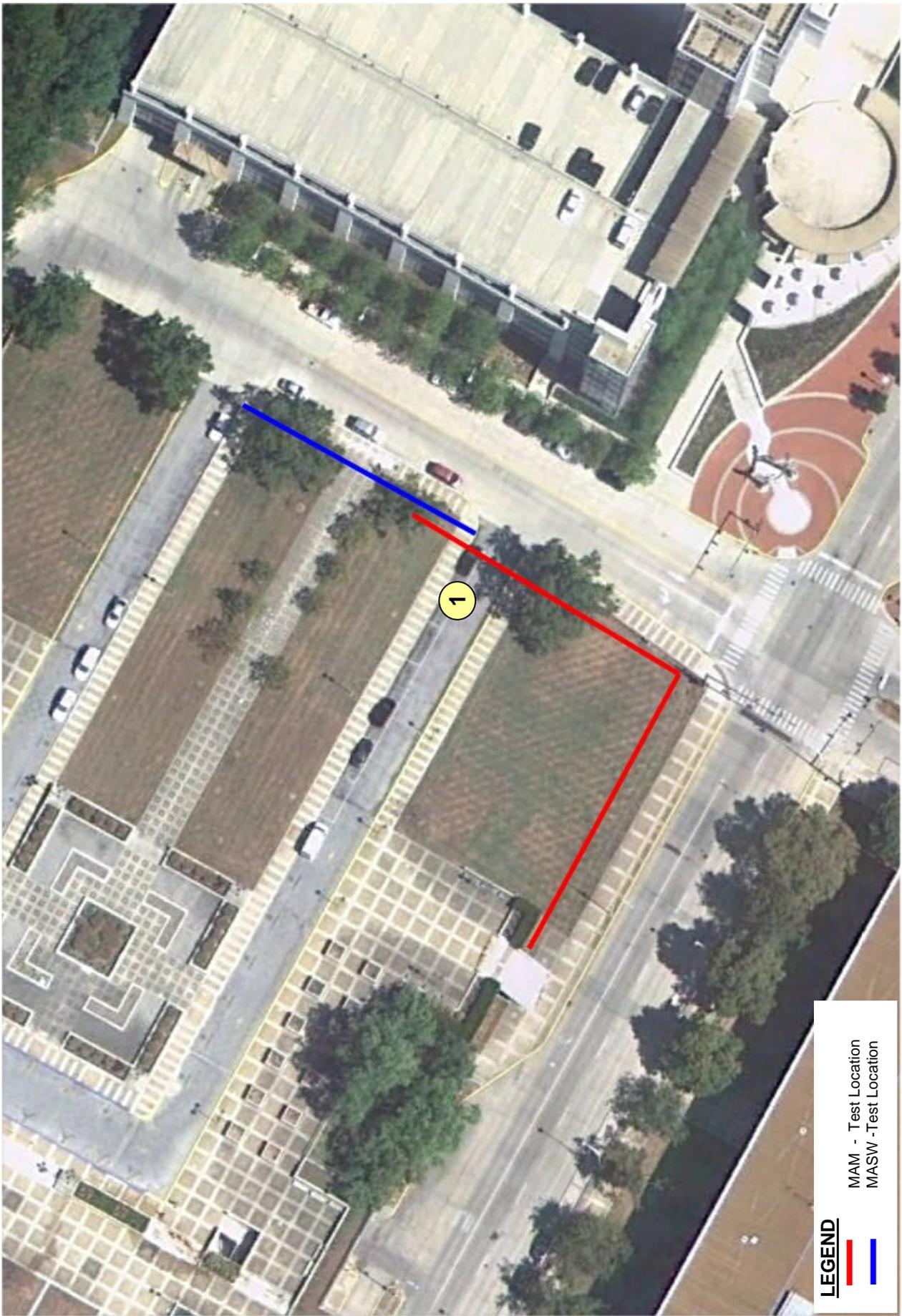
AF

CAPITAL PLAZA  
FRANKFORT, KENTUCKY

BORING LOCATION PLAN

FIGURE NO.

2



**LEGEND**

— MAM - Test Location  
— MASW - Test Location

FIGURE NO. **3**

**GEOPHYSICAL TEST LOCATION PLAN**  
 Capital Plaza  
 Frankfort, Kentucky



SCALE:	NTS
DRAWN BY:	JBC
CHECKED BY:	
DATE:	10/3/2011

PROJECT NO.: 1831-11-430

## **APPENDIX B**

### **TEST BORING RECORDS LEGEND**

#### **TEST BORING RECORDS**

#### **FIELD TESTING PROCEDURES**

# TEST BORING RECORD LEGEND

## FINE AND COARSE GRAINED SOIL INFORMATION

COARSE GRAINED SOILS (SANDS & GRAVELS)		FINE GRAINED SOILS (SILTS & CLAYS)			PARTICLE SIZE	
N	Relative Density	N	Consistency	Qu, KSF Estimated		
0-4	Very Loose	0-1	Very Soft	0-0.5	Boulders	Greater than 300 mm (12 in)
5-10	Loose	2-4	Soft	0.5-1	Cobbles	75 mm to 300 mm (3 to 12 in)
11-20	Firm	5-8	Firm	1-2	Gravel	4.74 mm to 75 mm (3/16 to 3 in)
21-30	Very Firm	9-15	Stiff	2-4	Coarse Sand	2 mm to 4.75 mm
31-50	Dense	16-30	Very Stiff	4-8	Medium Sand	0.425 mm to 2 mm
Over 50	Very Dense	Over 31	Hard	8+	Fine Sand	0.075 mm to 0.425 mm
					Silts & Clays	Less than 0.075 mm

The **STANDARD PENETRATION TEST** as defined by ASTM D 1586 is a method to obtain a disturbed soil sample for examination and testing and to obtain relative density and consistency information. A standard 1.4-inch I.D./2-inch O.D. split-barrel sampler is driven three 6-inch increments with a 140 lb. hammer falling 30 inches. The hammer can either be of a trip, free-fall design, or actuated by a rope and cathead. The blow counts required to drive the sampler the final two increments are added together and designate the N-value defined in the above tables.

## ROCK PROPERTIES

ROCK QUALITY DESIGNATION (RQD)		ROCK HARDNESS	
Percent RQD	Quality		
0-25	Very Poor	Very Hard:	Rock can be broken by heavy hammer blows.
25-50	Poor	Hard:	Rock cannot be broken by thumb pressure, but can be broken by moderate hammer blows.
50-75	Fair	Moderately Hard:	Small pieces can be broken off along sharp edges by considerable hard thumb pressure; can be broken with light hammer blows.
75-90	Good	Soft:	Rock is coherent but breaks very easily with thumb pressure at sharp edges and crumbles with firm hand pressure.
90-100	Excellent	Very Soft:	Rock disintegrates or easily compresses when touched; can be hard to very hard soil.

