REPORT OF GEOTECHNICAL EXPLORATION

AMERICAN ENGINEERS, INC.

JANUARY 2021

HMB PROFESSIONAL ENGINEERS INC.

GRANT COUNTY EQUALIZATION BASIN

CRITTENDEN, KENTUCKY

AMERICAN ENGINEERS, INC.

January 26, 2021

Mr. Benton Hanson, PE
HMB Professional Engineers, Inc.
3 HMB Circle
Frankfort, KY 40601

Re: Report of Geotechnical Exploration
Grant County Equalization Basin
Crittenden, Kentucky
AEI Project No. 220-308

Dear Mr. Hanson:

American Engineers, Inc. is pleased to submit this geotechnical report that details the results of our geotechnical exploration performed at the above referenced site.

The attached report describes the site and subsurface conditions and also details our recommendations for the proposed project. The Appendices to the report contains a drawing with a boring layout, typed boring logs and the results of laboratory testing.

We appreciate the opportunity to be of service to you on this project and hope to provide further support on this and other projects in the future. Please contact us if you have any questions regarding this report.

Respectfully,
AMERICAN ENGINEERS, INC.

Katy Bridges

Katy Bridges, EIT
Geotechnical Engineer

Dennis Mitchell

Dennis Mitchell, PE, PMP
Director of Federal
Geotechnical Services

REPORT OF GEOTECHNICAL EXPLORATION
GRANT COUNTY EQUALIZATION BASIN
CRITTENDEN, KENTUCKY

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REPORT OF GEOTECHNICAL EXPLORATION
GRANT COUNTY EQUALIZATION BASIN
CRITTENDEN, KENTUCKY

1 GENERAL SITE DESCRIPTION

The proposed site is located at the existing wastewater treatment plant at the end of Clairborne Drive, in Grant County, Kentucky. The estimated topographic relief over the site is 25 feet. At the time of the investigation, the site was covered in short mixed grasses and a few trees. A gravel driveway and a chain link fence are also within the footprint of the proposed equalization basin.

It is our understanding that a 200,000-gallon equalization basin is to be installed and tied into the existing gravity fed line connected to the comminutor pit. It is estimated that approximately 10 to 15 feet of cut may be required to achieve the bottom elevation of the proposed structure. Structure loads were unknown at the time of this investigation.

2 GENERAL SITE GEOLOGY

A review of available geologic mapping for the area (*Geology of the Walton Quadrangle, north-central Kentucky*, KGS, 2006) indicates that the site is underlain by Middle Ordovician-aged bedrock of the Kope Formation. The Kope Formation is composed primarily of interbedded shale and limestone. The shale is described as medium gray, greenish gray and light olive gray in color, commonly calcareous, silty, and weathers and slumps readily. The limestone is described as medium gray to light gray in color and coarse to fine grained.

Much of the middle and lower parts of the Kope Formation consists of soft, easily deformed shale, which is unstable and subject to slumping when wet. Over-steepened banks and artificial cuts should be avoided or be properly designed and drained. No other geologic hazards were readily apparent either during the investigation or upon review of available geologic mapping. It should be understood by the owner that it is impossible to fully identify the presence of all geologic hazards or the potential thereof during the course of a typical geotechnical investigation.

3 SCOPE OF WORK

The geotechnical exploration consisted of drilling four soil test borings and two rockline soundings. The soil test borings were advanced to auger refusal. Borings B-1, B-2 and B-3 were advanced until approximately ten feet of rock core was obtained. Borings B-1 and B-2 were drilled within the existing gravel drive adjacent to the proposed addition. Borings B-3 and B-4 were drilled across the site at a potential secondary location. Soundings S-1 and S-2 were drilled within the footprint of the proposed addition.

The boring was advanced by an AEI drill crew using a CME 850XR drill rig equipped with continuous flight hollow-stem augers and NQ2 sized coring equipment. Split-spoon samples were obtained at two and a half foot centers throughout the soil test boring. In addition, one Shelby tube sample was obtained in each of the borings except for Boring B-4. A Geotechnical Engineer was on-site throughout the investigation to log the recovered soil and rock samples, with particular attention given to soil type, color, relative moisture content, primary constituents and soil strength consistencies. Recovered samples were returned to the laboratory for additional classification and testing activities.

The natural moisture content of the soil samples was determined in the laboratory. The natural moisture content is denoted as (W%) and shown as a percentage of the dry weight of the soil on the boring logs. In addition, Atterberg limits and soil unconfined compressive strength tests were performed on samples representative of the predominant soil horizons. Slake durability index (SDI) testing was performed on representative rock core samples. The results of the laboratory tests are summarized in Appendix C.

The soils were classified in the laboratory in general accordance with the Unified Soil Classification System (USCS). The Unified symbol for each stratum is shown on the legend for the typed boring logs. The testing was performed in accordance with the generally accepted standards for such tests.

4 RESULTS OF THE EXPLORATION

4.1 *General*

Information provided in the Appendices for this report includes a boring layout, typed boring logs, results of the laboratory tests and other relevant geotechnical information. A description of the subsurface soil, bedrock and groundwater conditions follows.

4.2 *Subsurface Soil Conditions*

The generalized subsurface conditions encountered at the boring locations, including descriptions of the various strata and their depths and thicknesses are presented on the typed boring logs in Appendix B.

Gravel was encountered at the surface in Borings B-1, B-2 and B-3 with thicknesses ranging from three to four inches. Topsoil was encountered beneath the aggregate in Borings B-1 and B-2 with thicknesses of 20 and 21 inches. In Borings B-3 and B-4, silty lean clay was encountered directly beneath the crushed aggregate with a thickness of approximately 14 inches. The silty lean clay was described as light brown in color, moist of the anticipated optimum moisture content for compaction, and stiff in soil strength consistency. Beneath the surface materials, low to moderate plasticity clays were encountered. The lean clay was described as brown to gray in color, moist to wet of the anticipated optimum moisture for compaction and medium stiff to very stiff in soil strength consistency. In Boring B-2, a layer of gravel was encountered beneath the lean clay. The gravel is described as poorly graded, gray to white in color, and very dense. Beneath the lean clay and gravel, shale was encountered to auger refusal in Borings B-2, B-3 and B-4. The shale is described as tan to gray in color and highly weathered.

SPT-N values ranged from seven to 29 blows per foot (bpf) (excluding blow counts exceeding 50) with corresponding pocket penetrometer (Qp) values ranging from 1.5 to greater than 4.5 tons per square foot (tsf). Together, the SPT-N and Qp values are indicative of medium stiff to very stiff soil strength consistencies.

Atterberg limits testing was performed on samples representative of the predominant soil horizons. The results indicate that the clay soils classify as CL (Clay of Low plasticity), lean clay in accordance with the USCS. Liquid limit test results yielded values ranging from 40 to 44 percent with corresponding plasticity indices ranging from 17 to 25 percent, respectively. Natural moisture contents of the clay soils ranged from about 16 to 30 percent. Results of Atterberg limits and moisture content testing indicate that clays are typically at a moisture content within five percent wet or dry of the plastic limit. The results of laboratory testing are included in Appendix C.

Unconfined compressive strength testing was performed on selected samples representative of the predominant soil horizons. Unconfined compressive strength, or Q_u results ranged from about 4,322 to 9,242 pounds per square foot (psf) with corresponding dry densities ranging from 112.4 to 118.7 pounds per cubic foot, respectively. Unconfined compressive strength test results are summarized in Table 1 below.

