DDS ENGINEERING, PLLC
LAND SURVEYING • CIVIL AND GEOTECHNICAL ENGINEERING
CONSTRUCTION MATERIALS TESTING • SPECIAL INSPECTIONS

December 15, 2022

Mr. Tim Doelling
K Norman Berry Associates Architects
815 West Market Street, Suite 502
Louisville, Kentucky 40202

RE: Report of Geotechnical Engineering Subsurface Site Characterization
New WKU Hilltopper Fieldhouse
Western Kentucky University
Bowling Green, Warren County, Kentucky
DDS Project No – T5600G

Dear Mr. Doelling,

DDS ENGINEERING, PLLC has completed a geotechnical engineering subsurface site characterization for the new WKU Hilltopper Fieldhouse at Western Kentucky University in Bowling Green, Kentucky. Our exploration was performed in general accordance with our proposal dated October 25, 2022 and authorized by Mr. Tim Doelling of K. Norman Berry Architects on October 27, 2022.

The attached report describes our general understanding of the project, summarizes our scope of work, and presents our findings. The appendix contains site exhibits, boring records, and a summary of laboratory test results.

We appreciate the opportunity to be of service to you on this project. Please contact us if you have any questions or require any additional information.

Sincerely,

DDS ENGINEERING, PLLC

Matthew H. Rogers, PE
Senior Engineer
KY License Number 27075

STATE OF KENTUCKY
MATTHEW H. ROGERS
27075
LICENSED
PROFESSIONAL ENGINEER

12-15-22

REPORT OF GEOTECHNICAL ENGINEERING SUBSURFACE SITE CHARACTERIZATION

FOR

WESTERN KENTUCKY UNIVERSITY
NEW WKU HILLTOPPER FIELDHOUSE
BOWLING GREEN, WARREN COUNTY, KENTUCKY

PREPARED FOR

MR. TIM DOELLING
K NORMAN BERRY ASSOCIATES ARCHITECTS
815 WEST MARKET STREET, SUITE 502
LOUISVILLE, KENTUCKY 40202

PREPARED BY

DDS ENGINEERING, PLLC
148 CHESTER COURT
BOWLING GREEN, KENTUCKY 42103

DECEMBER 15, 2022

DDS PROJECT NUMBER – T5600G

TABLE OF CONTENTS

1 EXECUTIVE SUMMARY

DDS ENGINEERING, PLLC (DDS) has completed a geotechnical engineering subsurface site characterization for the proposed new Western Kentucky University (WKU) Hilltopper Fieldhouse. We understand the new Hilltopper Fieldhouse will be an indoor practice facility for WKU Athletics programs. The new facility will be located between L.T. Smith Stadium and Nick Denes Field. The site is currently grass covered and used as an outdoor practice facility for WKU Football.

Project details are limited at the time of this report's completion however we understand the new construction will consist of a single story, approximate 69,500 square feet structure with a finished floor elevation of 493.25 feet MSL. Structural loading information is not available however we have assumed column loads will be less than 300 kips. Based on a provided topographic survey and proposed finished floor elevation, we anticipated that less than 2 feet of cut or fill would be required to reach planned grades.

To characterize the subsurface conditions of the site, the field exploration consisted of drilling fourteen (14) soil test borings. Beneath the topsoil or asphalt, the borings encountered previously placed fill that generally consisted of dark brown to gray and black silt and clay. Most of the previously placed fill had a strong organic odor. Some of the recovered samples contained wood fragments and construction debris (brick fragments). The fill thickness ranged from about 1 ½ feet to 12 feet below the surface and extended to the auger refusal depth in seven of the borings. Residual soils were encountered beneath the previously placed fill in seven of the borings and consisted of orangish to brownish red lean to fat clay. Auger refusal depths ranged from 2.5 to 12.3 feet below the surface. Refusal material was sampled in three of the borings. The refusal material consisted of moderately hard to hard, white to light gray limestone.

Based on the results of our exploration and understanding of the proposed construction, we believe the site is adaptable for the proposed construction. However, the previously placed fill and Karst topography encountered within the borings pose some risk from a geotechnical standpoint and may have a financial impact to the project. A discussion of each item is presented throughout the text of this report and is detailed in the Discussion section of this report.

2 PROJECT AND SITE INFORMATION

Project information was received through email correspondence and phone conversations with Mr. Tim Doelling of K. Norman Berry Architects. We received an existing topographic survey drawing dated 7/20/16 and a drawing depicting the proposed building footprint. A screenshot of the proposed building footprint is shown in Figure 1 below.

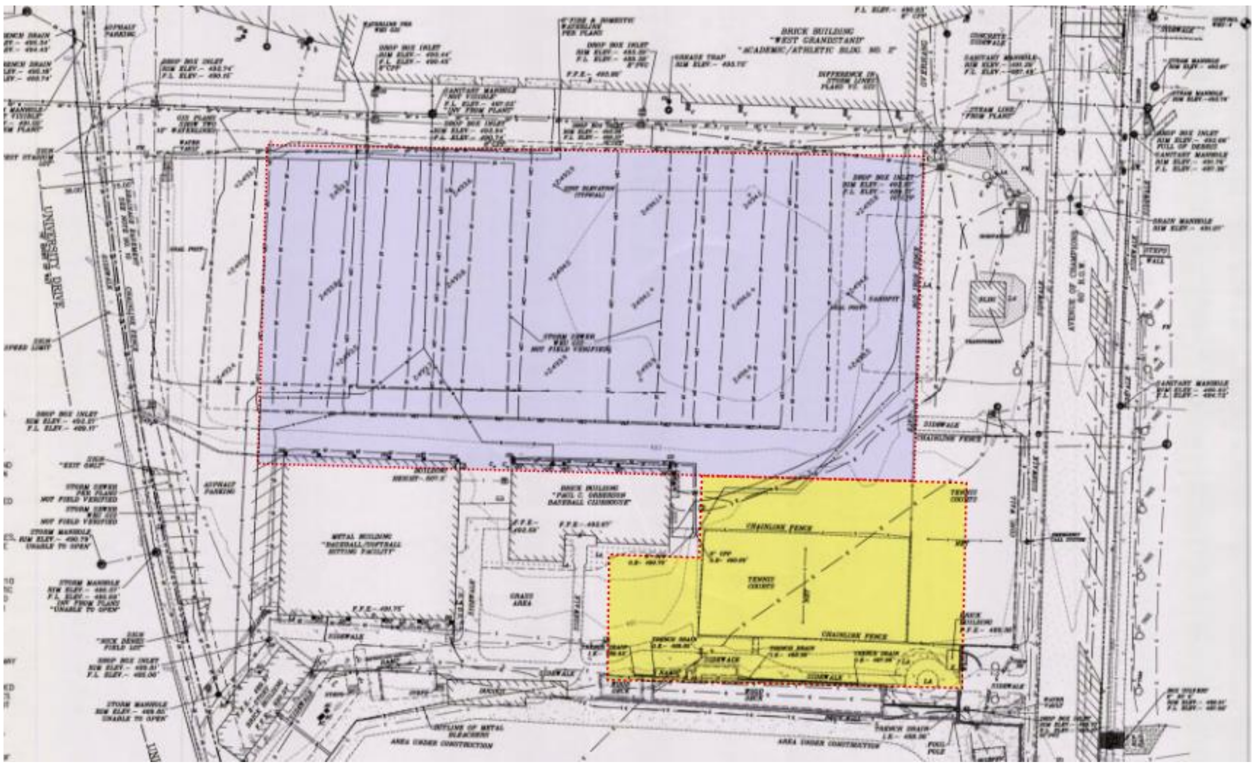


Figure 1 – Conceptual Building Footprint

We understand the new Hilltopper Fieldhouse will be a single story, approximate 69,500 square feet structure with a finished floor elevation of 493.25 feet MSL. Structural loading information was not available at the time of this report's completion, however we have assumed column loads will be less than 300 kips. Settlement tolerances are assumed to be about ¾ inches between columns and 1 inch total.

The site is bordered by University Boulevard to the northwest and Avenue of Champions to the southeast and lies between L.T. Smith Stadium and Nick Denes Field. The grass covered site is currently used an outdoor practice facility for WKU Football. Asphalt paved entrances and parking areas for LT Smith Stadium and Nick Denes Field are located off University Boulevard west - northwest of the practice field. Site drainage was judged to be fair with surface water directed by the terrain and into existing stormwater inlets.

A review of historical aerial photographs from Google Earth and Historical Information Gatherers indicates that the site has been previously developed and used for various purposes dating back to residential use in 1950. In 2008, the stie was used as staging and a laydown area during construction of the L.T. Smith Stadium's West Grandstand. A site vicinity exhibit, aerial photograph exhibits, and select photographs taken during our site visits are provided in the Appendix of this report.

2.1 SITE GEOLOGY

A review of published geological information from the United States Geological Survey (USGS) and the Kentucky Geological Survey (KGS) reveals that the site rests within Bowling Green South Quadrangle (GQ-235) and is underlain by the St. Genevieve Limestone formation. According to the USGS geologic survey, St. Genevieve Limestone formation is described as oolitic limestone that is fine to medium grained, crystalline, cherty, gray to white, and thin to thick bedded. The limestone is described to weather blocky, particularly near the base where large weathered blocks of chert as much as 2 feet long and 1 foot thick are common in soil residue.

We reviewed the KGS Geologic Map Information Service website for Karst potential. The KGS website indicates the site to have a very high potential for karst activity and maps several closed topographic depressions within ¼ mile of the site. To further characterize the bedrock and the Karst potential at the site, a geophysical survey such as electrical resistivity and additional rock coring would be required. A Karst potential exhibit referenced from the KGS website is provided in the Appendix. A more detailed discussion of Karst conditions is provided in the discussions section of this report.

3 FIELD EXPLORATION

The subsurface exploration consisted of drilling 14 soil test borings. Soil test borings were identified as borings B-1 through B-14 and were generally located per the requested boring location plan dated 8/18/2022 prepared by KNBA and Brown + Kubican. Some borings were offset from the planned locations to avoid site features such as underground utilities. The final boring locations and top of ground elevations were staked in the field using survey grade GPS techniques.

Our project engineer, Matt Rogers, PE visited the site on November 1st through November 3rd, 2022 to observe existing site features, assist with boring layout, coordinate potential existing utility conflicts, and direct the drilling operations. Soil test borings were advanced to auger refusal using hollow stem augers powered by a track mounted Diedrich D-50 drill rig. Select soil samples were obtained within soil test borings through split barrel sampling from standard penetration testing (SPT). The SPT test is a process in which a 140-pound hammer is dropped 30 inches driving the sampler into the ground. The number of blows needed to drive the sampler 12 inches is called the N value, which is the basis for many empirical design criteria in Geotechnical Engineering.

Refusal material was sampled in three of the borings using rock coring methods consisting of a diamond-studded bit fastened to the end of a hollow tube core barrel. During drilling, soil samples were visually classified and transported to the laboratory for further classification and testing. At the conclusion of drilling, the borings were backfilled with the auger cuttings. The boring advanced through asphalt pavement was capped with an asphalt cold patch. Procedures used during our field exploration were performed in general accordance with ASTM procedures and established engineering practice.

3.1 SUBSURFACE PROFILE

Thirteen of the borings were advanced in grass covered areas and one boring was advanced in an asphalt paved parking area. Topsoil ranged from about 4 to 10 inches thick and averaged about 6 inches thick. Boring B-13 was advanced in the asphalt paved entrance to L.T. Smith Stadium off University Boulevard. The boring encountered about 5 inches of asphalt and was underlain by about 11 inches of crushed limestone fill.

Beneath the topsoil or pavement, the borings encountered organic laden previously placed fill that generally consisted dark brown to gray and black silt and clay. Most of the previously placed fill had a strong organic odor. Some of the recovered samples contained wood fragments and construction debris (brick fragments). The fill thickness ranged from about 1 ½ feet to 12 feet below the surface and extended to the auger refusal depth in seven of the borings. Standard penetration resistance N values within the fill were inconsistent and ranged from 5 to 33 bpf indicating a firm to very stiff soil consistency. The higher N values within the fill were likely due to the presences of debris or crushed limestone within the predominate silt and clay soils. The fill thickness encountered in each boring is shown in Table 1 below. The below depths were fill thickness encountered in the borings and can vary across the site.

Table 1 – Previously Placed Fill Thickness Summary

BORING	DEPTH (FT)
B - 1	2.5
B - 2	5.3
B - 3	12.3
B - 4	9.2
B - 5	6.4
B - 6	1.6
B - 7	5.4

BORING	DEPTH (FT)
B - 8	4.5
B - 9	4.0
B - 10	2.6
B - 11	1.2
B - 12	2.2
B - 13	1.4
B - 14	3.0

Residual soils were encountered beneath the previously placed fill in seven of the borings. The residual soils consisted of orangish to brownish red lean to fat clay with some chert. SPT N values within the clay soils ranged from 9 to 11 bpf indicating a stiff soil consistency. Soil plasticity tests (Atterberg Limits) were performed on three of the residual clay samples recovered. The tests produced liquid limits of 42, 44, and 51 with associated plasticity indices of 24, 25, and 28, respectively. According to our laboratory tests and the Unified Soil Classification System, the residual clay samples were classified as either a lean clay (CL), clay with low to moderate plasticity or fat clay (CH), clay with moderate plasticity. Where encountered, the residual clay soils extended to auger refusal.

Auger refusal depths ranged from 2.5 feet to 12.3 feet below the surface. Auger refusal is defined as the depth below the ground surface at which the auger can no longer be advanced with the drilling technique used. In areas of limestone, refusal can be obtained by layers of limestone "floaters" suspended in a matrix of clay, rock pinnacles rising above the surrounding the bedrock surface, or widened joints in the bedrock that may extend below the surrounding bedrock. The auger refusal depth, top of ground elevation, and refusal elevation is provided in Table 2 below.