	$\text{Recovery} = \frac{\text{Length of Rock Core Recovered}}{\text{Length of Core Run}} \times 100$		$\text{RQD} = \frac{\text{Sum of 4 in. and longer Rock Pieces Recovered}}{\text{Length of Core Run}} \times 100$		<table border="0"> <tr> <td style="text-align: center;">63 REC</td> <td style="text-align: center;">BQ</td> <td style="text-align: center;">Inches</td> <td style="text-align: center;">1-7/16</td> </tr> <tr> <td style="text-align: center;">NQ</td> <td style="text-align: center;">NQ</td> <td></td> <td style="text-align: center;">1-7/8</td> </tr> <tr> <td style="text-align: center;">43 RQD</td> <td style="text-align: center;">HQ</td> <td></td> <td style="text-align: center;">2-1/2</td> </tr> </table>	63 REC	BQ	Inches	1-7/16	NQ	NQ		1-7/8	43 RQD	HQ		2-1/2
63 REC	BQ	Inches	1-7/16														
NQ	NQ		1-7/8														
43 RQD	HQ		2-1/2														

## SYMBOLS

KEY TO MATERIAL TYPES				SOIL PROPERTY SYMBOLS	
	Topsoil		High Plasticity Inorganic Silt or Clay	N:	Standard Penetration, BPF
	Asphalt		Organic Silts/Clays	M:	Moisture Content, %
	Crushed Limestone		Well-Graded Gravel	LL:	Liquid Limit, %
	Fill Material		Poorly-Graded Gravel	PI:	Plasticity Index, %
	Shot-rock Fill		Silty Gravel	Qp:	Pocket Penetrometer Value, TSF
	Low Plasticity Inorganic Silt		Clayey Gravel	Qu:	Unconfined Compressive Strength Estimated Qu, TSF
	High Plasticity Inorganic Silt		Well-Graded Sand	$\gamma_D$ :	Dry Unit Weight, PCF
	Low Plasticity Inorganic Clay		Poorly-Graded Sand	F:	Fines Content
	High Plasticity Inorganic Clay		Silty Sand	<b>SAMPLING SYMBOLS</b>	
	Low Plasticity Inorganic Silt or Clay		Clayey Sand		Undisturbed Sample
			Peat		No Sample Recovery
			Limestone		Split-Spoon Sample
			Sandstone		Water Level After Drilling
			Siltstone		Rock Core Sample
			Claystone		Auger or Bag Sample
			Weathered Rock		Extended Time Reading
			Dolomite		
			Granite		
			Gneiss		
			Schist		
			Amphibolite		
			Metagraywacke		
			Phyllite		



# TEST BORING RECORD

BORING NO: **B-1**

PROJECT: Capital Plaza		JOB NO: 1831-11-430	REPORT NO:
PROJECT LOCATION: Frankfort, KY			
ELEVATION: 495.1	BORING STARTED: 10/12/2011		BORING COMPLETED: 10/12/2011
DRILLING METHOD: 4" HSA	RIG TYPE: D-50		HAMMER: Auto
GROUNDWATER (ft): 24.0	BORING DIAMETER (IN): 4		SHEET 1 OF 1

Remarks:

Groundwater	ELEV. (FT.)	DEPTH (FT.)	MATERIAL DESCRIPTION	Lithology	Sample Type	Recovery (in)	RQD (%)	Qu	STANDARD PENETRATION RESISTANCE (N)					BLOWS /6"		
									0	10	20	30	40		50	
	497.4	0	Concrete - 8 inches													
	493.8	5	Gravel			16										3 - 4 - 5
	489.1	10	FILL - Lean Clay (CL) with gravel pieces, STIFF to FIRM, brown and gray, moist			0										5 - 4 - 4
		15	Lean Clay (CL) silty, FIRM to SOFT, brown and gray, moist			18										3 - 2 - 4
		20	Lean Clay (CL) silty, FIRM to SOFT, brown and gray, moist			18										2 - 1 - 3
	480.1	25	Silt (ML) clayey, FIRM, gray, moist			18										2 - 2 - 2
		30	Silt (ML) clayey, FIRM, gray, moist			18										2 - 2 - 3
	472.1	35	Sand (SM) silty, fine grained, FIRM, gray, wet			18										3 - 3 - 3
		40	Sand (SM) silty, fine grained, FIRM, gray, wet			18										6 - 6 - 7
		45	Sand (SM) silty, fine grained, FIRM, gray, wet			18										6 - 7 - 6
	453.1	50	Sand and gravel, coarse grained, DENSE to VERY DENSE, assorted colors, wet			18										6 - 6 - 7
		55	Sand and gravel, coarse grained, DENSE to VERY DENSE, assorted colors, wet			18										9 - 11 - 13
		60	Sand and gravel, coarse grained, DENSE to VERY DENSE, assorted colors, wet			18										21 - 26 - 21
		65	Sand and gravel, coarse grained, DENSE to VERY DENSE, assorted colors, wet			18										11 - 34 - 30
	433.5	70	Weathered Limestone			12										45 - 42 - 38
	433.1	75	Auger Refusal at 62.0 feet													
		80														
		85														

CRAIG2\_1831-11-430.GPJ\_QOR\_CORP.GDT\_12/8/11



# TEST BORING RECORD

BORING NO: **B-2**

PROJECT: Capital Plaza		JOB NO: 1831-11-430	REPORT NO:
PROJECT LOCATION: Frankfort, KY			
ELEVATION: 496.8	BORING STARTED: 10/11/2011		BORING COMPLETED: 10/11/2011
DRILLING METHOD: 4" HSA	RIG TYPE: D-50		HAMMER: Auto
GROUNDWATER (ft): 35.0	BORING DIAMETER (IN): 4		SHEET 1 OF 1

Remarks:

Groundwater	ELEV. (FT.)	DEPTH (FT.)	MATERIAL DESCRIPTION	Lithology	Sample Type	Recovery (in)	RQD (%)	Qu	STANDARD PENETRATION RESISTANCE (N)					BLOWS /6"	
									0	10	20	30	40		50
	496.8	0	Topsoil - 4 inches	[Cross-hatched]		12									4 - 5 - 6
		5	FILL - Lean Clay (CL) silty, STIFF, light brown, moist	[Diagonal lines]		14									12 - 18 - 19
	490.8	10	Lean Clay (CL) FIRM, brown and gray mottled, moist	[Diagonal lines]		16									7 - 6 - 7
		15				18									4 - 4 - 5
		20				18									4 - 4 - 3
	481.8	25	Silt (ML) clayey with little fine sand, SOFT, gray, moist to wet	[Vertical lines]		18									3 - 2 - 3
		30				18									2 - 3 - 2
		35				18									2 - 2 - 2
	461.8	40	Silt (ML) and fine sand, FIRM, gray, wet	[Vertical lines]		18									2 - 2 - 2
		45				18									3 - 4 - 4
		50				18									3 - 3 - 3
	451.8	55	Sand and Gravel, encountered cobbles below 60 feet	[Dotted]		12									4 - 4 - 50/0.3
		60				16									16 - 34 - 50/0.4
		65				16									20 - 9 - 14
		70				14									10 - 23 - 19
	430.7	75	Weathered Limestone	[Dotted]		16									21 - 27 - 42
		80	Auger Refusal at 66.4 feet												
		85													

CRAIG2\_1831-11-430.GPJ\_QOR\_CORP.GDT\_12/8/11



# TEST BORING RECORD

BORING NO: **B-3**

PROJECT: Capital Plaza		JOB NO: 1831-11-430	REPORT NO:
PROJECT LOCATION: Frankfort, KY			
ELEVATION: 498.7	BORING STARTED: 10/10/2011		BORING COMPLETED: 10/11/2011
DRILLING METHOD: 4" HSA	RIG TYPE: D-50		HAMMER: Auto
GROUNDWATER (ft): 25.0	BORING DIAMETER (IN): 4		SHEET 1 OF 1
Remarks:			