Table 1: Unconfined Testing Results

Boring ID	Sample Depth (feet)	Dry Density (pcf)	Unconfined Compressive Strength (psf)
B-1	4.0-5.7	114.3	4,322
B-2	3.0-5.0	112.4	5,540
B-3	4.0-6.0	118.7	9,242

4.3 *Bedrock Conditions*

Refusal, as would be indicated by the Driller on the field boring logs, indicates a depth where either essentially no downward progress can be made by the auger or where the N-value indicates essentially no penetration of the split-spoon sampler. It is normally indicative of a very hard or very dense material such as large boulders or the upper bedrock surface. Refusal was encountered in each soil test boring and rockline sounding. Auger refusal depths are given in Table 2 below.

Table 2: Auger Refusal Depths

Boring ID	Auger Refusal Depth (feet)	Elevation (feet)
B-1	5.7	775.8
B-2	10.2	765.7
B-3	8.0	767.4
B-4	5.2	781.7
S-1	2.1	761.4
S-2	2.5	761.1

The recovered rock core was typically described as shale, interbedded with limestone, brown to gray in color, soft to moderately hard and moderately to highly weathered. Rock core recovery percentages representative of the bedrock encountered ranged from 61 to 100 percent for all coring intervals, with Rock Quality Designation (RQD) percentages ranging from zero to 90 percent. The RQD percentages are representative of very poor to good quality rock.

Slake durability index (SDI) testing was performed on selected samples representative of the predominant bedrock horizons and revealed that the shale classifies as Class III Non-Durable rock. Individual testing results are given in Table 3 below.

Table 3: Slake Durability Index Testing

Boring ID	Sample Depth (feet)	Slake Durability Index	Classification
B-1	9.0	9	Class III / Non-Durable
B-2	12.6	8	Class III / Non-Durable
B-2	17.6	35	Class III / Non-Durable
B-3	9.0	28	Class III / Non-Durable

4.4 *Groundwater Conditions*

Groundwater was not encountered in the borings at the site during the investigation. However, while on-site, groundwater seepage along the slope was observed. It is anticipated that groundwater is seeping along the soil / bedrock interface and is daylighting on the slope face. A long time is required for hydrostatic groundwater levels to come to equilibrium in boreholes. The short-term groundwater levels reported by the drill crew are not generally indicative of the long-term groundwater level. To accurately determine the long-term groundwater level, as well as the seasonal and precipitation induced fluctuations of the groundwater level, it is necessary to install piezometers in the boring, and monitor them for an extended length of time. Frequently, groundwater conditions affecting construction in this region are caused by trapped or perched groundwater, which occurs within the soil materials or at the soil/rock interface in irregular, discontinuous locations. If these water bodies are encountered during excavation, they can produce seepage durations and rates that will vary depending on the recent rainfall activity and the hydraulic conductivity of the material.

4.5 *Seismic Conditions*

According to the International Building Code, 2012 Edition, and the subsurface conditions encountered in the borings, Site Class B may be used for any seismic structural design for structures bearing on bedrock.

Soil liquefaction analysis was outside the scope of this investigation. Prior studies on similar soil types indicate that the potential for liquefaction is low and is primarily dependent on the variability of site soils and earthquake severity.

Consideration for seismic loading and liquefaction potential beyond this level of investigation is left to the discretion of the structural and foundation design engineer.

5 ANALYSES AND RECOMMENDATIONS

The recommendations that follow are based on project information provided to AEI during the course of this investigation. Should the project parameters change, please notify us so that our recommendations can be reviewed and modified as necessary.

5.1 *Slope Stability*

Due to the significant cut and the potential instability of exposed soil and rock from the Kope Formation when wet, a soil nail wall with a reinforced shotcrete face should be constructed immediately following or concurrent with excavation of the eastern and a portion of the northern soil face. The soil nail wall should be designed by a licensed geo-professional and should be offset from the concrete wall of the equalization basin to allow for placement of a perforated pipe underdrain between the soil nail wall and the wall of the basin. The soil nail wall should be designed to withstand the lateral earth pressures imposed on the wall.

5.2 *General Site Work*

5.2.1 Topsoil stripping

Prior to earthwork operations, topsoil and surface plant material root mat should be stripped from both cut and fill areas.

5.2.2 Rock Removal

A grading plan with finished floor elevations was not provided at the time of this investigation. **However, it is anticipated that rock removal may be required to achieve**

the bottom elevation of the basin. Excavations which extend below the bedrock surface can be excavated vertically. **Prolonged exposure should be prevented due to the potential of the Kope Formation to degrade and become unstable when wet.** Rock removal in these areas as described above should be performed by hoe-ramming or a trackhoe.

5.2.3 Mud Mat Construction

Due to the potential for the Kope Formation to degrade and become unstable when wet, the bedrock surface should be sealed with a mud-mat / lean concrete. The mud mat should be placed directly on the bedrock surface regardless of the excavation slope. Once sealed, granular backfill or a combination of granular backfill / lean concrete can then be placed to achieve the proposed bottom elevation of the basin.

5.2.4 Basin Backfill Material

The near-surface soils on this site are low to moderately plastic clays that classify as lean clay (CL) in accordance with the USCS. These soils exhibit low potential to swell or shrink when exposed to long-term increases or decreases in moisture content. **These soils are suitable for use as fill material outside the basin provided they are wetted or dried to a moisture content suitable for compaction.**

Backfill above the sealed bedrock surface of the basin, as well as around the perimeter walls, should consist of free-draining crushed stone such as KYTC No. 57 or equivalent. To provide a seal at the surface, on-site clay soils may be utilized within the upper two to three feet to minimize surface water infiltration around the basin.

5.2.5 Fill Placement

Lean clay, CL, soil placed outside structure areas should be placed in maximum eight inch (loose thickness) horizontal lifts, with each lift being compacted to a minimum of 95 percent of the standard Proctor maximum dry density, at a moisture content from plus/ minus two percent of optimum. The compaction requirement may be reduced to 92 percent in proposed landscape areas. Based on the results of moisture content tests performed in the borings, drying of the on-site soils may be necessary to achieve moisture contents suitable for compaction. Representative and adequate field density testing should be performed by AEI to verify that compaction requirements have been met.

5.2.6 Soil Movement

Site grading should be maintained during construction so that positive drainage is promoted at all times. Final site grading should be accomplished in such a manner as to divert surface runoff away from the foundation elements. Precipitation runoff should be collected in storm sewers as quickly as possible. The soils at the site may be considered erodible and should be stabilized as soon as practical upon completion of each phase of construction.

The weathered shale encountered at the site is highly susceptible to deterioration once exposed. Special care should be taken to limit the exposure of the weathered shale.

5.2.7 Site Soil Practices

Working with the on-site soils will demand sensible construction practices and techniques. Some of these include:

- Prevent stripping too far in advance of actual earthwork needs. Problems arise when broad areas of clay/silt mixtures are exposed and allowed to become wet and soft from rainfall. Once saturated, deep rutting can occur by movement of construction equipment. **Shale bedrock belonging to the Kope Formation was encountered on-site and is highly unstable and susceptible to slumping when exposed. Special care should be taken to limit the exposure of the shale.**

- Strip areas to receive fill in small, sequential areas as needed. These areas should be limited to the contractor's abilities to reasonably place and compact fill material.

- Schedule earthwork construction to take full advantage of a summer season. Generally, the on-site soils need to be placed within two percent or less of optimum moisture content to achieve compaction and reduce the potential for subgrade volume change. This moisture range is difficult to achieve in the winter and early spring when rainfall activity is more prevalent and soil drying is not always possible.

- Maintain good surface drainage during earthwork construction. Grade construction areas on a daily basis if necessary to promote sheet drainage of precipitation and seal all engineered fill placed with a smooth drum steel roller at the end of each day.

- Perform frequent density tests during fill placement to confirm achievement of proper compaction.

5.3 Structure Foundations

5.3.1 Recommended Bearing Capacity Values

To minimize the potential for differential settlement of the basin and to provide the necessary global stability of the basin, a mat foundation is recommended.