Table 2 – Auger Refusal Summary

BORING NUMBER	SURFACE ELEV (FT)	REFUSAL DEPTH (FT)	REFUSAL ELEV (FT)
B - 1	493.5	3.7	489.8
B - 2	492.3	9.8	482.5
B - 3	491.5	12.5	479.0
B - 4	493.0	9.2	483.8
B - 5	494.1	6.7	487.4
B - 6	493.4	3.7	489.7
B - 7	494.0	5.4	488.6

BORING NUMBER	SURFACE ELEV (FT)	REFUSAL DEPTH (FT)	REFUSAL ELEV (FT)
B - 8	493.3	7.7	485.6
B - 9	493.3	4.3	489.0
B - 10	493.9	4.2	489.7
B - 11	493.4	4.5	488.9
B - 12	493.3	2.5	490.8
B - 13	493.9	3.2	490.7
B - 14	492.8	3.0	489.8

Refusal material was sampled by rock coring methods in Borings B-2, B-3, and B-14. The refusal material consisted of moderately hard to hard, white to light gray limestone. Some bedrock discontinuities including voids and/or soil filled seams were encountered within the sampled bedrock. A summary of the discontinuities are provided in Table 3 below.

Table 3 – Bedrock Discontinuities Encountered

BORING NUMBER	SURFACE ELEV (FT)	REFUSAL DEPTH (FT)	REFUSAL ELEV (FT)	NOTES
B - 2	492.3	9.8	482.5	VOID/SEAM (13.2'-13.5') (16.6'-17.0') (26.3'-26.5')
B - 3	491.5	12.5	479.0	VOID/SEAM (16.8'-17.1') (20.5'-20.7')
B - 14	492.8	3.0	489.8	CLAY SEAM FROM 4.1' - 6.2'

The rock quality designation (RQD) is an indication of the rock quality from an engineering standpoint and is measured by summing rock segments 4 inches and longer and dividing by the total length of the rock core run. The RQD's of the bedrock sampled ranged from about 61% to 100%. Several rock unconfined compressive strength tests were performed from the recovered bedrock and ranged from 1,930 to 3,215 kips per square foot (ksf). A summary of the bedrock RQD, recovery, and unconfined compressive strength test is provided in Table 3 below.

Table 4 – Summary of Bedrock RQD, REC, and Unconfined Compressive Strength

BORING NUMBER	CORE RUN DEPTH (FT)	REC. (%)	RQD (%)	TEST SAMPLE DEPTH (FT)	UNIT WEIGHT (PCF)	UNC. COMP. STRENGTH (KSF)
B - 2	9.8 - 19.8	92	65	(11.0) (12.0)	(167.6) (168.7)	(1,930) (3,215)
B - 2	19.8 - 29.8	93	83	(23.2) (27.0)	(163.0) (165.7)	(1,950) (2,225)
B - 3	12.5 - 22.0	96	62	13.8	168.4	3,050
B - 3	22.0 - 32.0	99	84	(22.2) (28.6)	(169.1) (167.9)	(3,090) (2,929)
B - 14	3.0 - 10.5	72	61	6.6	167.6	2,977
B - 14	10.5 - 20.5	100	100	(10.7)(13.5)(17.6)	(166.1) (160.0) (155.6)	(2,441) (2,424) (1,999)

The individual boring records are provided in the Appendix. The soil stratification symbols shown on the boring logs represent the approximate boundary of the subsurface strata. However, the transition may be more gradual than shown.

3.2 WATER LEVEL MEASUREMENTS

The borings were dry upon completion of soil augering. However, in Karst landscapes, water can accumulate in solution features and pockets between the soil and bedrock interface. Groundwater levels fluctuate with seasonal and cyclical climate variations in precipitation and may be higher or lower at other times. Typically, water conditions affecting construction projects in this type of geology are related to trapped or perched water which occurs in irregular, discontinuous locations within the soil overburden, or near the soil and bedrock interface. When these water bearing strata are exposed in excavations, such as cut slopes, utility or footing trenches, they can produce widely varying seepage durations and rates depending on recent rainfall activity and other site-specific characteristics of the area. In areas with Karst bedrock, the static groundwater table is typically encountered below the bedrock surface. To monitor water levels more accurately at this site, piezometers would be required to be installed. If groundwater seepage occurs, it should be controlled by conventional pump methods.

4 LABORATORY TESTING

Laboratory tests were conducted on select soil samples obtained during our exploration. Laboratory tests included natural moisture content, Atterberg limits plasticity tests, and rock unconfined compressive strength test. Laboratory test reports and a summary of laboratory tests data is provided in the Appendix of this report. The field and laboratory procedures used were in general accordance with ASTM procedures and established geotechnical engineering practice.

5 DISCUSSION

Based on the results of our exploration and understanding of the proposed construction, we believe the site is adaptable for the proposed construction. However, the Previously Placed Fill and Karst Topography and pose some risk from a geotechnical standpoint.

5.1 PREVIOUSLY PLACED FILL

The borings encountered previously placed fill that generally consisted dark brown to gray and black silt and clay. Most of the previously placed fill had a strong organic odor and some of the recovered samples contained wood fragments and construction debris (brick fragments). The fill thickness ranged from about 1 ½ feet to 12 feet below the surface and extended to the auger refusal depth in seven of the borings. The thicker and heavier organic laden fill was encountered in the central to southern portions of the site in borings B-3, B-4, B-5. Fill thickness in other areas of the site was about five feet or less. The fill thickness encountered in each boring is shown in Table 1 below.

Table 5 – Previously Placed Fill Thickness Summary

BORING	DEPTH (FT)
B - 1	2.5
B - 2	5.3
B - 3	12.3
B - 4	9.2
B - 5	6.4
B - 6	1.6
B - 7	5.4

BORING	DEPTH (FT)
B - 8	4.5
B - 9	4.0
B - 10	2.6
B - 11	1.2
B - 12	2.2
B - 13	1.4
B - 14	3.0

The organic laden material and construction debris suggest the fill was not placed with quality control typically required for building support. Uncontrolled fills can contain zones of less compact materials which can settle under their weight or new loading. There is an inherent risk of settlement and poor structural performance associated with supporting structures on uncontrolled fill. Also, it is difficult to predict the amount and rate at which settlement will occur. The following paragraphs discuss the geotechnical risks associated with uncontrolled fill.

<u>General Risks Associated with Uncontrolled Fill</u>

Whenever uncontrolled fill or organic laden soils are encountered, there is a risk of differential settlement, which could result in differential settlement of the foundations, cracked floor slabs, or depressions/dips in pavements if they are supported on the fill. Also, uncontrolled fills can contain zones of less compact materials which will settle under their weight or new loading.

<u>A Site-Specific Opinion on the Risks of Differential Settlement for Uncontrolled Fill</u>

When considering the risks of building over uncontrolled fill, the design team should consider 1) the consistency of the SPT penetration values, and 2) the composition of the fill materials. At this site, there is a large variability in the penetration resistance values suggesting variable support conditions. Additionally, the organic laden material within the fill may continue to degrade over time and cause variable support conditions.

Due to the risk of building on uncontrolled fills, uncontrolled fills and organic laden soils are typically removed and replaced with engineered fill. Fill thicknesses were encountered up to 12 feet below the surface in the southern portion of the site therefore we understand removing the old fill may not be economical. We have provided recommendations in the site preparation, foundations, and floor slab sections of this report to reduce, but not eliminate, the risk of excessive differential settlement because of the old fill. If the old fill is left in place, the owner must understand and accept the risk associated with supporting structures on the previously placed. As such, ***DDS*** cannot be held responsible for unacceptable building performance as a result of the old fill being left in place.

As will be discussed in later sections of this report, we are recommending that the foundations be supported below the fill and on limestone bedrock. We understand the floor slab will be grade supported. Therefore, we have provided recommendations in the site preparation and floor slab sections of this report to reduce, but not eliminate, the risk of excessive differential settlement of the slab on grade floor due to the old fill.

5.2 KARST TOPOGRAPHY

Surficial evidence of sinkholes or solutioning was not observed during our site visits, however some bedrock discontinuities were encountered within the sampled bedrock. The site is underlain by limestone bedrock of the St. Genevieve Limestone Formation which is a known Karst landscape that is susceptible to solutioning and sinkhole development. It should be understood and accepted by the owner that there is a risk of future ground subsidence when developing in any region where Karst activity is known.

As discussed in the site geology section of this report, the KGS website indicates the site to have a very high potential for Karst activity and maps several sinkholes within about ¼ mile of the site. Sinkholes generally form from the bottom up. During periods of high precipitation, water rises into voids near the top of the limestone bedrock. When the water recedes, wet soil falls into the subsurface voids creating a soil dome. Over time, the soil “dome” reaches the surface where a drop out will occur. All areas underlain by potentially soluble rock such as limestone are at risk due to solutioning and sinkhole activity.

Solution features which are in the process of forming a soil drop-out or sinkhole represent a risk to the project because they are difficult to detect even during construction. To completely explore a site to such an extent as to fully identify the possibility of future Karst related problems is not cost effective. It should be understood and accepted by the owner that there is a risk of future ground subsidence when developing in any region where karst activity is known. To further evaluate the site for potential Karst features, a geophysical survey using electrical resistivity could be performed.

Several methods are available to remediate sinkholes to reduce, but not eliminate, the risk of future damage to structures. Each sinkhole should be evaluated by the geotechnical engineer if remediation is required. If remediation is required, the common practice is to excavate all the soil from within the solution feature to competent bedrock. The most common remediation method is an inverted filter constructed of crushed limestone fill. For the crushed limestone inverted filter procedure, once all the soil and debris has either been removed to the throat of the sinkhole or a specific depth, the bottom and sides of the excavation are lined with a non-woven geotextile fabric. Crushed limestone is then placed in lifts into the excavation with larger rock at the bottom and progressively smaller stone to the top. The fabric is then lapped over of the crushed stone and a clay cap is then placed and compacted on top of the fabric. ***DDS*** should be contacted if a sinkhole is encountered during construction for guidance.

6 RECOMMENDATIONS

Our recommendations are based on the design information provided to us, the data obtained during the previously described exploration, and our experience. They do not reflect variations in the subsurface conditions which may exist between our borings and in unexplored areas of the site. If variations become apparent during construction, it will be necessary for us to re-evaluate our conclusions and recommendations based upon on-site observations.

6.1 SITE PREPARATION RECOMMENDATIONS

The site should be cleared of all pavements, topsoil, and other deleterious material before placement of engineered fill. Site stripping, clearing, and grubbing should extend a minimum of ten feet beyond perimeters of pavements and buildings.

Based on the proposed loading conditions and the previously placed fill encountered, we have recommended that the foundations be supported on limestone bedrock below the old fill. However, we understand the floor will be a grade supported concrete slab. As mentioned in previous sections of this report, previously placed organic laden fill up to about 12 feet below the surface was encountered in some of the borings. Uncontrolled fills are typically removed and replaced with engineered fill due to the inherent risk of differential slab support and detrimental cracking with supporting the floor slabs on the uncontrolled fill. If removing the old fill is not economical, we have recommended the following options to reduce, but not eliminate, the risk. 1) The old fill should be removed a minimum of six feet below the proposed subgrade elevation for the slab on grade floor and backfilled with engineered soil fill. The engineered fill should be placed and compacted per the recommendations in the engineered fill section of this report. 2) The undercut can be reduced if a crushed stone mat comprised of limestone is used as slab support. The crushed stone mat should consist of three feet of shot rock fill and twelve inches of dense graded aggregate. The shot rock fill should be placed and compacted per the shot rock fill recommendations section in this report. The dense graded aggregate should be placed in lifts up to 6 inches thick and compacted with a vibratory smooth drum roller. The aggregate base should be moist, but not wet, as the concrete is placed to reduce curling of the slab as the concrete cures.

The remainder of this site preparation recommendations section considers site preparation where residual soils are encountered after site clearing and grubbing. In areas of the site where residual soils are exposed after site striping, a detailed proofroll should be observed under the direction of the geotechnical engineer prior to fill placement. The proofroll can assist in detection of soft zones and karst features (sinkholes). The proofroll should consist of a heavily loaded tri-axle dump truck traversing the site in two perpendicular directions. Soils that deflect excessively under the loaded vehicle should be undercut to suitable soils or stabilized in place prior to fill placement. Any undercut or remediation should be performed under the direction and in the presence of a representative of the geotechnical engineer.

During and after construction, positive surface gradients should be maintained adjacent to buildings and pavements to direct surface water toward appropriate discharge facilities and away from foundations and slabs. Water must not be allowed to pond adjacent to the structures.

6.2 ENGINEERED SOIL FILL

Typically, soil containing less than three percent of organic material by volume, plasticity index (PI) less than 35, a minimum standard Proctor maximum dry density of 95 pounds per cubic foot, and a maximum particle size of 3 inches can be used as fill. The residual soils sampled at this site meet this criteria and can be used as engineered fill. However, the previously placed fill encountered within the borings should not be used as engineered fill.

Engineered soil fill should be placed in lifts of uniform thickness. The lift thickness should not exceed that which can be properly compacted throughout its entire depth with the equipment available, usually no more than eight inches. We recommend that structural fills supporting pavements, footings, and floor slabs be compacted to 98 percent of the standard Proctor maximum dry density (ASTM D-698). The moisture content of the fill soils will be dependent on the soil type being used. Assuming clay will be used as fill, the moisture content during fill placement should be maintained between about -1 percent to about +3% of its standard Proctor optimum moisture content. Field density testing should be conducted on each lift of structural soil fill. We recommend a technician under the direction of the geotechnical engineer continuously observe fill placement and perform field density test.

The on-site soils exhibit moderate plasticity and are sensitive to changes in moisture content. These soils will pump and rut during wet conditions. 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 grading operations, we recommend that the area be undercut to firm native soils and backfilled with engineered fill (compacted soil or crushed stone) or stabilized in place by the use of chemical stabilization. An alternative to wasting the wet clay soils is to temporarily stockpile this material for aeration and placement during dryer conditions. The contractor should be prepared to control water inflow into the project area by appropriate means. Groundwater seeps/flows are also commonly encountered near the soil/bedrock interface. Temporary drainage ditches or sump areas may be required to control any encountered water inflow.

6.3 SHOT ROCK FILL

Properly graded shot rock from blasted rock is generally suitable for use as engineered fill provided it is competent limestone. Shale must not be used as structural fill. If shot rock is planned to be used as fill, it should have a maximum particle size of 18 inches and have sufficient fines to fill the void space between rock contact points but no more than 20% fines. The shot rock fill should have good rock to rock contact and void of clay pockets. If rock to rock contact is not achieved, detrimental structural settlement can occur. If used, we recommend that the shot rock be limited to at least 1 foot below the top of the slab subgrade.