Groundwater	ELEV. (FT.)	DEPTH (FT.)	MATERIAL DESCRIPTION	Lithology	Sample Type	Recovery (in)	RQD (%)	Qu	STANDARD PENETRATION RESISTANCE (N)					BLOWS /6"	
									0	10	20	30	40		50
	498.3	0	Topsoil - 5 inches												3 - 5 - 7
		5	Lean Clay (CL) STIFF to FIRM, light brown and gray mottled, moist			16									9 - 11 - 11
		10				17									6 - 5 - 6
		15				16									5 - 5 - 5
		20	Silty Sand (SM) VERY LOOSE, brown and gray, moist			16									4 - 4 - 6
	480.7	25				18									3 - 2 - 4
		30	Silt (ML) clayey with fine sand, FIRM to SOFT, brown and gray, wet			18									1 - 2 - 1
	472.7	35				16									6 - 9 - 5
		40	Silty Lean Clay (CL-ML) SOFT to FIRM, gray, wet			18									5 - 5 - 3
	462.7	45				18									3 - 1 - 2
		50	Silty Gravel with Sand (GP-GM) coarse sand, VERY DENSE, assorted colors, wet			18									3 - 4 - 4
	451.7	55				18									3 - 4 - 3
		60				14									4 - 6 - 36
		65	Weathered Limestone			6									23 - 48 - 50/0.3
	434.7	70	Auger Refusal at 69.9 feet / Begin Coring			59/60	83	9,040 psi							22 - 24 - 30
	428.8	75	Limestone, light to medium gray, fine to medium grained, medium bedded with occasional very thin shale laminations			60/60	80	14,390 psi							45 - 50/0.4
	418.8	80	Coring Terminated at 79.9 feet												40 - 50/0.4
		85													

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# TEST BORING RECORD

BORING NO: **B-4**

PROJECT: Capital Plaza		JOB NO: 1831-11-430	REPORT NO:
PROJECT LOCATION: Frankfort, KY			
ELEVATION: 506.1	BORING STARTED: 10/12/2011		BORING COMPLETED: 10/12/2011
DRILLING METHOD: 4" HSA	RIG TYPE: D-50		HAMMER: Auto
GROUNDWATER (ft): 32.0	BORING DIAMETER (IN): 4		SHEET 1 OF 1

Remarks:

Groundwater	ELEV. (FT.)	DEPTH (FT.)	MATERIAL DESCRIPTION	Lithology	Sample Type	Recovery (in)	RQD (%)	Qu	STANDARD PENETRATION RESISTANCE (N)					BLOWS /6"	
									0	10	20	30	40		50
	506.8	0	Topsoil - 4 inches			14									4 - 7 - 9
		5	FILL - Lean Clay (CL) with few gravel pieces, few brick pieces, STIFF to SOFT, light brown, moist			16									3 - 9 - 11
		10				16									4 - 7 - 8
		15				14									6 - 3 - 4
	494.1	20	Lean Clay (CL) STIFF, light brown and gray, moist			4									2 - 1 - 2
		25				16									4 - 4 - 5
		30				18									4 - 5 - 8
	484.1	35	Silt (ML) clayey with little fine sand, VERY STIFF to STIFF, light brown and gray grading to gray, moist			18									6 - 8 - 9
		40				18									3 - 4 - 6
	474.1	45	Silty Sand (SM) very fine sand, FIRM, gray, wet			12									4 - 4 - 5
		50				16									4 - 6 - 8
		55				18									5 - 9 - 11
		60				18									6 - 9 - 12
	452.1	65	Sand with silt and few pea gravel pieces (SPG) DENSE to VERY DENSE, assorted colors, wet			16									9 - 9 - 14
		70				18									9 - 11 - 17
		75				16									22 - 24 - 23
		80				18									24 - 28 - 28
	430.4	85	Weathered Limestone			16									29 - 47 - 45
	430.1		Auger Refusal at 76.0 feet												

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# TEST BORING RECORD

BORING NO: **B-6**

PROJECT: Capital Plaza		JOB NO: 1831-11-430	REPORT NO:
PROJECT LOCATION: Frankfort, KY			
ELEVATION: 506.8	BORING STARTED: 10/6/2011		BORING COMPLETED: 10/6/2011
DRILLING METHOD: 4" HSA	RIG TYPE: B-80		HAMMER: Manual
GROUNDWATER (ft): 37.0		BORING DIAMETER (IN): 4	SHEET 1 OF 1
Remarks:			

Groundwater	ELEV. (FT.)	DEPTH (FT.)	MATERIAL DESCRIPTION	Lithology	Sample Type	Recovery (in)	RQD (%)	Qu	STANDARD PENETRATION RESISTANCE (N)					BLOWS /6"	
									0	10	20	30	40		50
	506.8	0	Topsoil - 3 inches			12									2 - 5 - 5
		5	FILL - Lean Clay (CL) STIFF to FIRM, light brown, moist			11									4 - 5 - 6
	499.8	5				13									3 - 3 - 5
		10	Lean Clay (CL) FIRM to STIFF, light brown to gray, moist			8									2 - 3 - 4
		10				10									3 - 2 - 3
		15				14									3 - 5 - 5
		20				17									3 - 3 - 4
		25				17									2 - 3 - 4
		30				6									3 - 5 - 6
	474.8	30	Silty Sand (SM) very fine sand, LOOSE, gray, wet			18									2 - 3 - 4
		35				18									1 - 1 - 2
		40				18									1 - 1 - 3
		45				18									woh - 1 - 6
		50				18									5 - 6 - 50/0.5
	451.4	55	Sand and Gravel (SPG) with few cobbles, DENSE, assorted colors, wet			17									6 - 48 - 28
	443.0	65	Weathered Limestone												
	442.9	65	Auger Refusal at 63.9 feet												
		70													
		75													
		80													
		85													

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# TEST BORING RECORD

BORING NO: **B-7**

PROJECT: Capital Plaza		JOB NO: 1831-11-430	REPORT NO:
PROJECT LOCATION: Frankfort, KY			
ELEVATION: 503.6	BORING STARTED: 10/7/2011		BORING COMPLETED: 10/7/2011
DRILLING METHOD: 4" HSA	RIG TYPE: B-80		HAMMER: Manual
GROUNDWATER (ft): 22.0		BORING DIAMETER (IN): 4	SHEET 1 OF 1

Remarks:

Groundwater	ELEV. (FT.)	DEPTH (FT.)	MATERIAL DESCRIPTION	Lithology	Sample Type	Recovery (in)	RQD (%)	Qu	STANDARD PENETRATION RESISTANCE (N)					BLOWS /6"	
									0	10	20	30	40		50
	503.9	0	Topsoil - 3 inches			5									3 - 5 - 5
		5	FILL - Lean Clay (CL) STIFF, light brown, moist			4									5 - 4 - 4
	496.6	10	Lean Clay (CL) VERY STIFF to STIFF, gray, moist			18									4 - 5 - 6
		16					16								5 - 7 - 7
		18					18								5 - 7 - 9
		15					18								4 - 7 - 12
		20				17									3 - 3 - 6
	479.7	25	Sandy Silt (SM) FIRM to SOFT,			18									2 - 2 - 3
		30					9								1 - 3 - 3
		35					18								1 - 2 - 2
		40					18								2 - 2 - 3
		45					18								1 - 1 - 3
	455.6	50	Sand and Gravel (SPG) with few cobbles, VERY FIRM to DENSE, assorted colors, wet			18									10 - 9 - 10
		55					18								11 - 10 - 18
	446.5	57.1	Auger Refusal at 57.1 feet												
		60													
		65													
		70													
		75													
		80													
		85													