The structure should be designed to bear on crushed aggregate or lean concrete overlying the bedrock surface. An allowable bearing capacity of **five** kips per square foot (ksf) is recommended for design of mat foundation elements bearing on the lean concrete mud-mat or crushed aggregate above the mud-mat.

These recommendations are provided in consideration of the field-testing, laboratory testing, local codes, and our experience with materials of similar description.

5.3.2 Lateral Earth Pressures

Bearing elevations were unknown at the time of this investigation. However, it is anticipated that 10 to 15 feet of cut will be required to achieve the bottom elevation of the basin.

Lateral earth pressures were calculated utilizing the Rankine earth pressure theory. Earth pressure coefficients from Table 3 should be used to determine the lateral earth pressures acting on the walls. For the portion of the basin wall adjacent to the soil nail wall, the earth pressures can be calculated utilizing the theory proposed by Spangler & Handy for fascia walls adjacent to a stable rock face. Using this theory, the lateral pressure on the fascia wall is a function of the weight of the soil between the walls and the coefficient of friction between the soil and two walls. Earth pressure coefficients from Table 4 were used to determine the lateral earth pressures acting on the walls.

Table 3: Soil Nail Wall Lateral Earth Pressure Coefficients

Lateral Earth Pressure Coefficients	Lean Clay
Equiv. Fluid Pressure (Above Water Table)	43 pcf
Equiv. Fluid Pressure (Below Water Table)	90 pcf
Active Coefficient	0.36
At Rest Coefficient	0.53
Passive Coefficient	2.77
Friction Angle	28°
Assumed Unit Wt.	117.5 pcf
Submerged Unit Wt.	140 pcf

Table 3: Basin Wall Lateral Earth Pressure Coefficients

Lateral Earth Pressure Coefficients	Granular Fill Between Soil Nail wall and Basin Wall
Equiv. Fluid Pressure (Above Slip Line 0’-8’)	43 pcf
Equiv. Fluid Pressure (Below Slip Line 8’-15’)	10 pcf
Active Coefficient	0.36
Friction Angle	38°
Assumed Unit Wt.	110 pcf

Note: Equivalent Fluid Pressure below the water table does not account for hydrostatic pressures acting on the wall.

The design of below grade walls should also include perforated pipe foundation drains to prevent hydrostatic pressures behind the wall. Specifically, perforated pipe foundation drains should be placed between the soil nail wall and basin wall to promote drainage.

5.3.3 Excavation Safety

Temporary excavations should be properly sloped in accordance with the Kentucky Occupational Safety and Health Standards for the Construction Industry 29 CFR Part 1926, Subpart P — Excavations. The soil overburden at the site consists of Type B soils. Type B soils can be laid back on temporary slopes not exceeding 1 Horizontal: 1 Vertical (1H:1V) in excavations not exceeding 20 feet in depth. Sloping or benching for excavations greater than 20 feet deep should be designed by a registered professional engineer.

If significant construction vibrations are anticipated adjacent to the slopes or if the slope is to be exposed for an extended period of time, slopes flatter than 1H: 1V may be required.

5.3.4 Footing Trenches

We recommend that the bottom of mat foundations extend a minimum of 24 inches below finished exterior grade to provide protection against frost penetration related problems in normal winters. Interior foundations not exposed to severe drying, freezing temperatures, and/or severe moisture fluctuations can be constructed at relatively shallow depths as appropriate for construction. Foundation construction should follow these recommendations:

- Foundation concrete should be placed in the excavations the same day the trenches are cut.
- **Exposed bearing surfaces should be protected from severe drying, freezing, and water accumulation. A concrete "mud-mat" shall be constructed over the bedrock to minimize degradation and instability of the Kope Formation.**

- Any loose soil, debris, or excess water should be removed from the bearing surface by hand cleaning prior to concrete placement.

- The foundation-bearing surface should be level or appropriately benched.
- Foundation materials that have deteriorated as a result of the elements should be removed prior to concrete placement.

- Foundation trenches should be "clean-cut" where possible and constructed without the use of forms.

- Reinforcing steel should be placed in all footings to provide strength to distribute loads on the foundation that may be overlying weak or more compressible foundation materials to stronger adjacent materials.

5.3.5 Acceptance of Foundation Bearing Surfaces

Prior to placement of reinforcing steel in spread or continuous footings, an AEI Engineer or Engineering Technician should review the bearing surface to verify that the design bearing capacity provided can be achieved. The footings should also be reviewed to verify that the bottom is level and free of mud, loose soil or other questionable material that might affect foundation support.

5.3.6 Potential Foundation Movement

A detailed settlement analysis was beyond the scope of this investigation. However, it is anticipated that less than ½ inch of total settlement will occur for mat foundations bearing on bedrock. Differential settlement is expected to be less than ¼ inch.

These estimates assume that the foundations are designed and constructed according to the recommendations in this report and in conjunction with sound foundation construction practice.

5.4 General Considerations

5.4.1 Construction Monitoring and Testing

Site problems can be avoided or reduced if proper field observation and testing services are provided. We recommend all foundation excavations, proof rolling, and site and subgrade preparation be monitored by AEI. Density tests should be performed to verify compaction and moisture content for all earthwork operations. Field observations should be performed prior to and during concrete placement operations.