If used, shot rock should be placed in lifts of uniform thickness no thicker than 18 inches and compacted with repeated passes with a D-8 or larger dozer. Since accurate field density tests cannot be obtained from shot rock, each lift of compacted fill should be observed by a technician under the direction of the geotechnical engineer.

6.4 Foundations and Foundation Settlement

Foundation and foundation settlement recommendations provided below are based upon empirical correlations from SPT N values, laboratory test data, our past experience, and assuming the site is prepared in accordance with our recommendations.

Column loads are assumed to be 300 kips or less and settlement tolerances are assumed to be about ¾ inches between columns and 1 inch total. Foundations bearing on the soil fill at the site will result in excessive settlement. Therefore, based on anticipated loading conditions and the previously placed fill encountered, we recommend the building be supported on the limestone bedrock encountered in the borings. Depending on the bearing capacity needed, the foundations can be supported on shallow spread footings, drilled shafts or a combination of drilled shafts and shallow spread footings.

6.4.1 Bedrock Supported Shallow Spread Footings

Foundations bearing on bedrock supported shallow spread footings should be designed for a net allowable bearing pressure of 5,000 pounds per square foot. This allowable bearing pressure is based upon soil excavations to "backhoe refusal." Pneumatic hammering (hoe ram) of the upper zones might be required to remove irregular bedrock surfaces such as pinnacles. If over excavation is required to reach competent rock, the excavation can be backfilled with lean concrete to the planned bottom of footing. However, before placement of structural concrete or lean concrete backfill, the bottom of excavation should be cleaned and should be observed by a representative of the geotechnical engineer.

If higher bearing capacities are needed to support the structure, the bedrock should be excavated beyond weathered and fractured rock at the soil rock interface. If rock is removed beyond weathered and fractured rock, a net allowable bearing pressure of 10,000 pounds per square foot can be allowed. If the higher bearing capacity is used for design, the geotechnical engineer should evaluate all bedrock excavations and be provided probe holes drilled into bedrock to evaluate conditions. At the least, pneumatic hammers can be anticipated for rock excavation.

If the required bearing capacity of 10,000 pounds per square foot bearing capacity is needed, 1 test hole for every standard isolated spread footing and 1 every 40 foot of continuous wall or strip footing should be drilled in the bottom of each excavated footing to determine if seams/voids are present in the rock beneath the bearing surface. The test holes should be a minimum of 2 inches in diameter and 4 feet deep. Additional rock removal will be at the discretion of the geotechnical engineer. If bedrock is encountered below the planned bottom of footing, lean concrete may be placed on the top of clean bedrock to the bottom of planned bottom of footing elevation.

6.4.1.1 GENERAL FOUNDATION RECOMMENDATIONS

All exterior soil supported foundations should bear below the frost line of the locality to protect against frost heave. In Warren County, the footing depth should be at least 24 inches below finished grade. Also, all foundations should have a minimum width of twenty-four inches to facilitate excavation, cleaning of the excavation, placement of reinforcing steel, and to reduce the risk of differential settlement. It is common that during construction, some loose soil remains in the bottom of the footing excavation. This loose soil should be carefully removed immediately before concrete placement.

All footing excavations should be evaluated by the geotechnical engineer before placement of reinforcing steel and concrete to verify that field conditions are consistent with those anticipated as a result of our subsurface exploration. Every effort should be made to place the foundation concrete the same day the excavations are complete.

6.4.2 DRILLED SHAFTS

If drilled shafts are used for foundation support, we recommend that drilled shafts be designed for a net allowable bearing pressure of 80 kips per square foot (ksf) and supported on competent unweathered limestone. Some discontinuities were observed within the bedrock as summarized in Table 3 of this report. It is critical that the bearing material be evaluated by a representative of geotechnical engineer during construction and prior to the placement of concrete. For planning and budgeting purposes, a minimum rock socket of 1 foot should be expected. We recommend that the geotechnical engineer's representative be provided with the actual bearing pressure at each column location. If a lower bearing pressure is planned at a particular column, it may be possible to reduce the inspection criteria and/or limit the amount of rock excavation. Please refer to the drilled shaft quality control section of this report for recommendations regarding shaft preparation, inspection, and concrete placement.

Drilled shafts should have a minimum diameter of 36 inches and should have a minimum length of two times the diameter. Based on voids and seams encountered in the borings and the pinnacled and slotted nature of the limestone bedrock in this area, the contractor should be prepared for full diameter rock coring. We anticipate that the weathered zone will vary across the site, however our borings primarily encountered the weathered zone, voids, and soil filled seams within the upper 5 feet of the limestone. For planning and budgeting purposes, a minimum rock socket of 1 foot should be expected. If excessive rock removal is encountered in the field, we can re-calculate the drilled shaft capacity using friction between the drilled shaft concrete and the bedrock socket. However, at this time, the entire drilled shaft capacity is from end bearing.

6.4.2.1 DRILLED SHAFT QUALITY CONTROL

The recommended rock bearing capacity provided assumes that each drilled shaft excavation will be evaluated by a representative of the geotechnical engineer prior to the placement of concrete. Historically, down hole inspections were performed inside a cased excavation to observe the top of rock surface and to perform test holes to check for bedrock discontinuities. Due to safety concerns, down hole entry requires steel casing, air quality monitoring, and other OSHA requirements for confined space work procedures. As an alternative to down hole inspections, evaluation of drilled shaft foundations can be performed by pre-drilling the column locations prior to foundation installation. The predrilling is performed by sampling the bedrock by rock coring methods at each column location. With this method, the length of the bedrock sampled should be two times the diameter of the shaft plus two feet below the planned bottom of shaft. The recovered bedrock is then analyzed, and a plan bottom of shaft is assigned at each location.

When concreting, the steel casing should be removed at a slower rate than the concrete is placed to prevent soil and water from entering the excavation. In addition, we recommend the concrete be specified with a 6 +/- 1 inch slump to fill all irregular areas and allow for proper consolidation around reinforcing steel. During construction, the drilled shaft should be excavated beyond weathered limestone and be free of all loose material, mud, and water at the bearing surface. Groundwater encountered during drilled shaft excavations should be controlled by conventional pump methods. If water enters the drilled shaft excavation, pumps should be used to maintain no more than 2 inches of water in bottom of the excavation to facilitate cleaning and allow for proper inspections. The pumps should be left in place until immediately prior to concreting. Prior to concreting, if more than four inches of standing water is observed at the bottom of excavation, a tremie pipe should be used for concrete placement.

A representative of the geotechnical engineer should evaluate drilled shaft construction to verify that field conditions are consistent with those anticipated as a result of our subsurface exploration. If we are not allowed to verify the subsurface conditions, we cannot take responsibility for the determination of the adequacy of the subsurface bedrock nor their ability to support the proposed structure. Also, every effort should be made to place the foundation concrete the same day the excavations are done. If this is not done, the excavations can be disturbed by rain, sloughing of the sides of the excavation, and excessive drying. If leaving the excavation open for more than a day is unavoidable, it is recommended that the foundation be observed by a representative of our office before the placement of concrete.

6.5 SEISMIC CONSIDERATIONS

Seismic design considerations and recommendations are based on the 2018 International Building Code and Chapter 20 of ASCE 7. Based on data obtained during our exploration, we recommend this site be assigned as a “Site Class C”. A site specific seismic study or hazard analysis was beyond the scope of this investigation.

6.6 FLOOR SLABS

We understand the floor will be a grade supported concrete slab. Floor loading conditions are unknown at the time of this report’s completion however we have assumed floor loads of about 100 psf. Up to about 12 feet of previously placed fill was encountered in the borings as mentioned in previous sections. Uncontrolled fills are typically removed and replaced with engineered fill due to the inherent risk of differential slab support and detrimental cracking with supporting the floor slabs on the uncontrolled fill. We provided recommendations in the Site Preparation section of this report to reduce, but not eliminate, the risk of leaving some of the old fill in place. The remainder of this section assumes site preparation is performed per our recommendations.

We recommend the floor slabs be "floating", fully ground supported and structurally independent of any building footings or walls. Slabs should be appropriately reinforced to support the proposed loads with an appropriate number of control joints. These recommendations will help to reduce the possibility of cracking and displacement of the floor slabs due to differential settlement. A vapor retarder beneath concrete slabs on grade should be used for slabs that will be covered by moisture sensitive material.

The floor slabs for buildings should be supported on a minimum of a 6 inch thick compacted layer of granular subbase material. The purpose of this layer is to help distribute concentrated loads and equalize moisture conditions beneath the slab. The crushed stone should be moist, but not wet, immediately prior to placement of concrete to reduce curling of the slab.

Floor slab subgrades are often disturbed by weather, utility installation, and construction traffic between completion of grading and slab placement. Therefore, before slab concrete placement, a thorough proofroll should be observed by the geotechnical engineer. The proofroll should be performed with a heavy rubber tired vehicle such as a loaded dump truck. Any areas deemed unacceptable to the engineer should be undercut and replaced with engineered fill or crushed stone in accordance the geotechnical engineer's recommendations.

6.7 EARTH SLOPES

A site plan was not provided at the time of this report's completion and a detailed slope stability analysis for any temporary or permanent conditions was beyond the scope of this report. However, we recommend that permanent fill slopes be constructed no steeper than 3H:1V. Fill slopes should be over built and cut back to the planned slope geometry.

The contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of the sides and bottom of the excavations. All excavations should be performed in accordance with Subpart P of OSHA Standards for the Construction Industry (29 CFR Part 1926 Subpart P).

7 Observation and Monitoring During Construction

We recommend that site preparation and foundations be monitored to verify that construction is performed in conformance with project specifications and recommendations provided in this report. We recommend that ***DDS*** be retained to provide these services. ***DDS*** routinely provides materials testing services and has an accredited testing laboratory. This will allow us to observe that construction is completed per our recommendations and that any changes necessary can be completed in a timely manner in the best interest of the owner. We cannot be responsible for interpretation of the data contained herein by others

8 Limitations of Recommendations

The conclusions and recommendations presented herein are based on our observations, data obtained from the subsurface exploration, and experience using the degree of care and skill ordinarily exercised under similar circumstances by competent members of the engineering profession. No guarantees can be made regarding the continuity of conditions between borings. The soil and rock conditions at other locations on the site may differ from those reflected by the boring locations referenced in this report. This office is not responsible for the conclusions, opinions, or recommendations of others based on the information contained in this report.

Our scope of services did not include any environmental assessment of site conditions including the presence of wetlands or hazardous or toxic material present in the subsoil or bedrock. Any references to odors, colors, or irregular conditions are from visual observations and informational purposes only.

Any revision in the plans for any of the proposed structures should be brought to the attention of the geotechnical engineer so that we may determine whether any changes in the foundation recommendations are necessary or additional exploration is required. If deviations from the noted subsurface conditions are encountered during the construction, they should also be brought to the attention of the geotechnical engineer. When the final design is complete, our office should be given the opportunity to provide the additional service of reviewing the foundation plan, grading plan, and other portions of the project impacted by this report. This review will allow us to check whether these documents are consistent with our recommendations.

We recommend that the owner hire our firm to perform all observation, testing, and inspection services related to recommendations contained in this report. This will allow us to observe that site preparation, engineered fill, and foundation construction is completed per our recommendations and that any changes necessary can be completed in a timely manner in the best interest of the owner. We cannot be responsible for interpretation of the data contained herein by others.

APPENDIX

SITE VICINITY EXHIBIT
AERIAL PHOTOGRAPH EXHIBIT
KARST POTENTIAL EXHIBIT
BORING LOCATION EXHIBITS
BORING RECORDS
SUMMARY OF LABORATORY TEST RESULTS
LABORATORY TEST DATA SHEETS
SITE PHOTOGRAPHS
HISTORICAL AERIAL PHOTOGRAPHS
FIELD TESTING PROCEDURES
LABORATORY TESTING PROCEDURES
INFORMATION ABOUT YOUR GEOTECHNICAL REPORT