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# TEST BORING RECORD

BORING NO: **B-8**

PROJECT: Capital Plaza		JOB NO: 1831-11-430	REPORT NO:
PROJECT LOCATION: Frankfort, KY			
ELEVATION: 502.6	BORING STARTED: 10/10/2011		BORING COMPLETED: 10/10/2011
DRILLING METHOD: 4" HSA	RIG TYPE: B-80		HAMMER: Manual
GROUNDWATER (ft): 32.0		BORING DIAMETER (IN): 4	SHEET 1 OF 1

Remarks:

Groundwater	ELEV. (FT.)	DEPTH (FT.)	MATERIAL DESCRIPTION	Lithology	Sample Type	Recovery (in)	RQD (%)	Qu	STANDARD PENETRATION RESISTANCE (N)					BLOWS /6"	
									0	10	20	30	40		50
	502.9	0	Topsoil - 5 inches			10									7 - 14 - 10
	496.6	5	FILL - Lean Clay (CL) with few gravel pieces, STIFF, brown, moist			18									5 - 7 - 8
		10	Lean Clay (CL) with little fine sand, VERY STIFF to STIFF, light brown, moist			18									4 - 5 - 8
		15				18									2 - 5 - 9
	484.6	20	Lean Clay (CL) silty with fine sand, STIFF to FIRM, gray, moist			18									3 - 5 - 5
		25				16									1 - 3 - 4
	476.6	30	Silty Sand (SM) increasing sand coarseness, gravel and cobbles with depth, LOOSE to VERY FIRM, assorted colors, moist to wet			4									1 - 1 - 2
		35				6									4 - 4 - 3
		40				12									5 - 6 - 6
		45				3									2 - 2 - 4
		50				12									6 - 12 - 6
		55				10									9 - 16 - 11
	447.3 445.7	56.9	Weathered Limestone Auger Refusal at 56.9 feet												6 - 9 - 50/0.3
		60													
		65													
		70													
		75													
		80													
		85													

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# TEST BORING RECORD

BORING NO: **B-9**

PROJECT: Capital Plaza		JOB NO: 1831-11-430	REPORT NO:
PROJECT LOCATION: Frankfort, KY			
ELEVATION: 502.2	BORING STARTED: 10/5/2011		BORING COMPLETED: 10/6/2011
DRILLING METHOD: 4" HSA	RIG TYPE: D-50		HAMMER: Auto
GROUNDWATER (ft): 34.0	BORING DIAMETER (IN): 4		SHEET 1 OF 1

Remarks:

Groundwater	ELEV. (FT.)	DEPTH (FT.)	MATERIAL DESCRIPTION	Lithology	Sample Type	Recovery (in)	RQD (%)	Qu	STANDARD PENETRATION RESISTANCE (N)					BLOWS /6"	
									0	10	20	30	40		50
	507.8	0	Topsoil - 4 inches			14									4 - 6 - 6
	496.2	5	FILL - Lean Clay (CL) with few gravel pieces, metal pieces, few shingle pieces, STIFF to VERY STIFF, brown, moist			4									3 - 8 - 10
		10	Lean Clay (CL) silty, VERY STIFF to STIFF, light brown, moist			3									14 - 12 - 13
		15				18									6 - 10 - 12
		20				18									5 - 6 - 8
	482.2	25	Silty Sand (SM) very fine sand, increasing coarseness with depth, LOOSE, gray, wet			8									4 - 6 - 6
		30				14									5 - 4 - 4
		35				18									3 - 4 - 4
	466.2	40	Sand (SP) medium grained, FIRM, gray, wet			18									2 - 3 - 3
		45				18									1 - 2 - 2
		50				18									5 - 8 - 12
		55				14									4 - 12 - 6
	446.2	60	Sand and Gravel, few cobbles, assorted colors, wet			16									6 - 6 - 3
		65				16									8 - 5 - 4
	440.2	70	Auger Refusal at 62.0 feet			18		7,880 psi							
		75	Limestone, light gray to white, very fine grained, medium to thickly bedded with very few very thin shale laminations			27/27	85	11,740 psi							
		80				60/60	100								
	430.2	85	Coring Terminated at 72.0 feet			32/32	100								

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# TEST BORING RECORD

BORING NO: **B-10**

PROJECT: Capital Plaza		JOB NO: 1831-11-430	REPORT NO:
PROJECT LOCATION: Frankfort, KY			
ELEVATION: 503.6	BORING STARTED: 10/7/2011		BORING COMPLETED: 10/7/2011
DRILLING METHOD: 4" HSA	RIG TYPE: B-80		HAMMER: Manual
GROUNDWATER (ft): 40.0		BORING DIAMETER (IN): 4	SHEET 1 OF 1
Remarks:			

Groundwater	ELEV. (FT.)	DEPTH (FT.)	MATERIAL DESCRIPTION	Lithology	Sample Type	Recovery (in)	RQD (%)	Qu	STANDARD PENETRATION RESISTANCE (N)					BLOWS /6"	
									0	10	20	30	40		50
	503.6	0	Topsoil - 2 inches			3									3 - 3 - 6
		5	Lean Clay (CL) silty, FIRM to STIFF, brown, moist			7									9 - 11 - 14
		10				18									5 - 6 - 8
		15				18									5 - 2 - 3
		20				18									2 - 2 - 4
		25	Silt (ML) with fine sand, clayey, FIRM to SOFT, gray, moist to wet			18									6 - 8 - 16
	479.1	30				18									3 - 5 - 7
		35				18									2 - 3 - 3
		40				18									1 - 2 - 2
	461.6	45	Silty Sand (SM) fine to medium grained, LOOSE, gray, wet			18									2 - 3 - 2
		50				18									1 - 1 - 2
		55				18									2 - 3 - 3
	446.6	60	Sand (SP) coarse grained with few gravel and cobble pieces, assorted colors, wet			18									2 - 4 - 6
		65													
		70													
	431.6	75	Auger Refusal at 72.0 feet												
		80													
		85													

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# TEST BORING RECORD

BORING NO: **B-11**

PROJECT: Capital Plaza		JOB NO: 1831-11-430	REPORT NO:
PROJECT LOCATION: Frankfort, KY			
ELEVATION: 500.3	BORING STARTED: 10/11/2011		BORING COMPLETED: 10/11/2011
DRILLING METHOD: 4" HSA	RIG TYPE: B-80		HAMMER: Manual
GROUNDWATER (ft): 18.0		BORING DIAMETER (IN): 4	SHEET 1 OF 1
Remarks:			