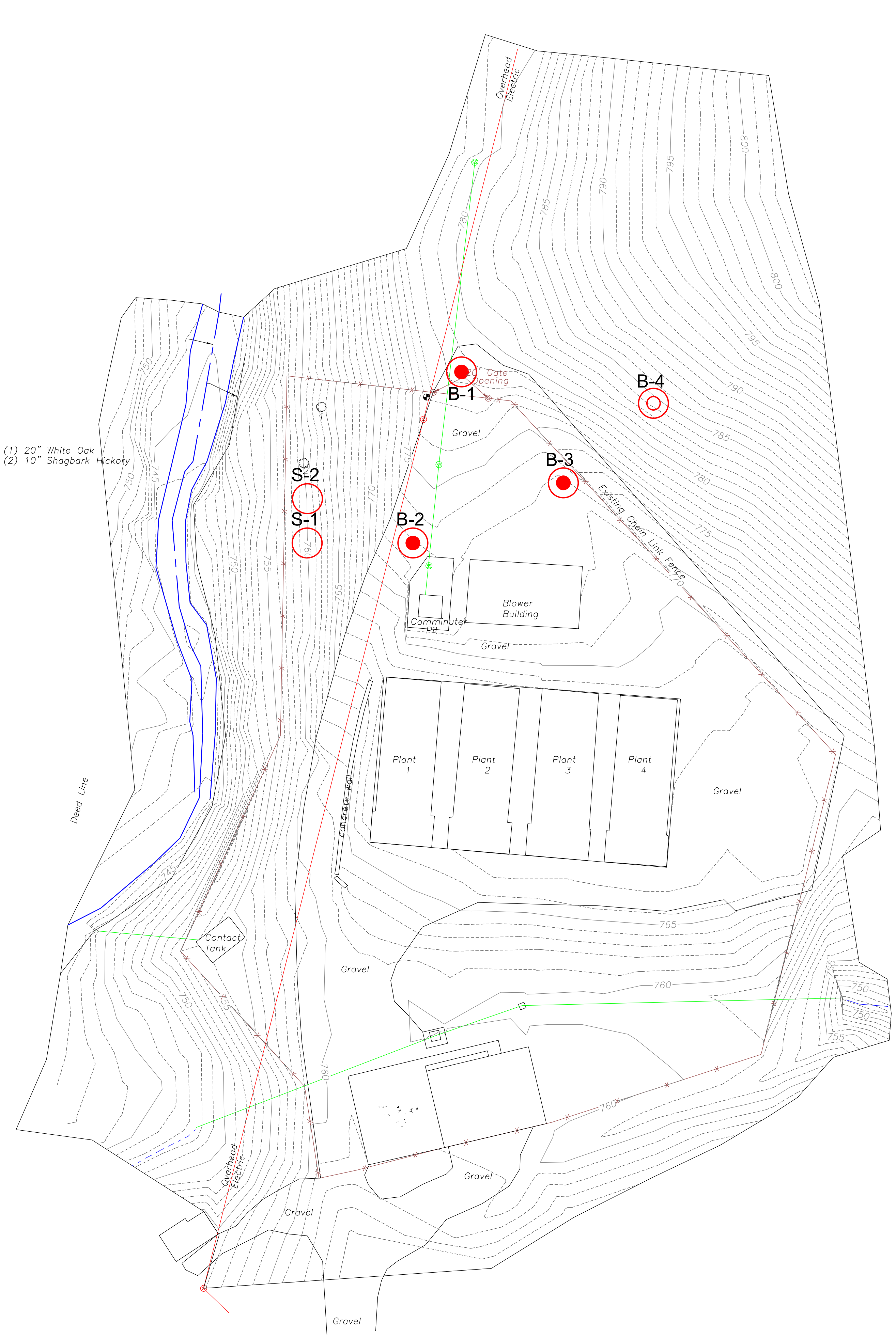
5.4.2 Limitations

The conclusions and recommendations presented herein are based on information gathered from the borings advanced during this exploration using the degree of care and skill ordinarily exercised under similar circumstances by competent members of the engineering profession. No warranties can be made regarding the continuity of conditions between the borings.

We will retain samples acquired for this project for a period of 30 days subsequent to the submittal date printed on the cover of this report. After this period, the samples will be discarded unless otherwise requested.

APPENDIX A

Boring Layout



LEGEND

- SOIL TEST BORING WITH STANDARD PENETRATION TESTS
- SOIL TEST BORING WITH STANDARD PENETRATION TESTS, UNDISTURBED SHELBY TUBES AND ROCK CORE
- MANUAL ROCKLINE SOUNDING

0' 50' 100' 150'

REVISIONS		
NO.	DATE	DESCRIPTION

BORING LAYOUT

CLIENT: HMB Professional Engineers, Inc.

PROJECT: Grant County Equalization Basin Grant County, KY

AEI
AMERICAN ENGINEERS, INC.
DESIGNING YOUR FUTURE
65 Aberdeen Drive Glasgow KY
270•651•7220

SCALE: 1"=50'

DATE: 12-30-2020

DRAWN BY: J. CHILDRESS

CHECKED BY: D. MITCHELL

FILE: T:/20 PROJECTS/220-308 Grant Co. Equilization Basin/Geotech/Reports/Support Information/GC Sewer Boring Layout.dgn

SHEET: B1

APPENDIX B

Boring Logs

FIELD TESTING PROCEDURES

The general field procedures employed by the Field Services Center are summarized in the following outline. The procedures utilized by the AEI Field Service Center are recognized methods for determining soil and rock distribution and ground water conditions. These methods include geophysical and in situ methods as well as borings.

Soil Borings are drilled to obtain subsurface samples using one of several alternate techniques depending upon the surface conditions. Borings are advanced into the ground using continuous flight augers. At prescribed intervals throughout the boring depths, soil samples are obtained with a split-spoon or thin-walled sampler and sealed in airtight glass jars and labeled. The sampler is first seated 6 inches to penetrate loose cuttings and then driven an additional foot, where possible, with blows from a 140 pound hammer falling 30 inches. The number of blows required to drive the sampler each six-inch increment is recorded. The penetration resistance, or “N-value” is designated as the number of hammer blows required to drive the sampler the final foot and, when properly evaluated, is an index to cohesion for clays and relative density for sands. The split spoon sampling procedures used during the exploration are in general accordance with ASTM D 1586. Split spoon samples are considered to provide *disturbed* samples, yet are appropriate for most engineering applications. Thin-walled (Shelby tube) samples are considered to provide *undisturbed* samples and obtained when warranted in general accordance with ASTM D 1587.

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.

Core Drilling Procedures for use on refusal materials. Prior to coring, casing is set in the boring through the overburden soils. Refusal materials are then cored according to ASTM D-2113 using a diamond bit attached to the end of a hollow double tube core barrel. This device is rotated at high speeds and the cuttings are brought to the surface by circulating water. Samples of the material penetrated are protected and retained in the inner tube, which is retrieved at the end of each drill run. Upon retrieval of the inner tube the core is recovered, measured and placed in boxes for storage.

The subsurface conditions encountered during drilling are reported on a field test boring record by the driller. 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 soil in general accordance with the procedures outlined in ASTM D 2487 and D 2488 and prepares the final boring records which are the basis for all evaluations and recommendations.

Representative portions of soil samples are placed in sealed containers and transported to the laboratory. In the laboratory, the samples are examined to verify the driller’s field classifications. Test Boring Records are attached which show the soil descriptions and penetration resistances.

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 designate 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.

Water table readings are normally taken in conjunction with borings and are recorded on the "Boring Logs". 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 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 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 as 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.

Sampling Terminology

Undisturbed Sampling: Thin-walled or Shelby tube samples used for visual examination, classification tests and quantitative laboratory testing. This procedure is described by ASTM D 1587. Each tube, together with the encased soil, is carefully removed from the ground, made airtight and transported to the laboratory. Locations and depths of undisturbed samples are shown on the "Boring Logs."

Bag Sampling: Bulk samples of soil are obtained at selected locations. These samples consist of soil brought to the surface by the drilling augers, or obtained from test pits or the ground surface using hand tools. Samples are placed in bags, with sealed jar samples of the material, and taken to our laboratory for testing where more mass material is required (i.e. Proctors and CBR's). The locations of these samples are indicated on the appropriate logs, or on the Boring Location Plan.

CLASSIFICATION SYSTEM FOR SOIL EXPLORATION

COHESIVE SOILS
(Clay, Silt, and Mixtures)

CONSISTENCY	SPT N-VALUE	Qu/Qp (tsf)
Very Soft	2 blows/ft or less	0 – 0.25
Soft	2 to 4 blows/ft	0.25 – 0.49
Medium Stiff	4 to 8 blows/ft	0.50 – 0.99
Stiff	8 to 15 blows/ft	1.00 – 2.00
Very Stiff	15 to 30 blows/ft	2.00 – 4.00
Hard	30 blows/ft or more	> 4.00

PLASTICITY

Degree of Plasticity	Plasticity Index (PI)
Low	0 – 7
Medium	8 – 22
High	over 22

NON-COHESIVE SOILS
(Silt, Sand, Gravel, and Mixtures)

DENSITY	SPT N-VALUE
Very Loose	4 blows/ft or less
Loose	4 to 10 blows/ft
Medium Dense	10 to 30 blows/ft
Dense	30 to 50 blows/ft
Very Dense	50 blows/ft or more

PARTICLE SIZE IDENTIFICATION

Boulders	12 inch diameter or more
Cobbles	3 to 12 inch diameter
Gravel	Coarse – 1 to 3 inch
	Medium – ½ to 1 inch
	Fine – ¼ to ½ inch
Sand	Coarse – 0.6mm to ¼ inch
	Medium – 0.2mm to 0.6mm
	Fine – 0.05mm to 0.2mm
Silt	0.05mm to 0.005mm
Clay	0.005mm

RELATIVE PROPORTIONS

Descriptive Term	Percent
Trace	1 – 10
Trace to Some	11 – 20
Some	21 – 35
And	36 – 50

NOTES

Classification – The Unified Soil Classification System is used to identify soil unless otherwise noted.