PREPARED BY

DDS
49 YEARS

DECEMBER 15, 2022

148 CHESTER COURT • BOWLING GREEN, KENTUCKY 42103 • PHONE (270) 843-2247 • FAX (270) 843-9323
www.ddsengineering.com

SHEET
SITE VICINITY MAP

JOB NUMBER	22-5600
DATE	11/29/2022
SCALE	NTS
REFERENCE:	
REFERENCED FROM KYTC OFFICIAL HIGHWAY MAPS	

PROJECT

WKU HILLTOPPER FIELDHOUSE
WKU - AVENUE OF CHAMPIONS
BOWLING GREEN, KENTUCKY

CLIENT

K. NORMAN BERRY ARCHITECTS
815 WEST MARKET STREET, SUITE 502
LEXINGTON, KENTUCKY 40202

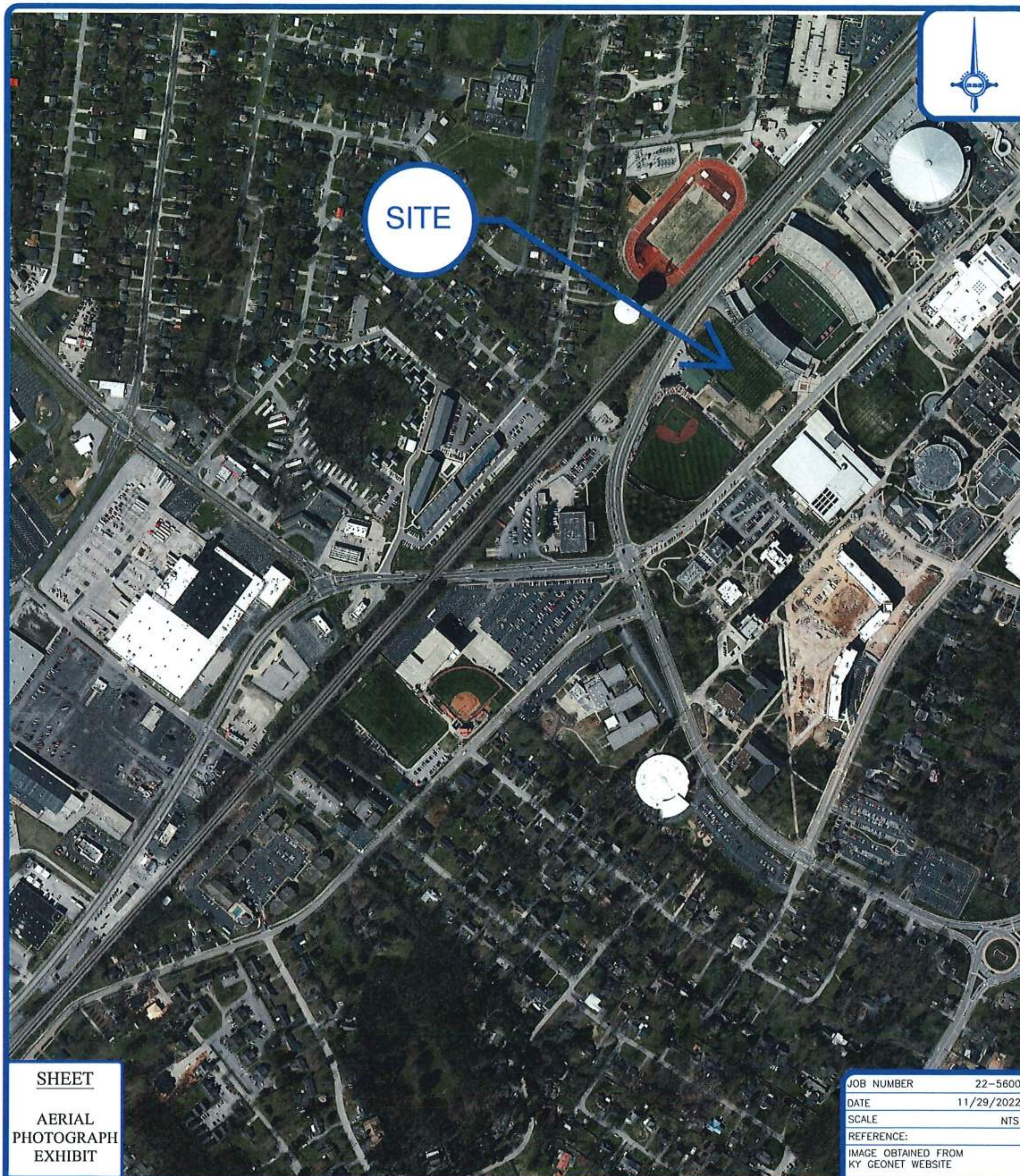
DDS 49 YEARS

DDS ENGINEERING, PLLC

LAND SURVEYING, CIVIL AND GEOTECHNICAL ENGINEERING, CONSTRUCTION MATERIALS TESTING, AND SPECIAL INSPECTION

148 CHESTER COURT
BOWLING GREEN, KY 42103

270-843-2247
WWW.DDSENGINEERING.COM



SHEET

AERIAL PHOTOGRAPH EXHIBIT

JOB NUMBER	22-5600
DATE	11/29/2022
SCALE	NTS
REFERENCE:	
IMAGE OBTAINED FROM KY GEONET WEBSITE	

PROJECT

WKU HILLTOPPER FIELDHOUSE
WKU - AVENUE OF CHAMPIONS
BOWLING GREEN, KENTUCKY

CLIENT

K. NORMAN BERRY ARCHITECTS
815 WEST MARKET STREET, SUITE 502
LEXINGTON, KENTUCKY 40202

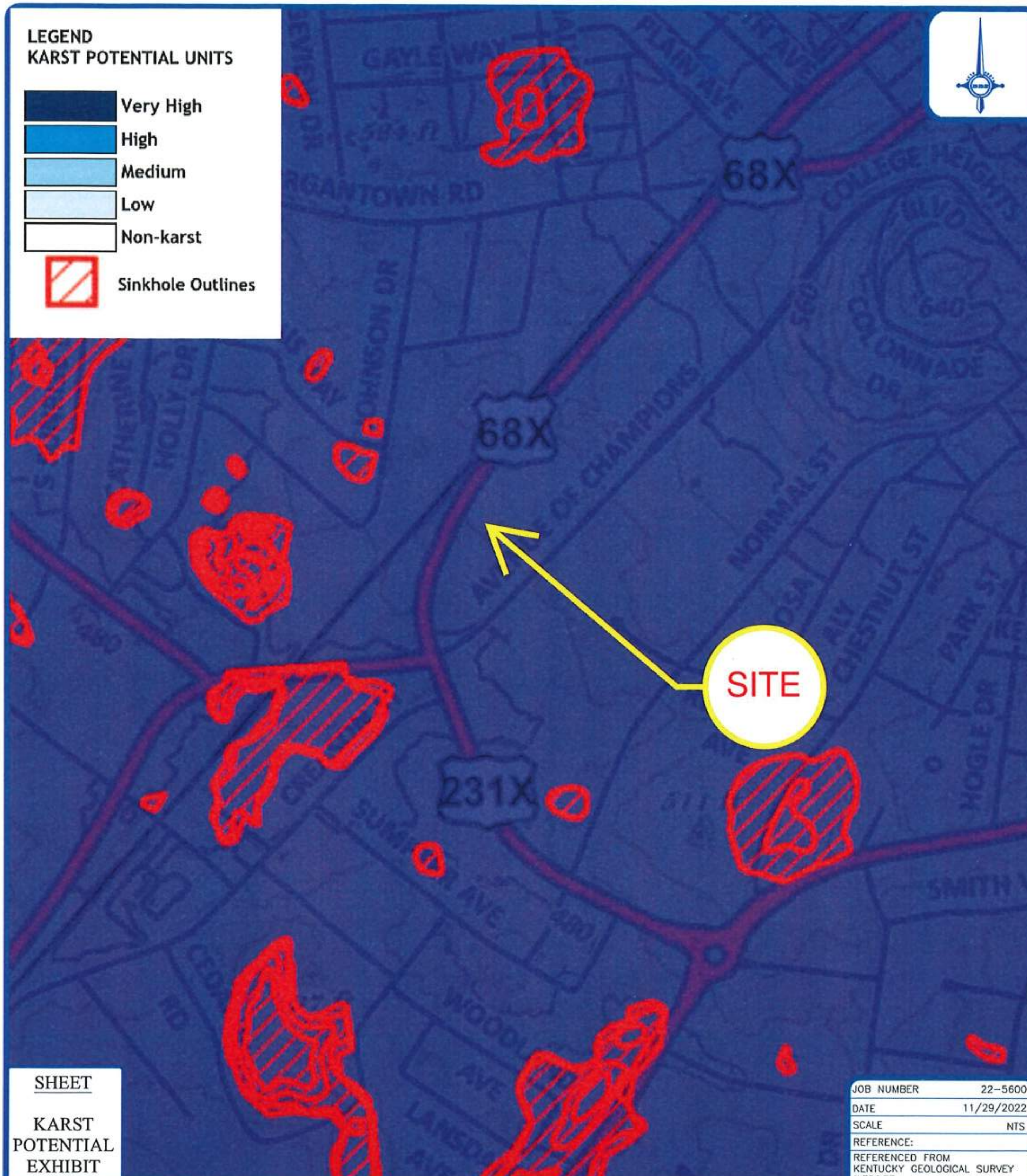
49 YEARS

DDS ENGINEERING, PLLC

LAND SURVEYING, CIVIL AND GEOTECHNICAL ENGINEERING, CONSTRUCTION MATERIALS TESTING, AND SPECIAL INSPECTION

148 CHESTER COURT
BOWLING GREEN, KY 42103

270-843-2247
WWW.DDSENGINEERING.COM



SHEET

KARST POTENTIAL EXHIBIT

JOB NUMBER	22-5600
DATE	11/29/2022
SCALE	NTS
REFERENCE:	
REFERENCED FROM KENTUCKY GEOLOGICAL SURVEY WEBSITE	

PROJECT

WKU HILLTOPPER FIELDHOUSE
WKU - AVENUE OF CHAMPIONS
BOWLING GREEN, KENTUCKY

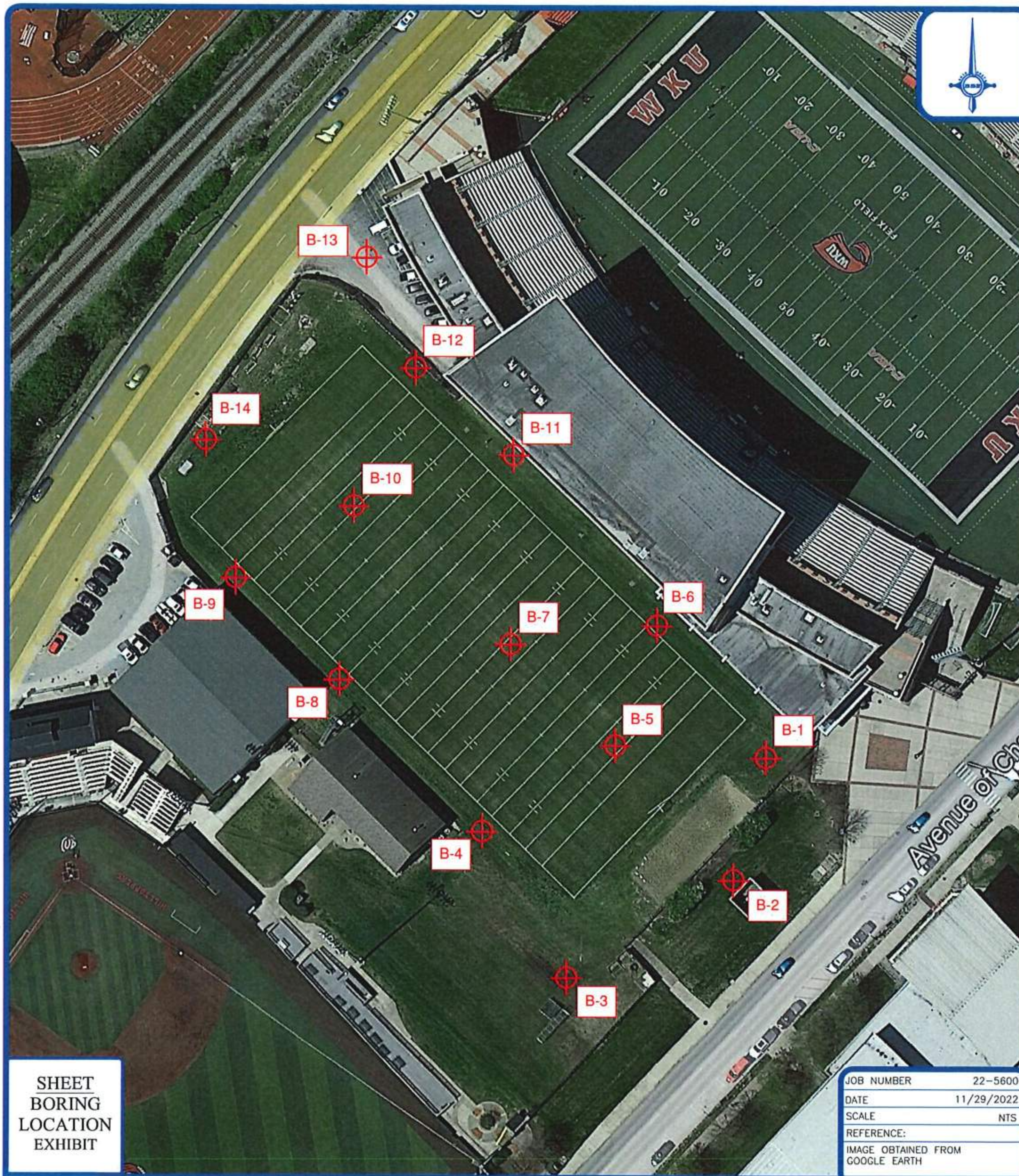
CLIENT

K. NORMAN BERRY ARCHITECTS
815 WEST MARKET STREET, SUITE 502
LEXINGTON, KENTUCKY 40202

DDS ENGINEERING, PLLC
LAND SURVEYING, CIVIL AND GEOTECHNICAL ENGINEERING, CONSTRUCTION MATERIALS TESTING, AND SPECIAL INSPECTION