Groundwater	ELEV. (FT.)	DEPTH (FT.)	MATERIAL DESCRIPTION	Lithology	Sample Type	Recovery (in)	RQD (%)	Qu	STANDARD PENETRATION RESISTANCE (N)					BLOWS /6"	
									0	10	20	30	40		50
	500.3	0	Topsoil - 2 inches			8									3 - 3 - 4
		5	Lean Clay (CL) FIRM, brown, moist			10									4 - 5 - 6
		18				18									2 - 2 - 3
		18				18									3 - 3 - 4
		18				18									3 - 5 - 7
		18				18									3 - 5 - 6
	479.9	20	Silty Sand (SM) clayey, SOFT to VERY SOFT, gray, wet			18									3 - 4 - 6
		25				18									1 - 2 - 2
		30				18									1 - 1 - 1
		35				18									1 - 1 - 1
		40				16									wot - wot - wot
		45				18									1 - 1 - 2
		50				18									2 - 2 - 5
	444.3	55	Sand (SP-SM) silty, coarse grained with few gravel and cobble pieces, DENSE to VERY DENSE, assorted colors, wet			16									16 - 24 - 35
		60				14									34 - 37 - 50/0.5
	432.3 432.2	70	Weathered Limestone Auger Refusal at 68.1 feet												
		75													
		80													
		85													

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# TEST BORING RECORD

BORING NO: **B-12**

PROJECT: Capital Plaza		JOB NO: 1831-11-430	REPORT NO:
PROJECT LOCATION: Frankfort, KY			
ELEVATION: 500.8	BORING STARTED: 10/10/2011		BORING COMPLETED: 10/10/2011
DRILLING METHOD: 4" HSA	RIG TYPE: B-80		HAMMER: Manual
GROUNDWATER (ft): 28.0		BORING DIAMETER (IN): 4	SHEET 1 OF 1
Remarks:			

Groundwater	ELEV. (FT.)	DEPTH (FT.)	MATERIAL DESCRIPTION	Lithology	Sample Type	Recovery (in)	RQD (%)	Qu	STANDARD PENETRATION RESISTANCE (N)					BLOWS /6"			
									0	10	20	30	40		50		
	500.8	0	Concrete - 5 inches														
	499.7	5	Gravel - 8 inches			16											11 - 11 - 10
	496.8	5	FILL - Lean Clay (CL) with few gravel pieces, VERY STIFF, brown, moist			18											2 - 2 - 3
	493.8	10	Lean Clay (CL) FIRM, gray, moist			18											2 - 3 - 5
		10	Lean Clay (CL) STIFF to FIRM, light brown and gray mottled, moist			0											4 - 5 - 7
		15				16											5 - 5 - 6
		20				18											5 - 7 - 11
		25				16											1 - 2 - 4
	472.8	30	Silty Sand (SM) increasing sand with depth, LOOSE, gray, wet			18											1 - 2 - 2
		35				18											1 - 2 - 5
		40				18											2 - 2 - 3
		45				18											2 - 3 - 4
	449.3	50	Sand with gravel and few cobbles (SPG) FIRM to VERY DENSE, assorted colors, wet			14											wot - wot - 1
		55				16											wot - 4 - 12
		60				12											13 - 24 - 50/0.4
	434.9	65	Weathered Limestone			14											24 - 20 - 23
	434.1	70	Auger Refusal at 66.7 feet														
		75															
		80															
		85															

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# TEST BORING RECORD

BORING NO: **B-13**

PROJECT: Capital Plaza		JOB NO: 1831-11-430	REPORT NO:
PROJECT LOCATION: Frankfort, KY			
ELEVATION: 503.4	BORING STARTED: 10/7/2011		BORING COMPLETED: 10/7/2011
DRILLING METHOD: 4" HSA	RIG TYPE: D-50		HAMMER: Auto
GROUNDWATER (ft): 34.0		BORING DIAMETER (IN): 4	SHEET 1 OF 1

Remarks:

Groundwater	ELEV. (FT.)	DEPTH (FT.)	MATERIAL DESCRIPTION	Lithology	Sample Type	Recovery (in)	RQD (%)	Qu	STANDARD PENETRATION RESISTANCE (N)					BLOWS /6"	
									0	10	20	30	40		50
	503.4	0	Topsoil - 5 inches			6									4 - 5 - 5
		5	FILL - Lean Clay (CL) silty with few gravel pieces, STIFF to FIRM, brown, black and gray, moist			12									5 - 7 - 8
		10				10									3 - 4 - 4
	493.4	15				16									2 - 2 - 3
		20	Lean Clay (CL) STIFF, light brown and gray mottled, moist			18									2 - 2 - 4
		25				18									4 - 5 - 6
		30				16									3 - 4 - 6
	478.4	35	Silt (ML) with fine sand, FIRM, gray, moist to wet			18									4 - 4 - 4
		40				18									3 - 2 - 5
	469.4	45	Silty Sand (SM) fine grained, LOOSE to FIRM, gray, wet			16									4 - 4 - 5
		50				18									5 - 7 - 7
		55				18									6 - 6 - 8
	449.4	60	Sand (SP) coarse, FIRM to VERY FIRM, assorted colors, wet			18									3 - 5 - 8
		65				18									3 - 6 - 8
	441.4	70	Sand and Gravel (SPG) DENSE, assorted colors, wet			18									7 - 9 - 11
		75				18									12 - 22 - 27
	430.4	80	Weathered Limestone			18									11 - 17 - 28
	429.7	85	Auger Refusal at 73.7 feet												

CRAIG2 1831-11-430.GPJ\_QOR\_CORP.GDT 12/8/11



# TEST BORING RECORD

BORING NO: **B-14**

PROJECT: Capital Plaza		JOB NO: 1831-11-430	REPORT NO:
PROJECT LOCATION: Frankfort, KY			
ELEVATION: 499.7	BORING STARTED: 10/6/2011		BORING COMPLETED: 10/6/2011
DRILLING METHOD: 4" HSA	RIG TYPE: D-50		HAMMER: Auto
GROUNDWATER (ft): 35.0		BORING DIAMETER (IN): 4	SHEET 1 OF 1

Remarks:

Groundwater	ELEV. (FT.)	DEPTH (FT.)	MATERIAL DESCRIPTION	Lithology	Sample Type	Recovery (in)	RQD (%)	Qu	STANDARD PENETRATION RESISTANCE (N)					BLOWS /6"	
									0	10	20	30	40		50
	499.4	0	Topsoil - 4 inches			2									7 - 10 - 7
	494.7	5	FILL - Lean Clay (CL) silty with fine sand, FIRM, brown, moist			12									4 - 3 - 4
			Lean Clay (CL) STIFF, brown and gray mottled, moist			16									2 - 2 - 3
		10				18									4 - 5 - 6
		15				18									3 - 4 - 7
	479.7	20	Lean to Fat Clay (CL-CH) with little fine sand, FIRM, gray, moist			18									4 - 4 - 7
		25				18									4 - 4 - 5
		30				18									3 - 2 - 3
		35	Silt (ML) with fine sand, sand increasing with depth, STIFF, gray, wet			18									2 - 2 - 3
	464.7	40				18									5 - 3 - 4
		45				18									7 - 6 - 6
		50				18									6 - 7 - 7
		55				18									5 - 5 - 6
	442.7	60	Sand (SP) fine to coarse grained, few pea gravels, DENSE, assorted colors, wet			18									5 - 4 - 3
	435.7	65	Weathered Limestone			18									25 - 32 - 22
	430.2	70	Auger Refusal at 69.5 feet			60/60	92	6,775 psi							12 - 21 - 23
		75	Limestone, light gray to white, very fine grained, medium to thickly bedded with very few very thin shale laminations			60/60	92	7,560 psi							
	420.2	80	Coring Terminated at 79.5 feet												
		85													