Standard "N" Penetration Test (SPT) (ASTM D1586) – Driving a 2-inch O.D., 1 3/8-inch I.D. sampler a distance of 1 foot into undisturbed soil with a 140-pound hammer free falling a distance of 30 inches. It is customary to drive the spoon 6-inches to seat the sampler into undisturbed soil, and then perform the test. The number of hammer blows for seating the spoon and making the tests are recorded for each 6 inches of penetration on the field drill long (e.g., 10/8/7). On the report log, the Standard Penetration Test result (i.e., the N value) is normally presented and consists of the sum of the 2nd and 3rd penetration counts (i.e., N = 8 + 7 = 15 blows/ft.)

Soil Property Symbols

Qu:	Unconfined Compressive Strength	N:	Standard Penetration Value (see above)
Qp:	Unconfined Comp. Strength (pocket pent.)	omc:	Optimum Moisture content
LL:	Liquid Limit, % (Atterberg Limit)	PL:	Plastic Limit, % (Atterberg Limit)
PI:	Plasticity Index	mdd:	Maximum Dry Density

AEI AMERICAN ENGINEERS, INC.
PROFESSIONAL ENGINEERING
65 Aberdeen Drive
Glasgow, KY 42141
(270) 651-7220

CLIENT HMB Professional Inc.
PROJECT NAME Grant County Equalization Basin
PROJECT NUMBER 220-308
PROJECT LOCATION Crittenden, KY
DATE STARTED 12/16/20 **COMPLETED** 12/16/20
GROUND ELEVATION 781.5 ft
DRILLING CONTRACTOR Adam Thompson
GROUND WATER LEVELS:
DRILLING METHOD HSA/ Diamond impregnated coring bit
AT TIME OF DRILLING ---
LOGGED BY Aaron Anderson **CHECKED BY** Dennis Mitchell
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	REMARKS
0		CRUSHED AGGREGATE (4 Inches)									
		TOPSOIL (20 Inches)									
		(CL) lean CLAY, brown with gray mottle, moist to wet, medium stiff	SPT 1	80	2-3-4 (7)	2.25	24				
5			ST 1	100		4.5+	21	44	25	19	Qu=4,322 psf
		SHALE, interbedded with limestone, brown to gray, soft to moderately hard, moderately to highly weathered, argillaceous	RC 1	61 (39)							.
10			RC 2	84 (36)							
15			RC 3	100 (52)							

Refusal at 5.7 feet.
Bottom of borehole at 19.0 feet.

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B-2

CLIENT HMB Professional Inc.
PROJECT NAME Grant County Equalization Basin
PROJECT NUMBER 220-308
PROJECT LOCATION Crittenden, KY
DATE STARTED 12/15/20 **COMPLETED** 12/15/20
GROUND ELEVATION 775.9 ft
DRILLING CONTRACTOR Adam Thompson
GROUND WATER LEVELS:
DRILLING METHOD HSA/ Diamond impregnated coring bit
AT TIME OF DRILLING ---
LOGGED BY Aaron Anderson **CHECKED BY** Dennis Mitchell
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	REMARKS
0		CRUSHED AGGREGATE (3 Inches)									
		TOPSOIL (21 Inches)									
		(CL) lean CLAY, brown with gray mottle, moist to wet, stiff	SPT 1	80	4-5-6 (11)	1.5	30				
			ST 1	70		4.5+	16	44	19	25	Qu=5,540 psf
5											.
		(GP) poorly graded GRAVEL, gray to white, very dense, sub-angular to sub-rounded	SPT 2	13	17-22-28 (50)	N/A					
10		weathered SHALE, tan to brown	SPT 3	100	12-22-50 (72)	4.5+	12				
		SHALE, interbedded with limestone, gray, soft to moderately hard, moderately weathered, argillaceous	RC 1	92 (0)							
			RC 2	98 (76)							
15											
			RC 3	96 (90)							
20											

Refusal at 10.2 feet.
Bottom of borehole at 22.6 feet.

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CLIENT HMB Professional Inc.
PROJECT NAME Grant County Equalization Basin
PROJECT NUMBER 220-308
PROJECT LOCATION Crittenden, KY
DATE STARTED 12/16/20 **COMPLETED** 12/16/20
GROUND ELEVATION 775.4 ft
DRILLING CONTRACTOR Adam Thompson
GROUND WATER LEVELS:
DRILLING METHOD HSA/ Diamond impregnated coring bit
AT TIME OF DRILLING ---
LOGGED BY Aaron Anderson **CHECKED BY** Dennis Mitchell
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	REMARKS
0		CRUSHED AGGREGATE (4 Inches)									
		(CL-ML) silty lean CLAY, light brown, moist									
		(CL) lean CLAY, gray to brown, moist, very stiff	SPT 1	87	9-9-12 (21)	4.5+	17				
5			ST 1	70		4.5+	17	40	23	17	Qu=9,242 psf
		weathered SHALE, gray	SPT 2	100	22-50	N/A	9				.
		SHALE, gray, soft to moderately hard, moderately weathered, argillaceous	RC 1	86 (50)							
10			RC 2	100 (20)							
15			RC 3	98 (64)							

Refusal at 8.0 feet.
Bottom of borehole at 19.4 feet.

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CLIENT HMB Professional Inc.
PROJECT NAME Grant County Equalization Basin
PROJECT NUMBER 220-308
PROJECT LOCATION Crittenden, KY
DATE STARTED 12/16/20 **COMPLETED** 12/16/20
GROUND ELEVATION 786.9 ft
DRILLING CONTRACTOR Adam Thompson
GROUND WATER LEVELS:
DRILLING METHOD HSA/ Diamond impregnated coring bit
AT TIME OF DRILLING ---
LOGGED BY Aaron Anderson **CHECKED BY** Dennis Mitchell
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	REMARKS
0		TOPSOIL (4 Inches)	SPT 1	100	1-4-8 (12)	2.5	18				
		(CL-ML) silty lean CLAY, trace gravel, light brown, moist, stiff									
		(CL) lean CLAY, gray to brown, moist, very stiff	SPT 2	93	7-15-14 (29)	4.5+	16				
5		weathered SHALE, gray	SPT 3	100	12-14-50 (64)	4.5+	17				

Refusal at 5.2 feet.
Bottom of borehole at 5.2 feet.

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CLIENT HMB Professional Inc.
PROJECT NAME Grant County Equalization Basin
PROJECT NUMBER 220-308
PROJECT LOCATION Crittenden, KY
DATE STARTED 12/17/20 **COMPLETED** 12/17/20
GROUND ELEVATION 763.5 ft
DRILLING CONTRACTOR Adam Thompson
GROUND WATER LEVELS:
DRILLING METHOD PROBE ROD
AT TIME OF DRILLING ---
LOGGED BY Aaron Anderson **CHECKED BY** Dennis Mitchell
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS			REMARKS
								LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	
0		OVERBURDEN (2.1 Feet)									

Refusal at 2.1 feet.
Bottom of borehole at 2.1 feet.

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CLIENT HMB Professional Inc.
PROJECT NAME Grant County Equalization Basin
PROJECT NUMBER 220-308
PROJECT LOCATION Crittenden, KY
DATE STARTED 12/17/20 **COMPLETED** 12/17/20
GROUND ELEVATION 763.6 ft
DRILLING CONTRACTOR Adam Thompson
GROUND WATER LEVELS:
DRILLING METHOD PROBE ROD
AT TIME OF DRILLING ---
LOGGED BY Aaron Anderson **CHECKED BY** Dennis Mitchell
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	REMARKS
0		OVERBURDEN (2.5 Feet)									

Refusal at 2.5 feet.
Bottom of borehole at 2.5 feet.