148 CHESTER COURT
BOWLING GREEN, KY 42103

270-843-2247
WWW.DDSENGINEERING.COM



SHEET
BORING LOCATION EXHIBIT

JOB NUMBER	22-5600
DATE	11/29/2022
SCALE	NTS
REFERENCE:	
IMAGE OBTAINED FROM GOOGLE EARTH	

PROJECT

WKU HILLTOPPER FIELDHOUSE
WKU - AVENUE OF CHAMPIONS
BOWLING GREEN, KENTUCKY

CLIENT

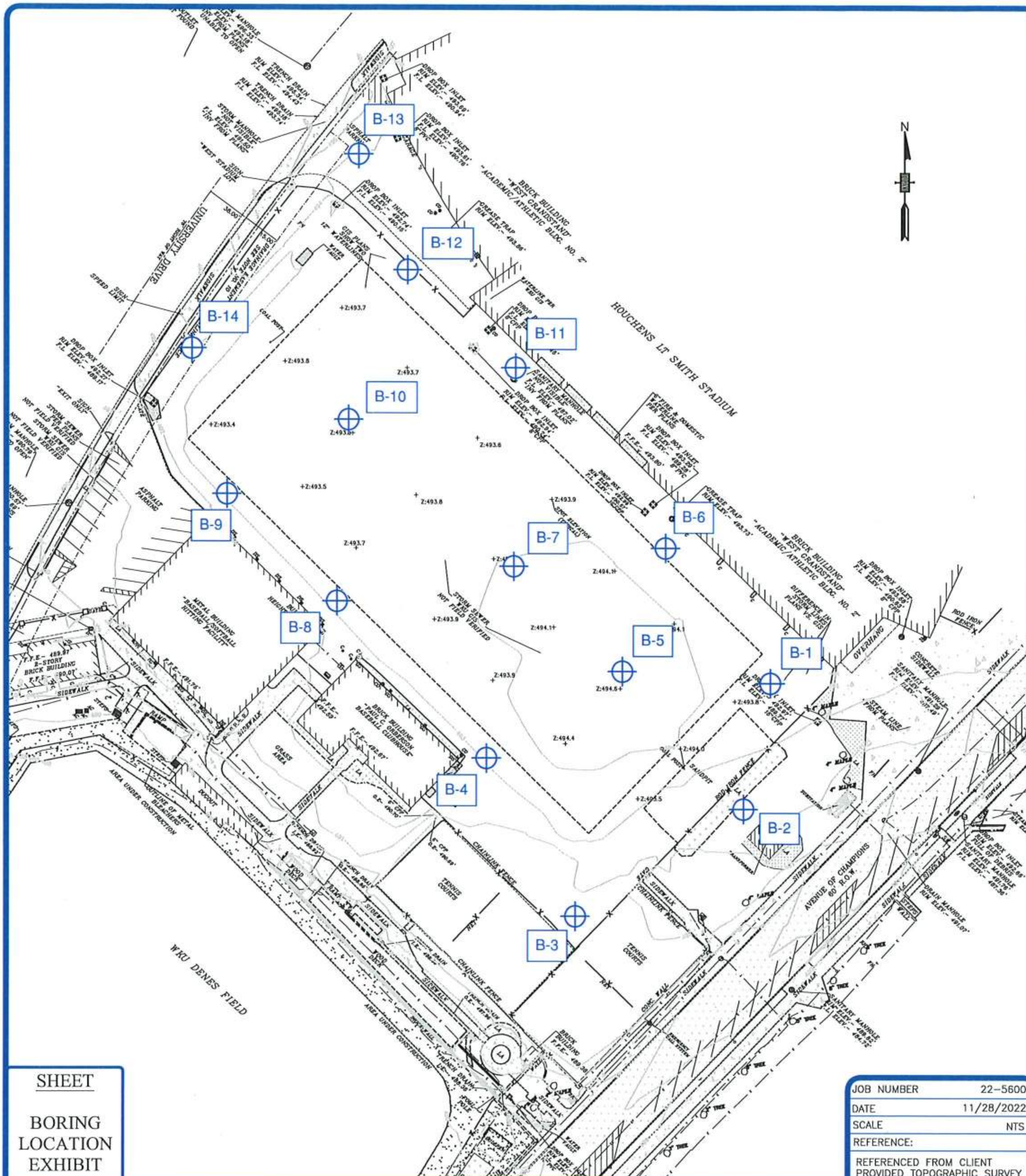
K. NORMAN BERRY ARCHITECTS
815 WEST MARKET STREET, SUITE 502
LEXINGTON, KENTUCKY 40202

DDS ENGINEERING, PLLC

LAND SURVEYING, CIVIL AND GEOTECHNICAL ENGINEERING, CONSTRUCTION MATERIALS TESTING, AND SPECIAL INSPECTION

148 CHESTER COURT
BOWLING GREEN, KY 42103

270-843-2247
WWW.DDSENGINEERING.COM



SHEET

BORING LOCATION EXHIBIT

JOB NUMBER	22-5600
DATE	11/28/2022
SCALE	NTS
REFERENCE:	
REFERENCED FROM CLIENT PROVIDED TOPOGRAPHIC SURVEY	

PROJECT

WKU HILLTOPPER FIELDHOUSE
WKU - AVENUE OF CHAMPIONS
BOWLING GREEN, KENTUCKY

CLIENT

K. NORMAN BERRY ARCHITECTS
815 WEST MARKET STREET, SUITE 502
LEXINGTON, KENTUCKY 40202

DDS ENGINEERING, PLLC
LAND SURVEYING, CIVIL AND GEOTECHNICAL ENGINEERING, CONSTRUCTION MATERIALS TESTING, AND SPECIAL INSPECTION

49 YEARS

148 CHESTER COURT
BOWLING GREEN, KY 42103

270-843-2247
WWW.DDSENGINEERING.COM

DDS ENGINEERING, PLLC
148 Chester Court
Bowling Green KY 42103
Telephone: 270-843-2247
Fax: 270-843-9323

BORING NUMBER B-1

PROJECT NAME WKU HILLTOPPER FIELDHOUSE **CLIENT** K NORMAN BERRY ARCHITECTS
PROJECT NUMBER T5600G **PROJECT LOCATION** BOWLING GREEN, KY
DATE STARTED 11/2/2022 **COMPLETED** 11/2/2022 **GROUND ELEVATION** 493.5 ft **WEATHER** 50-60, DRIZZLE
DRILLING CONTRACTOR HORN **RIG TYPE** DIEDRICH D-50 **GROUND WATER LEVELS:**
DRILLING METHOD 4 1/4" ID HSA **HAMMER** AUTOMATIC **AT TIME OF DRILLING** DRY
LOGGED BY MHR **CHECKED BY** MHR **AT END OF DRILLING** ---
NOTES **AFTER DRILLING** ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	MOISTURE CONTENT (%)	ATTERBERG LIMITS			ROCK UNCONFINED COMP. STRENGTH (KSF)
							LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	
0		TOPSOIL - 6 INCHES								
		FILL - STIFF, CLAY AND SOME CRUSHED LIMESTONE, BROWN	SPT	33	11-18-15 (33)					
		STIFF, CLAY, BROWNISH ORANGE	SPT	100	5-8-50/1"					

AUGER REFUSAL AT 3.7 FEET
BORING TERMINATED AT AUGER REFUSAL

GEOTECH BH MODIFIED W/ROCK UC - TESTING TEMPLATE.GDT - 12/1/22 15:45 - \\DDS-APP\SHARES\T\PROJECTS\5600 WKU FIELDHOUSE\05 - GEOTECHNICAL\5600 WKU FIELDHOUSE.GPJ

DDS ENGINEERING, PLLC
148 Chester Court
Bowling Green KY 42103
Telephone: 270-843-2247
Fax: 270-843-9323

BORING NUMBER B-2

PROJECT NAME WKU HILLTOPPER FIELDHOUSE
CLIENT K NORMAN BERRY ARCHITECTS
PROJECT NUMBER T5600G
PROJECT LOCATION BOWLING GREEN, KY
DATE STARTED 11/3/2022 **COMPLETED** 11/3/2022
GROUND ELEVATION 492.3 ft **WEATHER** 50-60, SUNNY
DRILLING CONTRACTOR HORN **RIG TYPE** DIEDRICH D-50
GROUND WATER LEVELS:
DRILLING METHOD HSA, NX **HAMMER** AUTOMATIC
AT TIME OF DRILLING DRY AFTER SOIL AUGERING
LOGGED BY MHR **CHECKED BY** MHR
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	ROCK UNCONFINED COMP. STRENGTH (KSF)
0	TOPSOIL - 10 INCHES								
	FILL, STIFF, CLAY SOME CRUSHED LIMESTONE, TRACE OF BRICK FRAGMENTS, DRY CLAY FILL - BROWN TO BLACK, ORGANIC LADEN	SPT	100	11-11-10 (21)					
		SPT	100	7-6-6 (12)	12.0				
5		SPT	100	5-4-7 (11)					
	STIFF, LEAN CLAY, ORANGISH TO REDDISH BROWN, SOME CHERT HARDER DRILLING T 8.2 FEET	SPT	100	3-4-5 (9)	21.2	42	18	24	
10	AUGER REFUSAL AT 9.8 FEET, BEGAN CORING	SPT	100	50/3"					
	LIMESTONE, MODERATELY HARD TO HARD, GRAY TO LIGHT GRAY WATER LOSS AT 10.4 FEET	RC	92 (65)						1930 3215
	VOID / SOIL SEAM								
15	LIMESTONE, MODERATELY HARD TO HARD, GRAY TO LIGHT GRAY								
	VOID / SOIL SEAM (16.6' - 17.0')								
	LIMESTONE, MODERATELY HARD TO HARD, GRAY TO LIGHT GRAY								
20									
	LIMESTONE, MODERATELY TO MEDIUM HARD, DARK GRAY, SOME CALCITE	RC	93 (83)						1950
25									
	VOID / SOIL SEAM (26.3' - 26.5')								
	LIMESTONE, MODERATELY HARD, LIGHT GRAY TO WHITE								2225

BORING TERMINATED AT 29.8 FEET

DDS ENGINEERING, PLLC
148 Chester Court
Bowling Green KY 42103
Telephone: 270-843-2247
Fax: 270-843-9323

BORING NUMBER B-3

PROJECT NAME WKU HILLTOPPER FIELDHOUSE
CLIENT K NORMAN BERRY ARCHITECTS
PROJECT NUMBER T5600G
PROJECT LOCATION BOWLING GREEN, KY
DATE STARTED 11/2/2022 **COMPLETED** 11/3/2022
GROUND ELEVATION 491.5 ft **WEATHER** 60-70, SUNNY
DRILLING CONTRACTOR HORN **RIG TYPE** DIEDRICH D-50
GROUND WATER LEVELS:
DRILLING METHOD HSA, NX **HAMMER** AUTOMATIC
AT TIME OF DRILLING DRY AFTER SOIL AUGERING
LOGGED BY MHR **CHECKED BY** MHR
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	ROCK UNCONFINED COMP. STRENGTH (KSF)
0	TOPSOIL - 8 INCHES								
	(FILL) CLAY, SOME CRUSHED STONE, SOME GLASS AND BRICK FRAGMENTS, STIFF, BROWN CLAY FILL - HEAVILY ORGANIC LADEN,	SPT	94	9-12-12 (24)					
		SPT	100	10-13-12 (25)	14.5				
5	(FILL) ORGANIC LADEN CLAY AND SILT, BLACK TO DARK BROWN, MOIST, FIRM	SPT	83	2-3-4 (7)					
		SPT	100	3-2-3 (5)	21.7	26	18	8	
10	(FILL) ORGANIC LADEN CLAY AND SILT, OLIVE GREEN TO DARK GRAY, SOFT, WET	SPT	100	2-3-14 (17)					
	WEATHERED LIMESTONE								
15	AUGER REFUSAL AT 12.5 FEET, BEGAN CORING LIMESTONE, MODERATELY HARD TO HARD, GRAY TO LIGHT GRAY, FINE GRAINED LOST WATER 16.8 FEET	RC	96 (62)						3050
	VOID / SOIL FILLED SEAM (16.8' - 17.1') LIMESTONE, MODERATELY HARD TO HARD, LIGHT GRAY								
20	VOID / SOIL FILLED SEAM (20.5' - 20.7') LIMESTONE, MODERATELY HARD TO HARD, LIGHT GRAY								3090
25	LIMESTONE, MODERATE TO MEDIUM HARD, DARK GRAY TO GRAY, SOME CALCITE	RC	99 (84)						2929
30	LIMESTONE, HARD TO MODERATELY HARD, LIGHT GRAY TO WHITE								

BORING TERMINATED AT 32.0 FEET

DDS ENGINEERING, PLLC
148 Chester Court
Bowling Green KY 42103
Telephone: 270-843-2247
Fax: 270-843-9323

BORING NUMBER B-4

PROJECT NAME WKU HILLTOPPER FIELDHOUSE **CLIENT** K NORMAN BERRY ARCHITECTS

PROJECT NUMBER T5600G **PROJECT LOCATION** BOWLING GREEN, KY

DATE STARTED 11/2/2022 **COMPLETED** 11/2/2022 **GROUND ELEVATION** 493.0 ft **WEATHER** 50-60, DRIZZLE

DRILLING CONTRACTOR HORN **RIG TYPE** DIEDRICH D-50 **GROUND WATER LEVELS:**

DRILLING METHOD 4 1/4" ID HSA **HAMMER** AUTOMATIC **AT TIME OF DRILLING** DRY

LOGGED BY MHR **CHECKED BY** MHR **AT END OF DRILLING** ---

NOTES **AFTER DRILLING** ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	ROCK UNCONFINED COMP. STRENGTH (KSF)
0		TOPSOIL - 6 INCHES								
		(FILL) SAND, DARK BROWN	SPT	100	2-3-5 (8)					
		(FILL) CLAY AND CRUSHED STONE, DARK BROWN								
		(FILL) CLAY, FIRM TO STIFF, DARK BROWN TO GRAY, SOME BLACK, ORGANIC LADEN, WOOD FRAGMENTS	SPT	100	3-4-7 (11)	22.6				
5			UD	100						
			SPT	89	4-4-6 (10)		51	23	28	
			SPT	89	3-6-50/4"					
		AUGER REFUSAL AT 9.2 FEET BORING TERMINATED AT AUGER REFUSAL								

DDS ENGINEERING, PLLC
148 Chester Court
Bowling Green KY 42103
Telephone: 270-843-2247
Fax: 270-843-9323

BORING NUMBER B-5

PROJECT NAME WKU HILLTOPPER FIELDHOUSE
CLIENT K NORMAN BERRY ARCHITECTS
PROJECT NUMBER T5600G
PROJECT LOCATION BOWLING GREEN, KY
DATE STARTED 11/2/2022 **COMPLETED** 11/2/2022
GROUND ELEVATION 494.1 ft **WEATHER** 60-70, CLOUDY
DRILLING CONTRACTOR HORN **RIG TYPE** DIEDRICH D-50
GROUND WATER LEVELS:
DRILLING METHOD 4 1/4" ID HSA **HAMMER** AUTOMATIC
AT TIME OF DRILLING DRY
LOGGED BY MHR **CHECKED BY** MHR
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	ROCK UNCONFINED COMP. STRENGTH (KSF)
0		TOPSOIL - 6 INCHES								
		(FILL) - SAND, DARK BROWN	SPT	100	2-3-12 (15)					
		(FILL) - CLAY, SOME CRUSHED LIMESTONE CLAY FILL - ORGANIC LADEN, MOIST	SPT	100	4-6-8 (14)	17.5				
5		(FILL) CLAY, BLACK TO GRAY, ORGANIC LADEN	SPT	100	6-6-8 (14)	20.4				
		WEATHERED LIMESTONE								

AUGER REFUSAL AT 6.7 FEET
BORING TERMINATED AT AUGER REFUSAL

DDS ENGINEERING, PLLC
148 Chester Court
Bowling Green KY 42103
Telephone: 270-843-2247
Fax: 270-843-9323