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# TEST BORING RECORD

BORING NO: **B-15**

PROJECT: Capital Plaza		JOB NO: 1831-11-430	REPORT NO:
PROJECT LOCATION: Frankfort, KY			
ELEVATION: 498.6	BORING STARTED: 10/11/2011		BORING COMPLETED: 10/11/2011
DRILLING METHOD: 4" HSA	RIG TYPE: B-80		HAMMER: Manual
GROUNDWATER (ft): 34.0	BORING DIAMETER (IN): 4		SHEET 1 OF 1

Remarks:

Groundwater	ELEV. (FT.)	DEPTH (FT.)	MATERIAL DESCRIPTION	Lithology	Sample Type	Recovery (in)	RQD (%)	Qu	STANDARD PENETRATION RESISTANCE (N)					BLOWS /6"			
									0	10	20	30	40		50		
	498.6	0	Concrete - 6 inches														
	497.5	5	Gravel - 7 inches			12											3 - 5 - 7
		10	Lean Clay (CL) silty, STIFF to FIRM, brown, moist to wet			18											2 - 3 - 5
		15		12													4 - 4 - 5
		20		17													4 - 7 - 8
		25		18													4 - 6 - 8
	479.4	20	Silt (ML) with fine sand, SOFT to VERY SOFT, gray, wet			7											2 - 2 - 2
		25		18													0 - 1 - 1
		30		18													1 - 0 - 2
		35		18													1 - 2 - 2
	464.6	35	Silt (ML) with fine to medium sand, SOFT to FIRM, gray, wet			18											1 - 2 - 2
		40		18													woh - 1 - 1
		45		18													1 - 2 - 3
		50		18													wot - 5 - 10
	446.6	55	Sand (SPG) coarse with gravel and few cobbles, increasing with depth, DENSE to FIRM, assorted colors, wet			18											18 - 26 - 34
		60		18													7 - 24 - 15
		65		16													24 - 13 - 13
		70		12													23 - 31 - 50/0.4
	428.9 428.1	70	Weathered Limestone														
		75	Auger Refusal at 70.5 feet														
		80															
		85															

CRAIG2\_1831-11-430.GPJ\_QOR\_CORP.GDT\_12/8/11

## FIELD TESTING PROCEDURES

**Field Operations:** The general field procedures employed by QORE Property Sciences are summarized in ASTM D 420 which is entitled "Investigating and Sampling Soils and Rocks for Engineering Purposes." This recommended practice lists recognized methods for determining soil and rock distribution and ground water conditions. These methods include geophysical and in situ methods as well as borings.

Borings are drilled to obtain subsurface samples using one of several alternate techniques depending upon the subsurface conditions. These techniques are:

- a. Continuous 2-1/2 or 3-1/4 inch I.D. hollow stem augers;
- b. Wash borings using roller cone or drag bits (mud or water);
- c. Continuous flight augers (ASTM D 1425).

These drilling methods are not capable of penetrating through material designated as "refusal materials." Refusal, thus indicated, may result from hard cemented soil, soft weathered rock, coarse gravel or boulders, thin rock seams, or the upper surface of sound continuous rock. Core drilling procedures are required to determine the character and continuity of refusal materials.

The subsurface conditions encountered during drilling are reported on a field test boring record by a field engineer who is on site to direct the drilling operations and log the recovered samples. The record contains information concerning the boring method, samples attempted and recovered, indications of the presence of various materials such as coarse gravel, cobbles, etc., and observations between samples. Therefore, these boring records contain both factual and interpretive information. The field boring records are on file in our office.

The soil and rock samples plus the field boring records are reviewed by a geotechnical engineer. The engineer classifies the soils in general accordance with the procedures outlined in ASTM D 2488 and prepares the final boring records that are the basis for all evaluations and recommendations.

The final boring records represent our interpretation of the contents of the field records based on the results of the engineering examinations and tests of the field samples. These records depict subsurface conditions at the specific locations and at the particular time when drilled. Soil conditions at other locations may differ from conditions occurring at these boring locations. Also, the passage of time may result in a change in the subsurface soil and ground water conditions at these boring locations. The lines designating the interface between soil or refusal materials on the records and on profiles represent approximate boundaries. The transition between materials may be gradual. The final boring records are included with this report. The detailed data collection methods used during this study are discussed on the following pages.

**Soil Test Borings:** Soil test borings were made at the site at locations shown on the attached Boring Plan. Soil sampling and penetration testing were performed in accordance with ASTM D 1586.

The borings were made by mechanically twisting a 5-5/8" outer diameter auger into the soil. At regular intervals, the drilling tools were removed and samples obtained with a standard 1.4 inch I.D., 2 inch O.D., split tube sampler. The sampler was first seated 6 inches to penetrate any loose cuttings, then driven an additional foot with blows of a 140-pound hammer falling 30 inches. The number of hammer blows required to drive the sampler the final foot was recorded and is designated the "penetration resistance".

Representative portions of the samples, thus obtained, were placed in glass jars and transported to the laboratory. In the laboratory, the samples were examined to verify the driller's field classifications. Test Boring Records are attached which graphically show the soil descriptions and penetration resistances.

**Soil Auger Soundings:** Soil auger soundings were made at the site at the locations shown on the attached Boring Location Plan. The soundings were performed by mechanically twisting a steel auger into the soil. However, unlike the soil test borings, a smaller diameter solid stem auger was used and no split-spoon samples were obtained. The driller provided a general description of the soil encountered by observing the soils brought to the surface by the twisting auger. The auger was advanced until refusal materials were encountered and the refusal depth was noted by the driller. The auger is then withdrawn and the depths to water or caved materials are then measured and recorded by the driller.

Soil auger soundings provide a rapid, economical method of obtaining the approximate bedrock depth, groundwater depth, and general soil conditions at locations where detailed soil testing and sampling is not required.

**Water Level Readings:** Water table readings are normally taken in conjunction with borings and are recorded on the "Test Boring Records". These readings indicate the approximate location of the hydrostatic water table at the time of our field investigation. Where impervious soils are encountered (clayey soils) the amount of water seepage into the boring is small, and it is generally not possible to establish the location of the hydrostatic water table through water level readings. The ground water table may also be dependent upon the amount of precipitation at the site during a particular period of time. Fluctuations in the water table should be expected with variations in precipitation, surface run-off, evaporation and other factors.

The time of boring water level reported on the boring records is determined by field crews as the drilling tools are advanced. The time of boring water level is detected by changes in the drilling rate, soil samples obtained, etc. Additional water table readings are generally obtained at least 24 hours after the borings are completed. The time lag of at least 24 hours is used to permit stabilization of the ground water table which has been disrupted by the drilling operations. The readings are taken by dropping a weighted line down the boring or using an electrical probe to detect the water level surface. Occasionally the borings will cave-in, preventing water level readings from being obtained or trapping drilling water above the caved-in zone. The cave-in depth is also measured and recorded on the boring records.