APPENDIX C

Laboratory Testing Results

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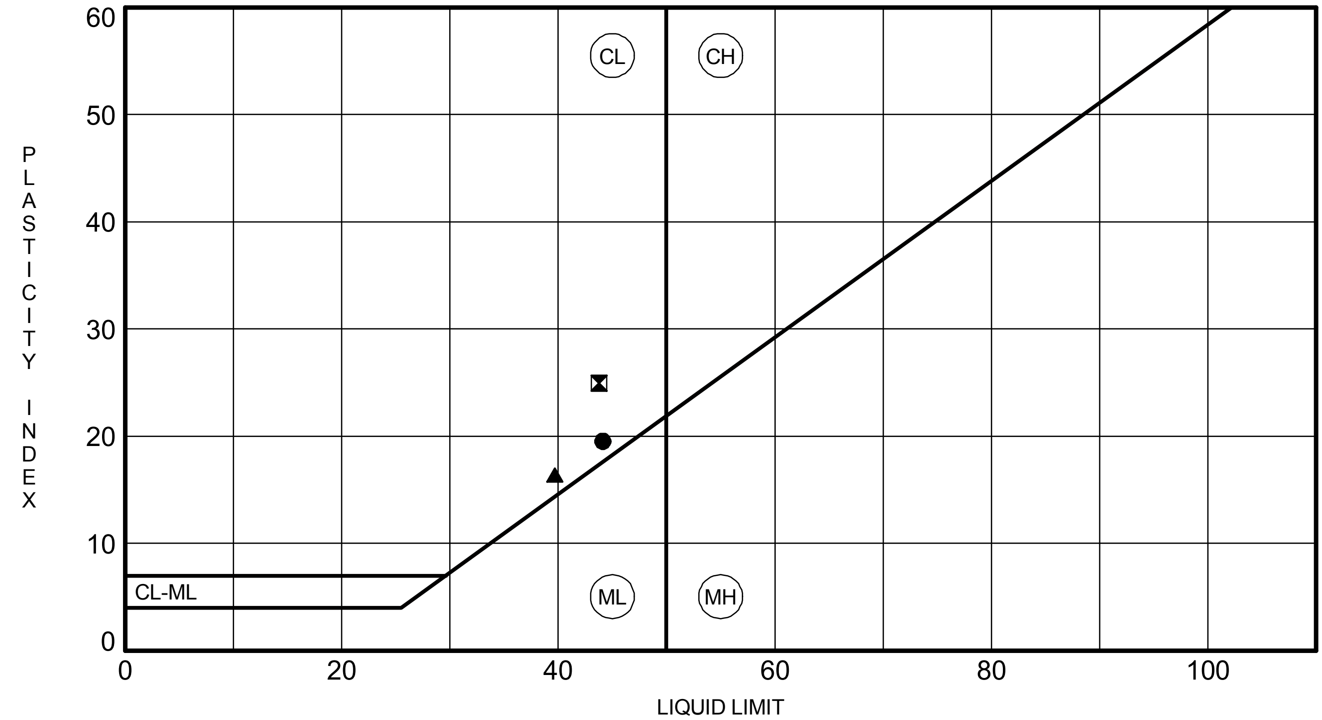
ATTERBERG LIMITS' RESULTS

CLIENT HMB Professional Inc. **PROJECT NAME** Grant County Equalization Basin

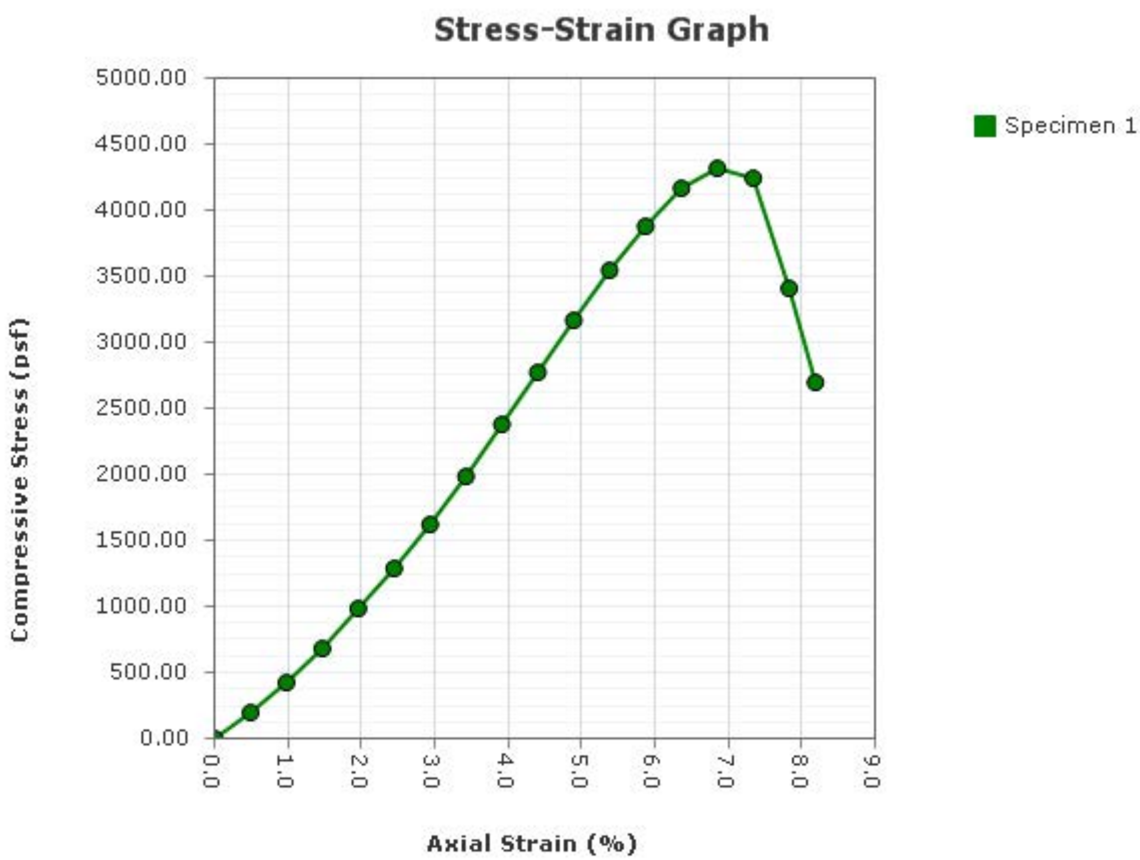
PROJECT NUMBER 220-308 **PROJECT LOCATION** Crittenden, KY



	BOREHOLE	DEPTH	LL	PL	PI	Fines	Classification
●	**B-1**	**4.0**	**44**	**25**	**19**		**(CL) LEAN CLAY**
⧗	**B-2**	**3.0**	**44**	**19**	**25**		**(CL) LEAN CLAY**
▲	**B-3**	**4.0**	**40**	**23**	**17**		**(CL) LEAN CLAY**

Unconfined Compression Test

ASTM D2166



Project:	Grant County Equalization Basin
Project Number:	220-308
Received Date:	12/30/2020
Sampling Date:	12/30/2020
Sample Number:	ST-1
Sample Depth:	4-6 ft
Boring Number:	B-1
Location:	Grant County, KY
Client Name:	HMB Professional Engineers, Inc.
Remarks:	

Unconfined Compression Test

ASTM D2166

	Specimen Number							
Before Test	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
Moisture Content (%):	17.5							
Wet Density (pcf)	134.3							
Dry Density (pcf)	114.3							
Saturation (%):	98.0							
Void Ratio:	0.486							
Height (in)	5.1100							
Diameter (in)	2.8450							
Strain Limit @ 15% (in)	0.8							
Height To Diameter Ratio:	1.80							
Test Data	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
Failure Angle (°):	0							
Strain Rate (in/min)	0.1							
Strain Rate (%/min):	1.96							
Unconfined Compressive Strength (psf)	4322.17							
Undrained Shear Strength (psf)	2161.09							
Strain at Failure (%):	6.85							

Specific Gravity:	2.72	Plastic Limit:	0	Liquid Limit:	0
Type:	UD	Soil Classification:			