BORING NUMBER B-6

PROJECT NAME WKU HILLTOPPER FIELDHOUSE
CLIENT K NORMAN BERRY ARCHITECTS
PROJECT NUMBER T5600G
PROJECT LOCATION BOWLING GREEN, KY
DATE STARTED 11/2/2022 **COMPLETED** 11/2/2022
GROUND ELEVATION 493.4 ft **WEATHER** 50-60, DRIZZLE
DRILLING CONTRACTOR HORN **RIG TYPE** DIEDRICH D-50
GROUND WATER LEVELS:
DRILLING METHOD 4 1/4" ID HSA **HAMMER** AUTOMATIC
AT TIME OF DRILLING DRY
LOGGED BY MHR **CHECKED BY** MHR
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	ROCK UNCONFINED COMP. STRENGTH (KSF)
0		TOPSOIL - 7 INCHES								
		(FILL) ORGANIC LADEN CLAY, SOME CRUSHED LIMESTONE	SPT	100	4-5-8 (13)					
		STIFF, LEAN CLAY, ORANGISH BROWN								
			SPT	100	4-13-50/0"					

AUGER REFUSAL AT 3.7 FEET
BORING TERMINATED AT AUGER REFUSAL

GEOTECH BH MODIFIED W/ROCK UC - TESTING TEMPLATE.GDT - 12/1/22 15:45 - \\DDS-APP\SHARES\T\PROJECTS\5600 WKU FIELDHOUSE\05 - GEOTECHNICAL\5600 WKU FIELDHOUSE.GPJ

DDS ENGINEERING, PLLC
148 Chester Court
Bowling Green KY 42103
Telephone: 270-843-2247
Fax: 270-843-9323

BORING NUMBER B-7

PROJECT NAME WKU HILLTOPPER FIELDHOUSE
CLIENT K NORMAN BERRY ARCHITECTS
PROJECT NUMBER T5600G
PROJECT LOCATION BOWLING GREEN, KY
DATE STARTED 11/2/2022 **COMPLETED** 11/2/2022
GROUND ELEVATION 494.0 ft **WEATHER** 60-70, CLOUDY
DRILLING CONTRACTOR HORN **RIG TYPE** DIEDRICH D-50
GROUND WATER LEVELS:
DRILLING METHOD 4 1/4" ID HSA **HAMMER** AUTOMATIC
AT TIME OF DRILLING DRY
LOGGED BY MHR **CHECKED BY** MHR
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	ROCK UNCONFINED COMP. STRENGTH (KSF)
0		TOPSOIL - 4 INCHES								
		(FILL) - SAND, DARK BROWN	SPT	100	2-7-5 (12)					
		(FILL) - ORGANIC LADEN CLAY, SOME CRUSHED LIMESTONE, DARK BROWN TO BLACK, SOFT	SPT	100	2-4-4 (8)					
5			SPT	180	8-50/2"/4"					

AUGER REFUSAL AT 5.4 FEET
BORING TERMINATED AT AUGER REFUSAL

DDS ENGINEERING, PLLC
148 Chester Court
Bowling Green KY 42103
Telephone: 270-843-2247
Fax: 270-843-9323

BORING NUMBER B-8

PROJECT NAME WKU HILLTOPPER FIELDHOUSE
CLIENT K NORMAN BERRY ARCHITECTS
PROJECT NUMBER T5600G
PROJECT LOCATION BOWLING GREEN, KY
DATE STARTED 11/2/2022 **COMPLETED** 11/2/2022
GROUND ELEVATION 493.3 ft **WEATHER** 50-60, CLOUDY
DRILLING CONTRACTOR HORN **RIG TYPE** DIEDRICH D-50
GROUND WATER LEVELS:
DRILLING METHOD 4 1/4" ID HSA **HAMMER** AUTOMATIC
AT TIME OF DRILLING DRY
LOGGED BY MHR **CHECKED BY** MHR
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	ROCK UNCONFINED COMP. STRENGTH (KSF)
0		TOPSOIL - 6 INCHES								
		(FILL) SAND, DARK BROWN	SPT	89	5-5-7 (12)					
		(FILL) DENSE GRADED AGGREGATE								
		(FILL) ORGANIC LADEN CLAY, BROWN TO BLACK	SPT	100	3-5-6 (11)					
5		STIFF, LEAN CLAY, BROWNISH TO ORANGISH RED, SOME CHERT	SPT	100	2-4-7 (11)	23.0	44	19	25	
		WEATHERED LIMESTONE	SPT	100	4-50/3"/3"					

AUGER REFUSAL AT 7.7 FEET
BORING TERMINATED AT AUGER REFUSAL

DDS ENGINEERING, PLLC
148 Chester Court
Bowling Green KY 42103
Telephone: 270-843-2247
Fax: 270-843-9323

BORING NUMBER B-9

PROJECT NAME WKU HILLTOPPER FIELDHOUSE | **CLIENT** K NORMAN BERRY ARCHITECTS

PROJECT NUMBER T5600G | **PROJECT LOCATION** BOWLING GREEN, KY

DATE STARTED 11/2/2022 **COMPLETED** 11/2/2022 | **GROUND ELEVATION** 493.3 ft **WEATHER** 50-60, CLOUDY

DRILLING CONTRACTOR HORN **RIG TYPE** DIEDRICH D-50 | **GROUND WATER LEVELS:**

DRILLING METHOD 4 1/4" ID HSA **HAMMER** AUTOMATIC | **AT TIME OF DRILLING** DRY

LOGGED BY MHR **CHECKED BY** MHR | **AT END OF DRILLING** ---

NOTES | **AFTER DRILLING** ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	ROCK UNCONFINED COMP. STRENGTH (KSF)
0		TOPSOIL - 6 INCHES								
		(FILL) SAND, DARK BROWN	SPT	100	2-5-5 (10)					
		(FILL) CLAY, CRUSHED LIMESTONE, SOME ORGANICS CLAY FILL - BROWNISH RED	SPT	61	3-24-27 (51)					
		WEATHERED LIMESTONE								

AUGER REFUSAL AT 4.3 FEET
BORING TERMINATED AT AUGER REFUSAL

DDS ENGINEERING, PLLC
148 Chester Court
Bowling Green KY 42103
Telephone: 270-843-2247
Fax: 270-843-9323

BORING NUMBER B-10

PROJECT NAME WKU HILLTOPPER FIELDHOUSE **CLIENT** K NORMAN BERRY ARCHITECTS
PROJECT NUMBER T5600G **PROJECT LOCATION** BOWLING GREEN, KY
DATE STARTED 11/2/2022 **COMPLETED** 11/2/2022 **GROUND ELEVATION** 493.9 ft **WEATHER** 60-70, CLOUDY
DRILLING CONTRACTOR HORN **RIG TYPE** DIEDRICH D-50 **GROUND WATER LEVELS:**
DRILLING METHOD 4 1/4" ID HSA **HAMMER** AUTOMATIC **AT TIME OF DRILLING** DRY
LOGGED BY MHR **CHECKED BY** MHR **AT END OF DRILLING** ---
NOTES **AFTER DRILLING** ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	ROCK UNCONFINED COMP. STRENGTH (KSF)
0		TOPSOIL - 5 INCHES								
		(FILL) SAND, DARK BROWN	SPT	100	2-2-6 (8)					
		(FILL) CLAY, SOFT, WET, DARK BROWN TO GRAY								
		STIFF, LEAN CLAY, ORANGISH TO BROWNISH RED	SPT	100	4-14-19 (33)	25.4				
		SPOON AND AUGER LED OFF AT 3.3 FEET								

AUGER REFUSAL AT 4.2 FEET
BORING TERMINATED AT AUGER REFUSAL

DDS ENGINEERING, PLLC
148 Chester Court
Bowling Green KY 42103
Telephone: 270-843-2247
Fax: 270-843-9323

BORING NUMBER B-11

PROJECT NAME WKU HILLTOPPER FIELDHOUSE **CLIENT** K NORMAN BERRY ARCHITECTS
PROJECT NUMBER T5600G **PROJECT LOCATION** BOWLING GREEN, KY
DATE STARTED 11/2/2022 **COMPLETED** 11/2/2022 **GROUND ELEVATION** 493.4 ft **WEATHER** 50-60, DRIZZLE
DRILLING CONTRACTOR HORN **RIG TYPE** DIEDRICH D-50 **GROUND WATER LEVELS:**
DRILLING METHOD 4 1/4" ID HSA **HAMMER** AUTOMATIC **AT TIME OF DRILLING** DRY
LOGGED BY MHR **CHECKED BY** MHR **AT END OF DRILLING** ---
NOTES **AFTER DRILLING** ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	ROCK UNCONFINED COMP. STRENGTH (KSF)
0		TOPSOIL - 6 INCHES								
		(FILL) - CLAY, CRUSHED STONE, ASPHALT FRAGS	SPT	100	16-10-5 (15)					
		FIRM, LEAN CLAY, BROWNISH RED	SPT	100	4-4-6 (10)	30.0				
		WEATHERED LIMESTONE								

AUGER REFUSAL AT 4.5 FEET
BORING TERMINATED AT AUGER REFUSAL

DDS ENGINEERING, PLLC
148 Chester Court
Bowling Green KY 42103
Telephone: 270-843-2247
Fax: 270-843-9323

BORING NUMBER B-12

PROJECT NAME WKU HILLTOPPER FIELDHOUSE
CLIENT K NORMAN BERRY ARCHITECTS
PROJECT NUMBER T5600G
PROJECT LOCATION BOWLING GREEN, KY
DATE STARTED 11/2/2022 **COMPLETED** 11/2/2022
GROUND ELEVATION 493.3 ft **WEATHER** 60-70, DRIZZLE
DRILLING CONTRACTOR HORN **RIG TYPE** DIEDRICH D-50
GROUND WATER LEVELS:
DRILLING METHOD 4 1/4" ID HSA **HAMMER** AUTOMATIC
AT TIME OF DRILLING DRY
LOGGED BY MHR **CHECKED BY** MHR
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	ROCK UNCONFINED COMP. STRENGTH (KSF)
0		TOPSOIL - 6 INCHES								
		(FILL) CLAY AND CRUSHED LIMESTONE	SPT	33	5-5-50/3"					
		WEATHERED LIMESTONE								

AUGER REFUSAL AT 2.5 FEET
BORING TERMINATED AT AUGER REFUSAL

DDS ENGINEERING, PLLC
148 Chester Court
Bowling Green KY 42103
Telephone: 270-843-2247
Fax: 270-843-9323

BORING NUMBER B-13

PROJECT NAME WKU HILLTOPPER FIELDHOUSE
CLIENT K NORMAN BERRY ARCHITECTS
PROJECT NUMBER T5600G
PROJECT LOCATION BOWLING GREEN, KY
DATE STARTED 11/2/2022 **COMPLETED** 11/2/2022
GROUND ELEVATION 493.9 ft **WEATHER** 60-70, CLOUDY
DRILLING CONTRACTOR HORN **RIG TYPE** DIEDRICH D-50
GROUND WATER LEVELS:
DRILLING METHOD 4 1/4" ID HSA **HAMMER** AUTOMATIC
AT TIME OF DRILLING DRY
LOGGED BY MHR **CHECKED BY** MHR
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	ROCK UNCONFINED COMP. STRENGTH (KSF)
0		ASPHALT - 5 INCHES								
		(FILL) CRUSHED LIMESTONE								
		FIRM, FAT CLAY, BROWNISH ORANGE	SPT	72	3-4-50/3"	28.3	51	23	28	

AUGER REFUSAL AT 3.2 FEET
BORING TERMINATED AT AUGER REFUSAL

DDS ENGINEERING, PLLC
148 Chester Court
Bowling Green KY 42103
Telephone: 270-843-2247
Fax: 270-843-9323

BORING NUMBER B-14

PROJECT NAME WKU HILLTOPPER FIELDHOUSE
CLIENT K NORMAN BERRY ARCHITECTS
PROJECT NUMBER T5600G
PROJECT LOCATION BOWLING GREEN, KY
DATE STARTED 11/2/2022 **COMPLETED** 11/2/2022
GROUND ELEVATION 492.8 ft **WEATHER** 60-70, CLOUDY
DRILLING CONTRACTOR HORN **RIG TYPE** DIEDRICH D-50
GROUND WATER LEVELS:
DRILLING METHOD HSA, NX **HAMMER** AUTOMATIC
AT TIME OF DRILLING DRY AFTER SOIL AUGERING
LOGGED BY MHR **CHECKED BY** MHR
AT END OF DRILLING ---
NOTES
AFTER DRILLING 11.80 ft / Elev 481.00 ft AFTER CORING

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	ROCK UNCONFINED COMP. STRENGTH (KSF)
0		TOPSOIL - 5 INCHES								
		(FILL) CLAY, DRY, STIFF, BROWN TO REDDISH BROWN	SPT	94	6-10-9 (19)					
			SPT	100	50/5"/5"					
		AUGER REFUSAL AT 3.0 FEET, BEGAN CORING								
5		LIMESTONE, MODERATELY HARD TO HARD, GRAY TO LIGHT GRAY								
		CLAY SEAM (4.1' - 6.2')								
		LIMESTONE, MODERATELY HARD TO HARD, LIGHT GRAY TO WHITE	RC	72 (61)						2977
10										2441
		LIMESTONE, MODERATE TO MEDIUM HARD, GRAY TO DARK GRAY								2424
15		LIMESTONE, MODERATELY HARD TO HARD, LIGHT GRAY TO WHITE	RC	100 (100)						
										1999
20										

BORING TERMINATED AT 20.5 FEET

DDS ENGINEERING, PLLC
148 Chester Court
Bowling Green KY 42103
Telephone: 270-843-2247
Fax: 270-843-9323