## **APPENDIX C**

**SUMMARY OF LABORATORY TEST DATA**

**LABORATORY TESTING PROCEDURES**



422 Codell Drive, Lexington, KY 40509

### Laboratory Data Summary



**PROJECT NAME:** Capital Plaza

**PROJECT NUMBER:** 1831-11-430

**REPORT DATE:** 10/28/11

BORING NO.	SAMPLE DEPTH, FT.	SAMPLE TYPE	USCS	NATURAL MOISTURE CONTENT, %	ATTERBERG LIMITS			MAX. DRY DENSITY PCF / OPTIMUM MOISTURE %	UNIT WEIGHT, PCF		UNCONFINED COMPRESSIVE STRENGTH, PSI	MATERIAL FINER THAN NO. 200, %	SPECIFIC GRAVITY	CBR, %
					LL.	P.L.	P.I.		WET	DRY				
B-3	4.0 - 5.5	SPT	CL	17.7	36	21	15							
B-3	6.5 - 8.0	SPT		19.7										
B-3	9.0 - 10.5	SPT		22.0										
B-3	14.0 - 15.5	SPT	CL	23.7	36	20	16							
B-3	24.0 - 25.5	SPT	SM	24.5							15.8			
B-3	34.0 - 35.5	SPT	ML	27.6							60.3			
B-3	44.0 - 45.5	SPT	CL-ML	30.2							73.5			
B-3	59.0 - 60.5	SPT	GP-GM	9.5							8.9			
B-3	71.0 - 71.5	CORE		0.4					167.3	166.7	9,040			
B-3	76.0 - 76.5	CORE		0.5					167.4	166.5	14,390			
B-5	6.5 - 8.0	SPT	CL	22.6	35	21	14							
B-5	9.0 - 10.5	SPT		19.0										
B-5	14.0 - 15.5	SPT		20.6										
B-5	29.0 - 30.5	SPT	CL	37.4							95.5			
B-5	34.0 - 35.5	SPT	CL	32.7							90.1			





422 Codell Drive, Lexington, KY 40509

### Laboratory Data Summary



**PROJECT NAME:** *Capital Plaza*

**PROJECT NUMBER:** 1831-11-430

**REPORT DATE:** 10/28/11

BORING NO.	SAMPLE DEPTH, FT.	SAMPLE TYPE	USCS	NATURAL MOISTURE CONTENT, %	ATTERBERG LIMITS			MAX. DRY DENSITY PCF / OPTIMUM MOISTURE %	UNIT WEIGHT, PCF		UNCONFINED COMPRESSIVE STRENGTH, PSI	MATERIAL FINER THAN NO. 200, %	SPECIFIC GRAVITY	CBR, %
					LL.	P.L.	P.I.		WET	DRY				
B-14	6.5 - 8.0	SPT	CL	25.4	41	23	18							
B-14	9.0 - 10.5	SPT		19.9										
B-14	14.0 - 15.5	SPT		24.5										
B-14	19.0 - 20.5	SPT		34.5										
B-14	24.0 - 25.5	SPT	CH	38.4	51	27	24					92.2		
B-14	29.0 - 30.5	SPT	CL	37.0	36	22	14					93.1		
B-14	44.0 - 45.5	SPT	ML	40.0								74.5		
B-14	49.0 - 50.5	SPT	ML	36.8								81.7		
B-14	70.0 - 70.5	CORE		0.1					167.4	167.2	6,775			
B-14	75.5 - 76.0	CORE		0.2					167.3	166.9	7,560			
B-15	4.0 - 5.5	SPT	CL	23.2	37	20	17							
B-15	6.5 - 8.0	SPT		24.0										
B-15	29.0 - 30.5	SPT	ML	34.7								68.0		
B-15	44.0 - 45.5	SPT	ML	22.3								36.3		

## LABORATORY TESTING PROCEDURES

**Soil Classification:** Soil classifications provide a general guide to the engineering properties of various soil types and enable the engineer to apply past experience to current problems. In our investigations, samples obtained during drilling operations are examined in our laboratory and visually classified by an engineer. The soils are classified according to consistency (based on number of blows from standard penetration tests), color and texture. These classification descriptions are included on our "Test Boring Records."

The classification system discussed above is primarily qualitative and for detailed soil classification two laboratory tests are necessary: grain size tests and plasticity tests. Using these test results the soil can be classified according to the AASHTO or Unified Classification Systems (ASTM D 2487). Each of these classification systems and the in-place physical soil properties provides an index for estimating the soil's behavior. The soil classification and physical properties obtained are presented in this report.

**Compaction Tests:** Compaction tests are run on representative soil samples to determine the dry density obtained by a uniform compactive effort at varying moisture contents. The results of the test are used to determine the moisture content and unit weight desired in the field for similar soils. Proper field compaction is necessary to decrease future settlements, increase the shear strength of the soil and decrease the permeability of the soil.

The two most commonly used compaction tests are the Standard Proctor test and the Modified Proctor test. They are performed in accordance with ASTM D 698 and D 1557, respectively. Generally, the Standard Proctor compaction test is run on samples from building or parking areas where small compaction equipment is anticipated. The Modified compaction test is generally performed for heavy structures, highways, and other areas where large compaction equipment is expected. In both tests a representative soil sample is placed in a mold and compacted with a compaction hammer. Both tests have four alternate methods.

Test	Method	Hammer Wt./Fall	Mold Diam.	Run on Matl. Finer Than	No. of Layers	No. of Blows/Layer
Standard	A	5.5 lb./12"	4"	No. 4 sieve	3	25
D 698	B	5.5 lb./12"	4"	3/8" sieve	3	25
	C	5.5 lb./12"	6"	3/4" sieve	3	56

Test	Method	Hammer Wt./Fall	Mold Diam.	Run on Matl. Finer Than	No. of Layers	No. of Blows/Layer
Modified	A	10 lb./18"	4"	No. 4 sieve	5	25
D 1557	B	10 lb./18"	4"	3/8" sieve	5	25
	C	10 lb./18"	6"	3/4" sieve	5	56

The moisture content and unit weight of each compacted sample is determined. Usually 4 to 5 such tests are run at different moisture contents. Test results are presented in the form of a dry unit weight versus moisture content curve. The compaction method used and any deviations from the recommended procedures are noted in this report.

**Atterberg Limits:** Portions of the samples are taken for Atterberg Limits testing to determine the plasticity characteristics of the soil. The plasticity index (PI) is the range of moisture content over which the soil deforms as a plastic material. It is bracketed by the liquid limit (LL) and the plastic limit (PL). The liquid limit is the moisture content at which the soil becomes sufficiently "wet" to flow as a heavy viscous fluid. The plastic limit is the lowest moisture content at which the soil is sufficiently plastic to be manually rolled into tiny threads. The liquid limit and plastic limit are determined in accordance with ASTM D 4318.

**Moisture Content:** The Moisture Content is determined according to ASTM D 2216.

## **APPENDIX D**

**ACI 302.1R-96**

**GUIDE FOR CONCRETE FLOOR AND SLAB CONSTRUCTION**

***ADDENDUM***  
**GUIDE FOR CONCRETE FLOOR AND SLAB CONSTRUCTION**  
**(302.1R-96)**  
Vapor Retarder Location

The report of ACI Committee 302, "Guide for Concrete Floor and Slab Construction (ACI 302.1R-96)" states in section 4.1.5 that "if a vapor barrier or retarder is required due to local conditions, these products should be placed under a minimum of 4 in. (100 mm) of trimable, compactible, granular fill (not sand)." ACI Committee 302 on Construction of Concrete Floors, and Committee 360 on Design of Slabs on Ground have found examples where this approach may have contributed to floor covering problems.