Project:	Grant County Equalization Basin
Project Number:	220-308
Sampling Date:	12/30/2020
Sample Number:	ST-1
Sample Depth:	4-6 ft
Boring Number:	B-1
Location:	Grant County, KY
Client Name:	HMB Professional Engineers, Inc.
Remarks:	

Specimen 1 Failure Sketch

Specimen 2 Failure Sketch

Specimen 3 Failure Sketch

Specimen 4 Failure Sketch

Specimen 5 Failure Sketch

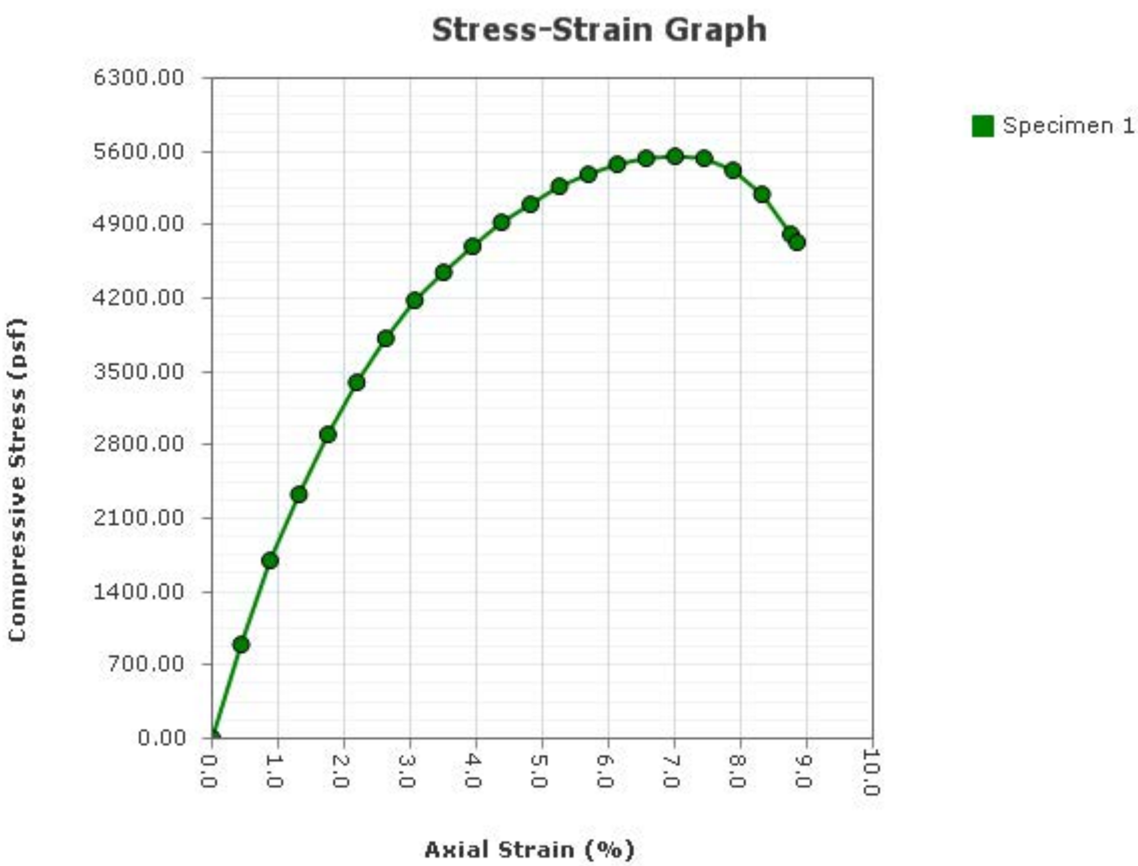
Specimen 6 Failure Sketch

Specimen 7 Failure Sketch

Specimen 8 Failure Sketch

Unconfined Compression Test

ASTM D2166



Project:	Grant County Equalization Basin
Project Number:	220-308
Received Date:	12/30/2020
Sampling Date:	12/30/2020
Sample Number:	ST-1
Sample Depth:	3-5 ft
Boring Number:	B-2
Location:	Grant County, KY
Client Name:	HMB Professional Engineers, Inc.
Remarks:	

Unconfined Compression Test

ASTM D2166

	Specimen Number							
Before Test	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
Moisture Content (%):	19.3							
Wet Density (pcf)	134.2							
Dry Density (pcf)	112.4							
Saturation (%):	103.2							
Void Ratio:	0.510							
Height (in)	5.7100							
Diameter (in)	2.8500							
Strain Limit @ 15% (in)	0.9							
Height To Diameter Ratio:	2.00							
Test Data	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
Failure Angle (°):	0							
Strain Rate (in/min)	0.1							
Strain Rate (%/min):	1.75							
Unconfined Compressive Strength (psf)	5540.44							
Undrained Shear Strength (psf)	2770.22							
Strain at Failure (%):	7.44							

Specific Gravity:	2.72	Plastic Limit:	0	Liquid Limit:	0
Type:	UD	Soil Classification:			

Project:	Grant County Equalization Basin
Project Number:	220-308
Sampling Date:	12/30/2020
Sample Number:	ST-1
Sample Depth:	3-5 ft
Boring Number:	B-2
Location:	Grant County, KY
Client Name:	HMB Professional Engineers, Inc.
Remarks:	