SUMMARY OF LABORATORY RESULTS

PROJECT NAME WKU HILLTOPPER FIELDHOUSE **CLIENT** K NORMAN BERRY ARCHITECTS

PROJECT NUMBER T5600G **PROJECT LOCATION** BOWLING GREEN, KY

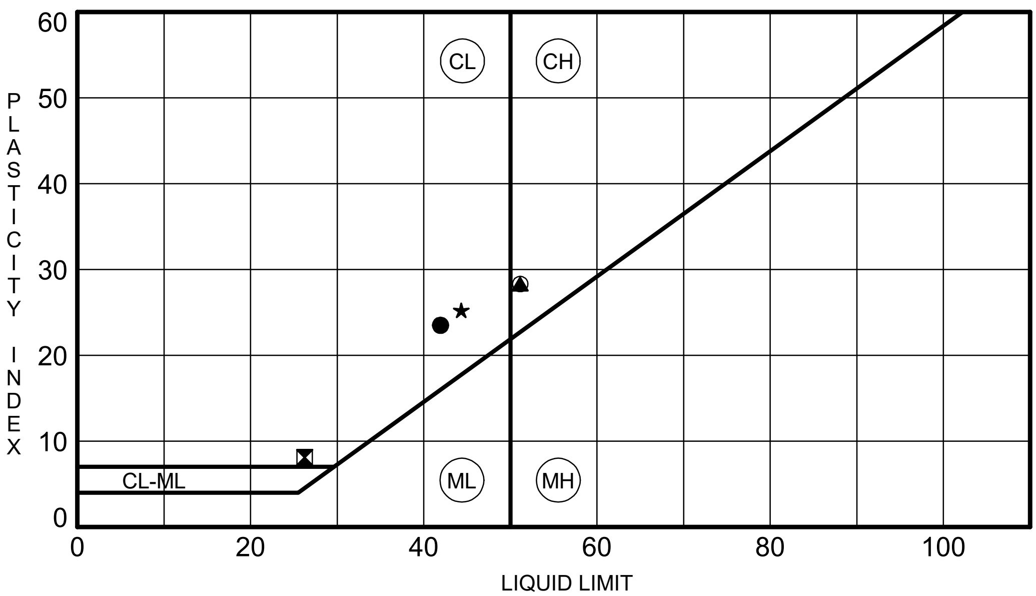
Boring No.	Depth (ft)	Sample Type	Liquid Limit	Plastic Limit	Plasticity Index	%<#200 Sieve	Class-ification	Moisture Content (%)	Unit Weight (pcf)	Soil Unconfined Comp (psf)	Rock Unconfined Comp. (ksf)
B-2	2.5	SS						12.0			
B-2	7.0	SS	42	18	24		CL	21.2			
B-2	11.0	RC							167.6		1930
B-2	12.0	RC							168.7		3215
B-2	23.2	RC							163.0		1950
B-2	27.0	RC							165.7		2225
B-3	2.5	SS						14.5			
B-3	7.0	SS	26	18	8		CL	21.7			
B-3	13.8	RC							168.4		3050
B-3	22.2	RC							169.1		3090
B-3	28.6	RC							167.9		2929
B-4	2.5	SS						22.6			
B-4	4.7	UD	51	23	28		CH				
B-5	2.5	SS						17.5			
B-5	4.5	SS						20.4			
B-8	4.5	SS	44	19	25		CL	23.0			
B-10	2.5	SS						25.4			
B-11	2.5	SS						30.0			
B-13	1.5	SS	51	23	28		CH	28.3			
B-14	6.6	RC							167.6		2977
B-14	10.7	RC							166.1		2441
B-14	13.5	RC							160.0		2424
B-14	17.6	RC							155.6		1999

DDS ENGINEERING, PLLC
148 Chester Court
Bowling Green KY 42103
Telephone: 270-843-2247
Fax: 270-843-9323

ATTERBERG LIMITS RESULTS

ASTM D4718

PROJECT NAME	WKU HILLTOPPER FIELDHOUSE	**PROJECT NUMBER**	T5600G
CLIENT	K NORMAN BERRY ARCHITECTS	**SAMPLE DATE**	11/2/2022
TEST BY	ACN	**TEST DATE**	11/4/2022
REVIEW BY	MHR	**REVIEW DATE**	11/8/2022



	BOREHOLE	DEPTH	LL	PL	PI	Fines	Classification
●	**B-2**	**7.0**	**42**	**18**	**24**		**(CL) - LEAN CLAY, ORANGISH TO REDDISH BROWN**
⊠	**B-3**	**7.0**	**26**	**18**	**8**		**FILL - SILTY CLAY, DARK BROWN TO BLACK (CL)**
▲	**B-4**	**4.7**	**51**	**23**	**28**		**FILL - FAT CLAY , DK BROWN TO GRAY (CH)**
★	**B-8**	**4.5**	**44**	**19**	**25**		**(CL) - LEAN CLAY, BROWINSH TO ORANGISH BROWN**
⊙	**B-13**	**1.5**	**51**	**23**	**28**		**(CH) - FAT CLAY, BROWNISH ORANGE**

PHOTOGRAPHS

PHOTOGRAPH 1 - VIEW OF SITE FROM WEST TO EAST

PHOTOGRAPH 2 - VIEW OF SITE FROM NORTH TO THE SOUTHEAST

PHOTOGRAPHS

PHOTOGRAPH 3 - VIEW FROM EAST TO NORTHWEST

PHOTOGRAPH 4 - VIEW FROM SOUTH TO NORTH

PHOTOGRAPHS

PHOTOGRAPH 5 - VIEW FROM PRACTICE FIELD TO THE EAST

PHOTOGRAPH 6 - VIEW FROM PRACTICE FIELD TO THE NORTHWEST

PHOTOGRAPHS

PHOTOGRAPH 7 - SETTING UP ON SOIL TEST BORING B-7

PHOTOGRAPH 8 - PLAYING SURFACE REMOVED PRIOR TO DRILLING

PHOTOGRAPHS

PHOTOGRAPH 9 - DRILLING SOIL TESTING BORING B-12

PHOTOGRAPH 10 - SOIL TEST BORING AND ROCK CORING AT B-3

PHOTOGRAPHS

PHOTOGRAPH 11 - ORGANIC LADEN FILL / WOOD FRAGMENTS B-4

PHOTOGRAPH 12 - BLACK ORGANIC LADEN CLAY - B-5

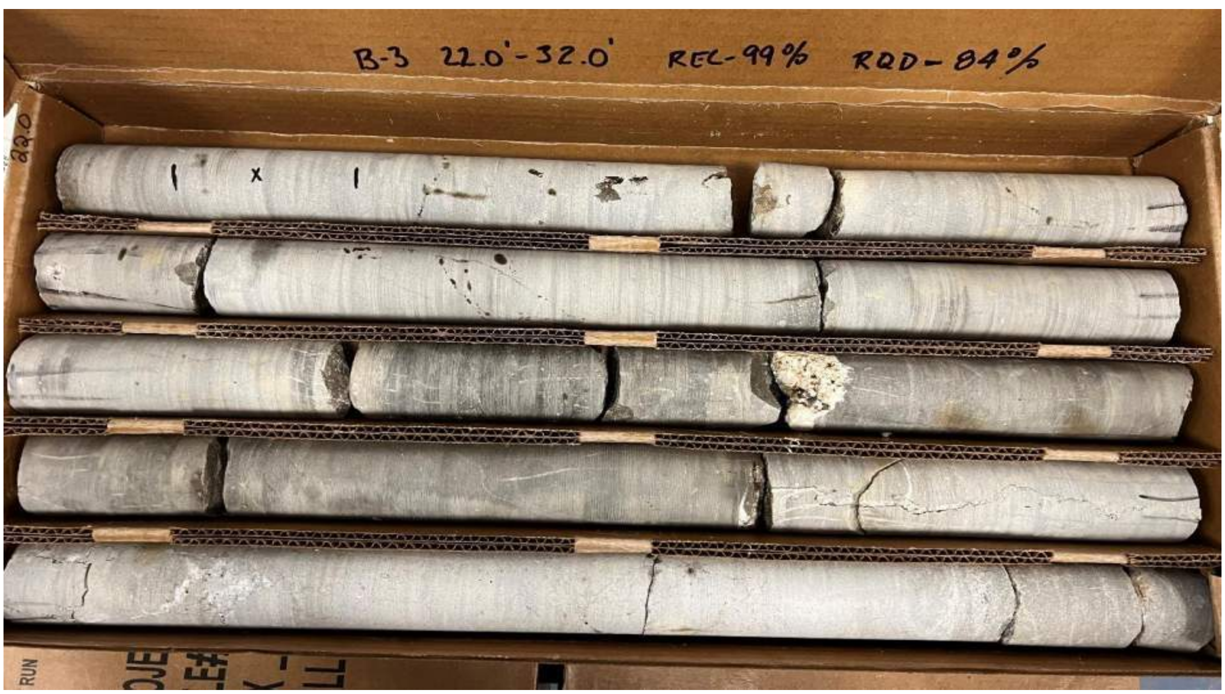
PHOTOGRAPHS

PHOTOGRAPH 13 - B-2 ROCK CORE (9.8' - 19.8')

PHOTOGRAPH 14 - B-2 ROCK CORE (19.8' - 29.8')

PHOTOGRAPHS

PHOTOGRAPH 15 - B-3 ROCK CORE (12.5' - 22.0')



PHOTOGRAPH 16 - B-3 ROCK CORE (22.0' - 32.0')

PHOTOGRAPHS

PHOTOGRAPH 17 - B-4 ROCK CORE (3.0' - 10.5')

PHOTOGRAPH 18 - B-4 ROCK CORE (10.5' - 20.5')

FIELD TESTING PROCEDURES

DDS ENGINEERING, PLLC (DDS) subcontracted the drilling and sampling to Horn and Assocatiates. Horn and Associates performs field tests in general accordance with the American Society for Testing and Materials (ASTM) or the United States Army Corps of Engineers procedures. These procedures are generally recognized as the basis for uniformity and consistency of test results in the geotechnical engineering profession. All work is initiated and supervised by qualified geotechnical professionals.

Subsequent portions of this attachment briefly describe our field testing procedures. Where applicable, we have referenced these procedures to ASTM standards which contain specific descriptions of apparatus, procedures, reporting, etc.

SOIL TEST BORING, ASTM D-1586

The borings were made with a hollow-stem auger powered by a drill rig. At regular intervals, soil samples were obtained through the hollow augers with a standard 1.4 inch I.D., 2.0-inch O.D. split-tube sampler.

The sampler was initially 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 as the standard penetration resistance (SPT N-value). Penetration resistance, when properly evaluated, is an index to soil consistency and strength.

In the field, our geotechnical professional logged and described the samples as they were obtained. Representative portions of each soil sample were labeled and sealed, then transported to our laboratory. The samples were examined by a graduate geotechnical engineer or geologist to visually check the field descriptions. Boring data, including sample intervals, penetration resistances, soil descriptions, and groundwater level are shown on the attached Test Boring Records.

AUGER REFUSAL MATERIALS

Auger refusal is a term that describes subsurface materials sufficiently competent to prevent further penetration by our drilling augers. Our criterion for auger refusal is the inability of our drill rig to advance the augers with 300 psi down pressure. Typically, refusal materials exhibit penetration resistances in excess of 100 blows per foot. Refusal materials can be hard cemented soil, soft weathered rock, coarse gravel or boulders, rubble or other hard debris, 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.

UNDISTURBED SAMPLING, ASTM D-1587

Split-tube samples obtained in conjunction with penetration testing are suitable for visual examination and classification tests but are not sufficiently intact for quantitative testing. Relatively undisturbed samples suitable for quantitative laboratory testing were obtained by slowly and uniformly pushing sections of 3-inch O.D., steel tubing into the soil at the desired sampling levels. The length of the soil sample was measured and recorded immediately after removing a sampling tube and the encased soil from the ground. The ends of the tube were then sealed with plastic caps, and tape, and transported to our laboratory in protective containers. The locations of the undisturbed samples are shown on the Test Boring Records.

ROCK CORING, ASTM D-2113

After we encountered refusal to the power auger, steel casing was inserted in the borehole so that overlying soil and rock materials would not collapse and block the drilling tools. Then refusal materials were cored using a diamond-studded bit fastened to the end of a hollow tube core barrel. This device was rotated at high speed by the drill rig, and the cuttings were brought to the surface by circulating water. Core samples of the materials penetrated were protected and retained in the swivel-mounted inner tube of the core barrel. Upon completion of each coring "run", the core barrel was brought to the surface and the samples removed and placed in core boxes. The samples were logged in the field by the driller, then returned to the laboratory.

At the laboratory, the rock's relative hardness, percent recovery, and rock quality designation were determined by a geotechnical engineer or geologist. Criteria for defining hardness are listed below

Recovery is the ratio of the sample length obtained to the length of core run, expressed as a percent. The percent recovery is related to rock soundness and continuity. Rock descriptions and recoveries are shown on the Test Boring Records. The [3.NX].NQ size designates a bit which obtains rock cores [5. 21/8] 17/8 inches in diameter.

The rock quality designation (RQD) is also a way of describing the rock soundness andcontinuity. RQD is the ratio of the sum of the lengths of rock pieces 4 or more inches long to the length of the core run expressed as a percent. Rock described as a function of RQD is as follows:

RELATIVE HARDNESS OF ROCK

Very Soft	Pieces 1 inch or more in thickness can be broken by finger pressure; can be scratched readily by fingernail
Soft	May be broken with fingers.
Medium	May be scratched with a nail; corners and edges may be broken with fingers.
Mod. Hard	Moderate blow of hammer. required to break sample
Hard	Hard blow of hammer required to break sample.
Very Hard	Several hard blows of hammer required to break sample.

RELATIVE QUALITY OF ROCK CORES

Quality	**RQD**
Very Poor	0 to 25%
Poor	25 to 50%
Fair	50 to 75%
Good	75 to 90%
Excellent	90 to 100%

CORRELATION OF
STANDARD PENETRATION RESISTANCE
WITH
RELATIVE COMPACTNESS AND CONSISTENCY

SAND AND GRAVEL

Standard Penetration Resistance (Blows/Foot)	Relative Compactness
0-4	Very Loose
5-10	Loose
11-20	Firm
21-30	Very Firm
31-50	Dense
Over 50	Very Dense

SILT AND CLAY

Standard Penetration Resistance (Blows/Foot)	Consistency
0-2	Very Soft
3-4	Soft
5-8	Firm
9-15	Stiff
16-30	Very Stiff
31-50	Hard
Over 50	Very Hard

LABORATORY TESTING PROCEDURES

DDS ENGINEERING, PLLC (DDS) performed laboratory testing on select soil and rock samples obtained from the field exploration. The laboratory tests were performed in general accordance with the American Society for Testing and Materials (ASTM). These procedures are generally recognized as the basis for uniformity and consistency of test results in the geotechnical engineering profession. The tests are performed by skilled technicians trained in ASTM procedures. The laboratory equipment is well maintained and calibrated at least yearly.