Based on the review of the details of problem installations, it became clear that the fill course above the vapor retarder can take on water from rain, wet-curing, wet-grinding or cutting, and cleaning. Unable to drain, the wet or saturated fill provides an additional source of water that contributes to moisture-vapor emission rates from the slab well in excess of the 3 to 5 lb/1000 ft<sup>2</sup>/24 h (1.46 to 2.44 kg/100 m<sup>2</sup>/24 h) recommendation of the floor covering manufacturers.

As a result of these experiences, and the difficulty in adequately protecting the fill course from water during the construction process, caution is advised on the use of the granular fill layer when moisture-sensitive finishes are to be applied to the slab surface.

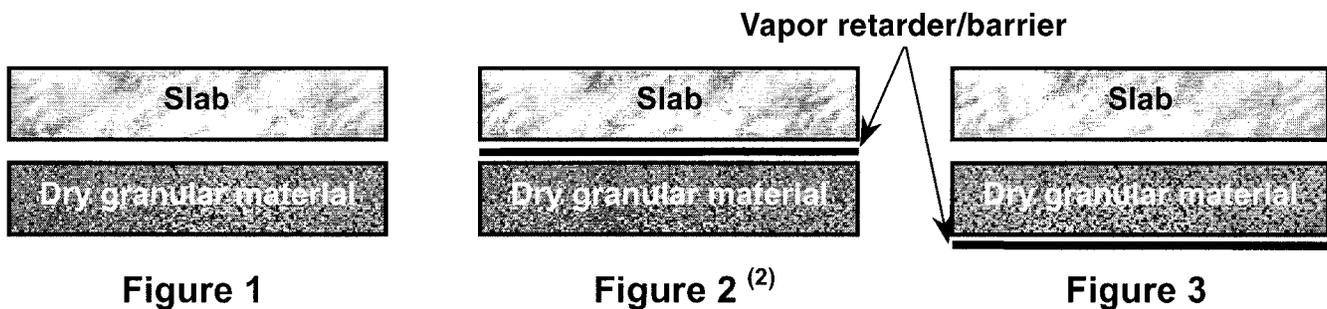
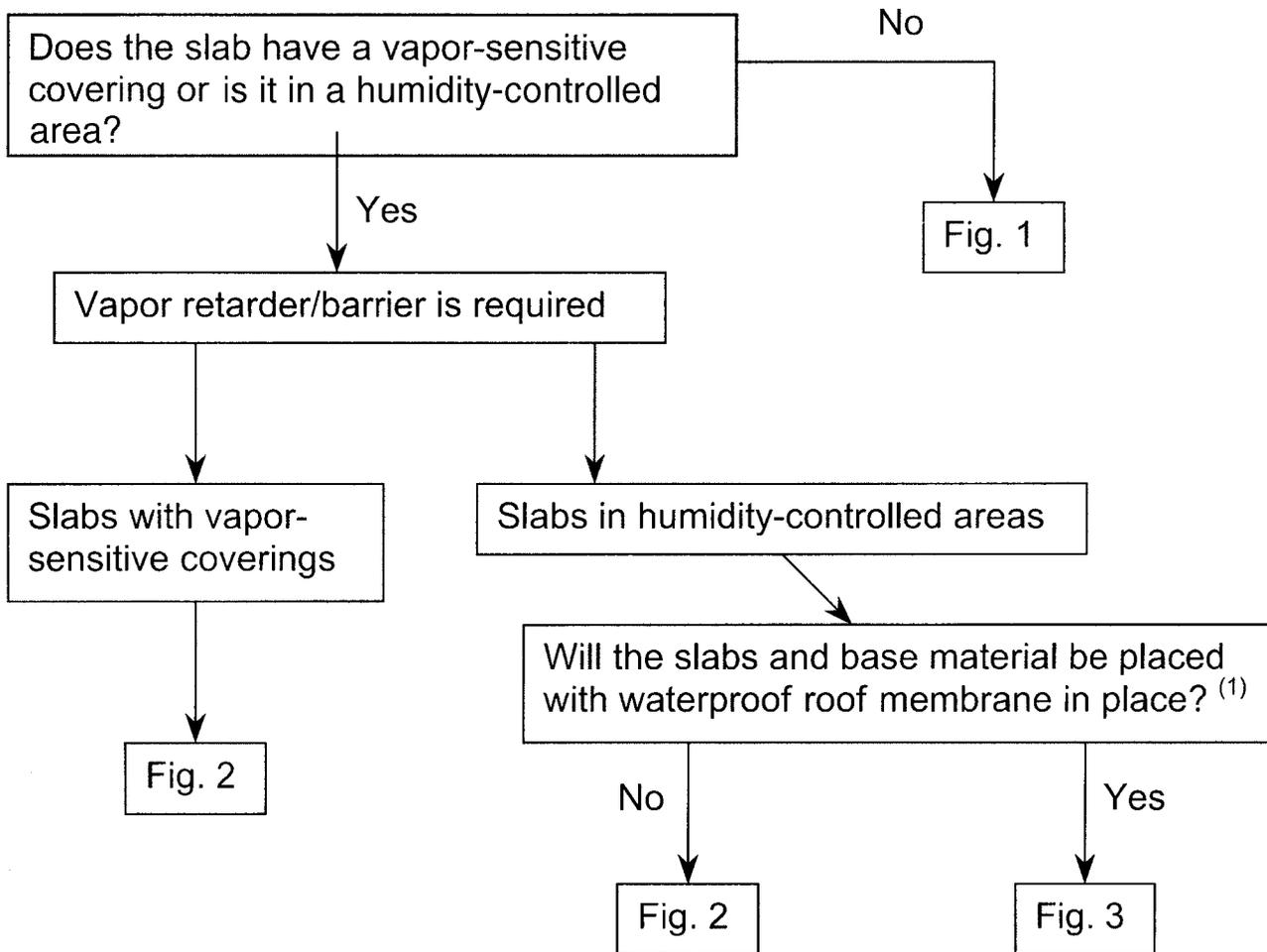
The committees believe that when the use of a vapor retarder or barrier is required, the decision whether to locate the retarder or barrier in direct contact with the slab or beneath a layer of granular fill should be made on a case-by-case basis.

Each proposed installation should be independently evaluated by considering the moisture sensitivity of subsequent floor finishes, anticipated project conditions and the potential effects of slab curling and cracking.

The following chart can be used to assist in deciding where to place the vapor retarder. The anticipated benefits and risks associated with the specified location of the vapor retarder should be reviewed with all appropriate parties before construction.

**ADDENDUM**  
**GUIDE FOR CONCRETE FLOOR AND SLAB CONSTRUCTION**  
**(302.1R-96)**

**Flow Chart for Location of Vapor Retarder/Barrier**



- (1) If granular material is subject to future moisture infiltration, use Fig. 2
- (2) If Fig. 2 is used, reduced joint spacing, a concrete with low shrinkage potential, or other measures to minimize slab curling will likely be required.