Specimen 1 Failure Sketch

Specimen 2 Failure Sketch

Specimen 3 Failure Sketch

Specimen 4 Failure Sketch

Specimen 5 Failure Sketch

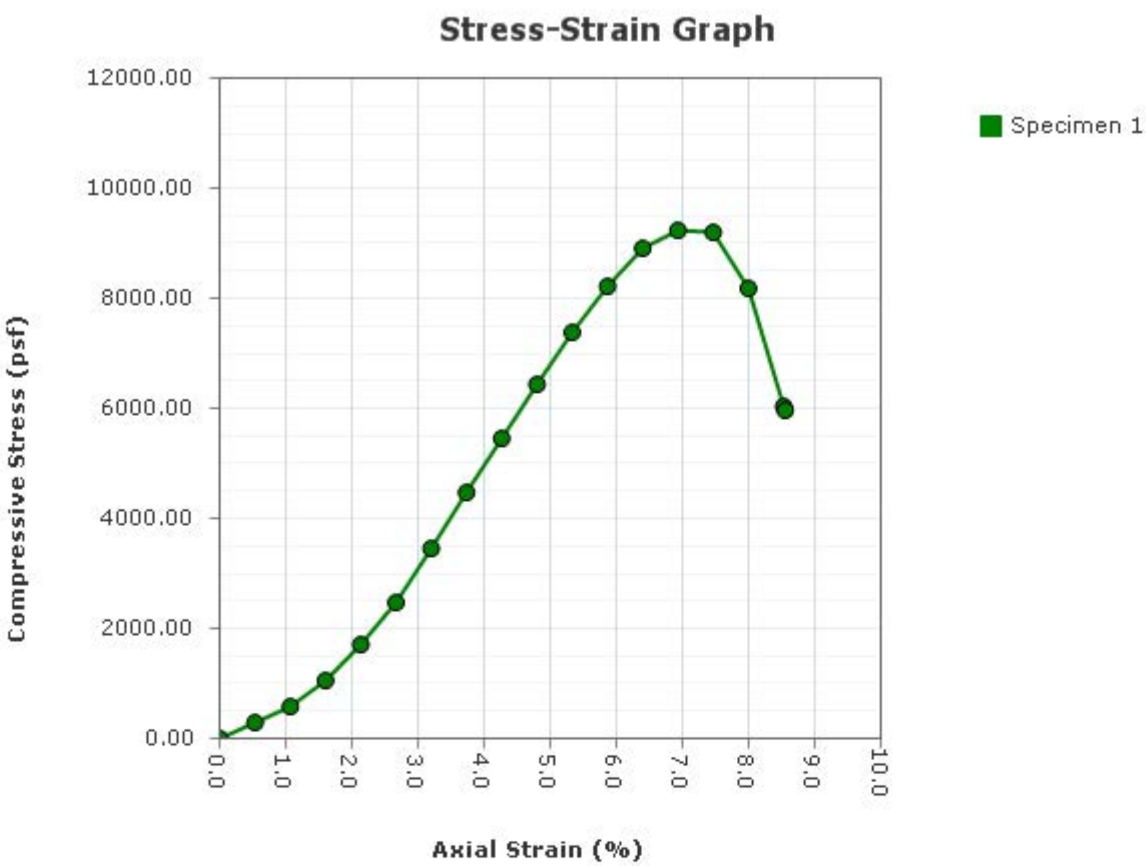
Specimen 6 Failure Sketch

Specimen 7 Failure Sketch

Specimen 8 Failure Sketch

Unconfined Compression Test

ASTM D2166



Project:	Grant County Equalization Basin
Project Number:	220-308
Received Date:	12/30/2020
Sampling Date:	12/30/2020
Sample Number:	ST-1
Sample Depth:	4-6 ft
Boring Number:	B-3
Location:	Grant County, KY
Client Name:	HMB Professional Engineers, Inc.
Remarks:	

Unconfined Compression Test

ASTM D2166

	Specimen Number							
Before Test	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
Moisture Content (%):	15.1							
Wet Density (pcf)	136.7							
Dry Density (pcf)	118.7							
Saturation (%):	95.6							
Void Ratio:	0.430							
Height (in)	4.6900							
Diameter (in)	2.8400							
Strain Limit @ 15% (in)	0.7							
Height To Diameter Ratio:	1.65							
Test Data	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
Failure Angle (°):	0							
Strain Rate (in/min)	0.09							
Strain Rate (%/min):	1.92							
Unconfined Compressive Strength (psf)	9242.69							
Undrained Shear Strength (psf)	4621.35							
Strain at Failure (%):	6.93							

Specific Gravity:	2.72	Plastic Limit:	0	Liquid Limit:	0
Type:	UD	Soil Classification:			

Project:	Grant County Equalization Basin
Project Number:	220-308
Sampling Date:	12/30/2020
Sample Number:	ST-1
Sample Depth:	4-6 ft
Boring Number:	B-3
Location:	Grant County, KY
Client Name:	HMB Professional Engineers, Inc.
Remarks:	

Specimen 1 Failure Sketch	Specimen 2 Failure Sketch	Specimen 3 Failure Sketch	Specimen 4 Failure Sketch	Specimen 5 Failure Sketch	Specimen 6 Failure Sketch	Specimen 7 Failure Sketch	Specimen 8 Failure Sketch
							

Your Geotechnical Engineering Report

To help manage your risks, this information is being provided because subsurface issues are a major cause of construction delays, cost overruns, disputes, and claims.

Geotechnical Services are Performed for Specific Projects, Purposes, and People

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering exploration conducted for an engineer may not fulfill the needs of a contractor or even another engineer. Each geotechnical engineering exploration and report is unique and is prepared solely for the client. No one except the client should rely on the geotechnical engineering report without first consulting with the geotechnical engineer who prepared it. The report should not be applied for any project or purpose except the one originally intended.

Read the Entire Report

To avoid serious problems, the full geotechnical engineering report should be read in its entirety. Do not only read selected sections or the executive summary.

A Unique Set of Project-Specific Factors is the Basis for a Geotechnical Engineering Report

Geotechnical engineers consider a numerous unique, project-specific factors when determining the scope of a study. Typical factors include: the client's goals, objectives, project costs, risk management preferences, proposed structures, structures on site, topography, and other proposed or existing site improvements, such as access roads, parking lots, and utilities. Unless indicated otherwise by the geotechnical engineer who conducted the original exploration, a geotechnical engineering report should not be relied upon if it was:

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

Typical changes that can lessen 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 multi-story hotel to a parking lot
- finished floor elevation, location, orientation, or weight of the proposed structure, anticipated loads or
- project ownership

Geotechnical engineers cannot be held liable or responsible for issues that occur because their report did not take into account development items of which they were not informed. The geotechnical engineer should always be notified of any project changes. Upon notification, it should be requested of the geotechnical engineer to give an assessment of the impact of the project changes.

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that exist at the time of the exploration. A geotechnical engineering report should not be relied upon if its reliability could be in question due to factors such as man-made events as construction on or adjacent to the site, natural events such as floods, earthquakes, or groundwater fluctuation, or time. To determine if a geotechnical report is still reliable, contact the geotechnical engineer. Major problems could be avoided by performing a minimal amount of additional analysis and/or testing.

Most Geotechnical Findings are Professional Opinions

Geotechnical site explorations identify subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field logs and laboratory data and apply their professional judgment to make conclusions about the subsurface conditions throughout the site. Actual subsurface conditions may differ from those indicated in the report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risk associated with unanticipated conditions.

The Recommendations within a Report Are Not Final

Do not put too much faith on the construction recommendations included in the report. The recommendations are not final due to geotechnical engineers developing them principally from judgment and opinion. Only by observing actual subsurface conditions revealed during construction can geotechnical engineers finalize their recommendations. Responsibility and liability cannot be assumed for the recommendations within the report by the geotechnical engineer who developed the report if that engineer does not perform construction observation.

A Geotechnical Engineering Report Is Subject To Misinterpretation

Misinterpretation of geotechnical engineering reports has resulted in costly problems. The risk of misinterpretation can be lowered after the submittal of the final report by having the geotechnical engineer consult with appropriate members of the design team. The geotechnical engineer could also be retained to review crucial parts of the plans and specifications put together by the design team. The geotechnical engineering report can also be misinterpreted by contractors which can result in many problems. By participating in pre-bid and preconstruction meetings and providing construction observations by the geotechnical engineer, many risks can be reduced.

Final Boring Logs Should not be Re-drawn

Geotechnical engineers prepare final boring logs and testing results based on field logs and laboratory data. The logs included in a final geotechnical engineering report should never be redrawn to be included in architectural or design drawings due to errors that could be made. Electronic reproduction is acceptable, along with photographic reproduction, but it should be understood that separating logs from the report can elevate risk.

Contractors Need a Complete Report and Guidance

By limiting what is provided for bid preparation, contractors are not liable for unforeseen subsurface conditions although some owners and design professionals believe the opposite to be true. The complete geotechnical engineering report, accompanied with a cover letter or transmittal, should be provided to contractors to help prevent costly problems. The letter states that the report was not prepared for purposes of bid development and the report's accuracy is limited. Although a fee may be required, encourage the contractors to consult with the geotechnical engineer who prepared the report and/or to conduct additional studies to obtain the specific types of information they need or prefer. A prebid conference involving the owner, geotechnical engineer, and contractors can prove to be very valuable. If needed, allow contractors sufficient time to perform additional studies. Upon doing this you might 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.

Closely Read Responsibility Provisions

Geotechnical engineering is not as exact as other engineering disciplines. This lack of understanding by clients, design professionals, and contractors has created unrealistic expectations that have led to disappointments, claims, and disputes. To minimize such risks, a variety of explanatory provisions may be included in the report by the geotechnical engineer. To help others recognize their own responsibilities and risks, many of these provisions indicate where the geotechnical engineer's responsibilities begin and end. These provisions should be read carefully, questions asked if needed, and the geotechnical engineer should provide satisfactory responses.

Environmental Issues/Concerns are not Covered

Unforeseen environmental issues can lead to project delays or even failures. Geotechnical engineering reports do not usually include environmental findings, conclusions, or recommendations. As with a geotechnical engineering report, do not rely on an environmental report that was prepared for someone else.

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