Subsequent portions of this attachment briefly describe our testing procedures. Where applicable, we have referenced these procedures to ASTM standards which contain specific descriptions of apparatus, procedures, reporting, etc.

MOISTURE CONTENT DETERMINATION, ASTM D-2216

The moisture content of soils is an indicator of various physical properties, including strength and compressibility. Selected samples obtained during exploratory drilling were taken from their sealed containers. Each sample was weighed and then placed in an oven heated to $110^{o}C + 5^{o}$. The sample remained in the oven until the free moisture had evaporated. The dried sample was removed from the oven, allowed to cool, and reweighed. The moisture content was computed by dividing the weight of evaporated water by the weight of the dry sample. The results, expressed as a percent.

UNIT WEIGHT DETERMINATIONS, ASTM D-7263

Soil samples were selected for unit weight determination. Unit weight is an indicator of various physical properties of the soil, including strength, compressibility and permeability. Each sample was prepared by trimming the ends and measuring its length and diameter. The specimen volume was computed from these dimensions. The sample was then weighed and the moisture content determined to compute the wet and dry unit weights.

ATTERBERG LIMITS DETERMINATION, ASTM D-4318

Representative samples were subjected to Atterberg limits testing to determine the soil's plasticity characteristics. The plasticity index (PI) is the range of moisture content through 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 wet enough to flow as a viscous fluid. To determine the liquid limit, a soil specimen is first washed through a No. 40 sieve. The materials finer than the No. 40 sieve are retained and dried until the soil is in a viscous fluid state. A portion of this soil is then placed in a brass cup of standardized dimensions. A groove is cut through the middle of the soil specimen with a grooving tool of standard dimensions. The cup is attached to a cam that lifts the cup 10 mm, and then allows the cup to fall onto a hard rubber base. The cam is rotated at about 2 cps until the two halves of the soil specimen come in contact at the bottom of the groove for a distance of 1/2 inch. The number of blows required to achieve this 1/2 inch contact is recorded, and part of the specimen is subjected to a moisture content determination. The remainder of the specimen is allowed to air dry for a short time, and the grooving process and cam action repeated. This testing sequence is repeated until more than 25 blows is required to achieve the required groove contact. After the number of blows vs. moisture content for the various test points are plotted on arithmetic graph paper, the moisture content corresponding to 25 blows is designated the liquid limit.

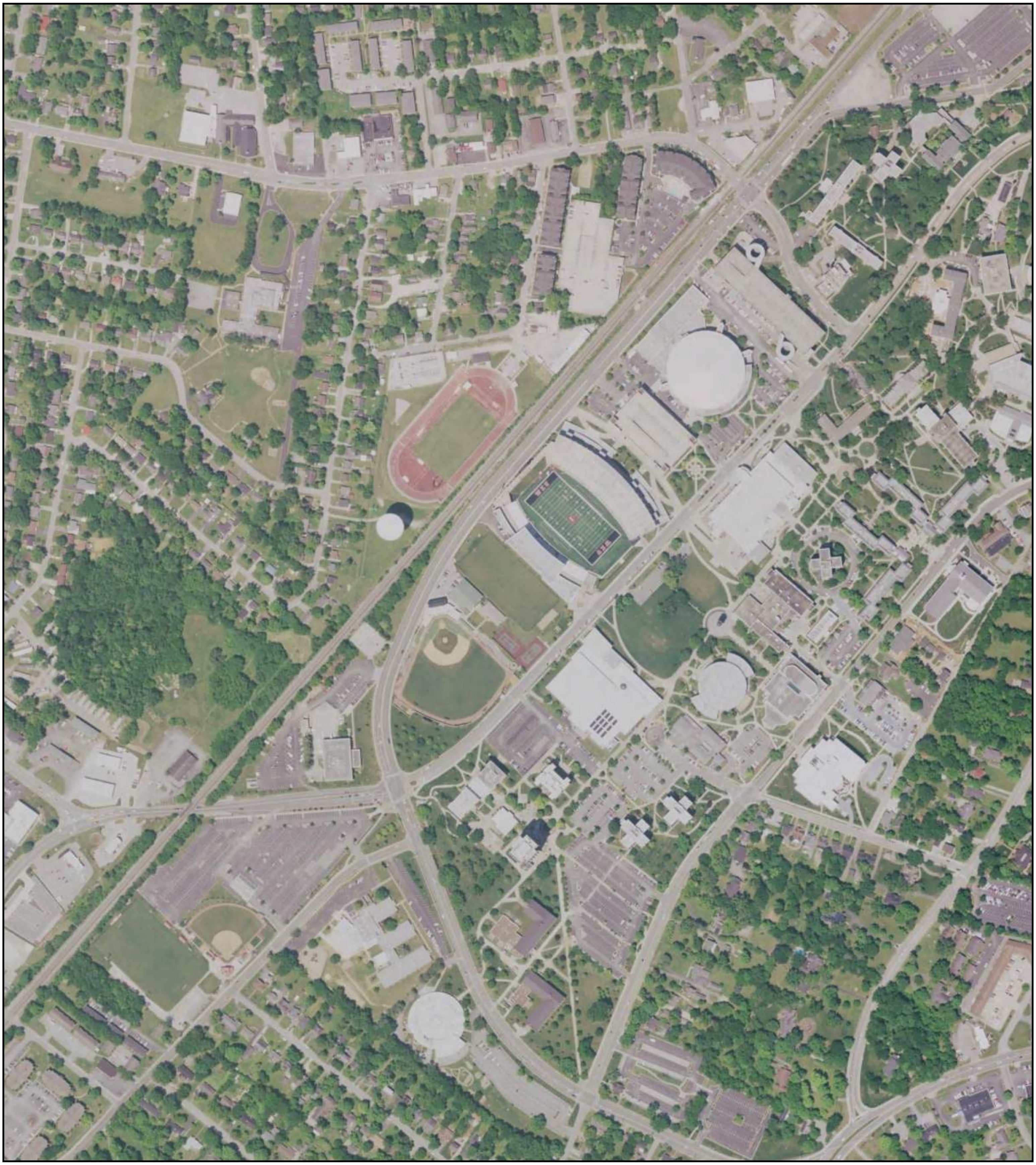
148 CHESTER COURT • BOWLING GREEN, KENTUCKY 42103 • PHONE (270) 843-2247 • FAX (270) 843-9323
www.ddsengineering.com

The plastic limit (PL) is the lowest moisture content at which the soil is sufficiently plastic to be manually rolled into threads 1/8" in diameter. The plastic limit is determined by taking a pat of soil remaining from the liquid limit test, and repeatedly rolling, kneading, and air drying it until the soil breaks into threads about 1/8 inches in diameter and 3/8 inches long. The moisture content of these soil threads is then determined, and is designated the plastic limit.

UNCONFINED COMPRESSION TEST, ASTM D-2216

An unconfined compression test was performed to determine the approximate soil shear strength (cohesion) parameter "C". A relatively undisturbed soil sample obtained from driving a 3” outside diameter thin wall sample was selected for testing. The sample was trimmed on both ends and then placed in a compression testing machine. The sample was compressed at a constant rate of strain, and load measurements were made as the sample failed in undrained shear. The maximum load on the specimen was recorded, and the resultant stress calculated. This calculated stress is known as the unconfined compressive strength (qu), and is divided by 2 to obtain "C", the shear strength or apparent cohesion.

WKU Fieldhouse 1605 Avenue of Champions Bowling Green, KY	N	**2020** HIG Project # 2039655 Client Project # T5600G Approximate Scale 1: 6,000 (1"=500') www.historicalinfo.com	



WKU Fieldhouse 1605 Avenue of Champions Bowling Green, KY	N	**2016** HIG Project # 2039655 Client Project # T5600G Approximate Scale 1: 6,000 (1"=500') www.historicalinfo.com	HIG

WKU Fieldhouse
1605 Avenue of Champions
Bowling Green, KY

N

2012

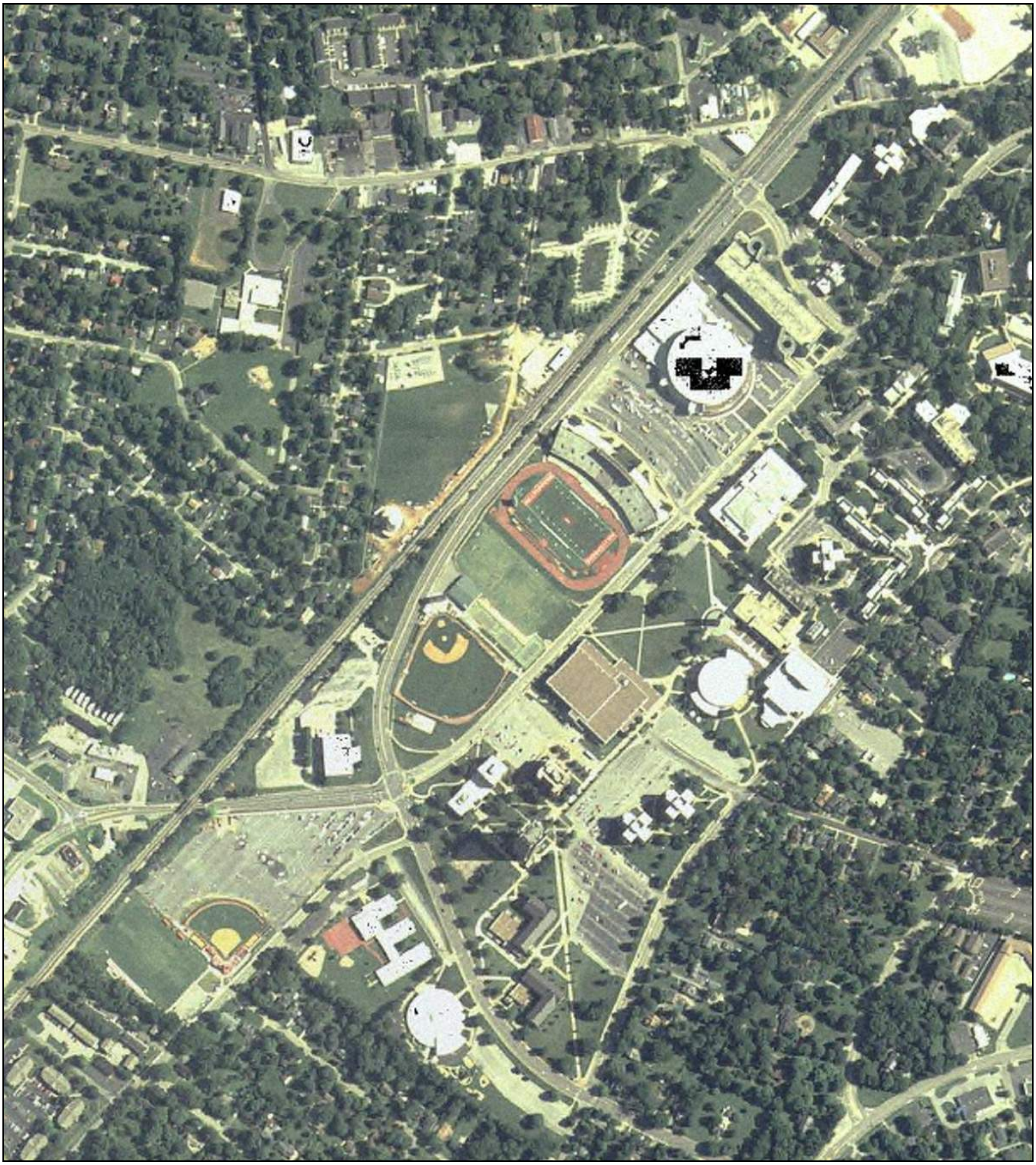
HIG Project # 2039655
Client Project # T5600G
Approximate Scale 1: 6,000 (1"=500')
www.historicalinfo.com

WKU Fieldhouse
1605 Avenue of Champions
Bowling Green, KY

N

2008

HIG Project # 2039655
Client Project # T5600G
Approximate Scale 1: 6,000 (1"=500')
www.historicalinfo.com



WKU Fieldhouse 1605 Avenue of Champions Bowling Green, KY	N	**2004** HIG Project # 2039655 Client Project # T5600G Approximate Scale 1: 6,000 (1"=500') www.historicalinfo.com

WKU Fieldhouse
1605 Avenue of Champions
Bowling Green, KY

N

1993

HIG Project # 2039655
Client Project # T5600G
Approximate Scale 1: 6,000 (1"=500')
www.historicalinfo.com

WKU Fieldhouse
1605 Avenue of Champions
Bowling Green, KY

N

1982

HIG Project # 2039655
Client Project # T5600G
Approximate Scale 1: 6,000 (1"=500')
www.historicalinfo.com

WKU Fieldhouse 1605 Avenue of Champions Bowling Green, KY	N	**1980** HIG Project # 2039655 Client Project # T5600G Approximate Scale 1: 6,000 (1"=500') www.historicalinfo.com	

WKU Fieldhouse
1605 Avenue of Champions
Bowling Green, KY

N

1967

HIG Project # 2039655
Client Project # T5600G
Approximate Scale 1: 6,000 (1"=500')
www.historicalinfo.com

WKU Fieldhouse
1605 Avenue of Champions
Bowling Green, KY

N

1958

HIG Project # 2039655
Client Project # T5600G
Approximate Scale 1: 6,000 (1"=500')
www.historicalinfo.com

HIG

WKU Fieldhouse
1605 Avenue of Champions
Bowling Green, KY

N

1954

HIG Project # 2039655
Client Project # T5600G
Approximate Scale 1: 6,000 (1"=500')
www.historicalinfo.com

WKU Fieldhouse
1605 Avenue of Champions
Bowling Green, KY

N

1950

HIG Project # 2039655
Client Project # T5600G
Approximate Scale 1: 6,000 (1"=500')
www.historicalinfo.com

Important Information about This

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.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering 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 given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full.*

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

- the site's size or shape;
- 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;
- the elevation, configuration, location, orientation, or weight of the proposed structure;
- the 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. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered.*

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed.* The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation.*

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include 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 personnel, equipment, and techniques used to perform an environmental study – e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

GEOPROFESSIONAL BUSINESS ASSOCIATION

Telephone: 301/565-2733

e-mail: info@geoprofessional.org www.geoprofessional.org