Geotechnical Exploration
Northern Kentucky University
New Residence Hall
Highland Heights, Kentucky

Prepared for:

Moody Nolan
Covington, Kentucky

Prepared by:

Geotechnology, Inc.
Erlanger, Kentucky

Date:

August 7, 2019

Geotechnology Project No.:

J032441.01

SAFETY
QUALITY
INTEGRITY
PARTNERSHIP
OPPORTUNITY
RESPONSIVENESS

August 7, 2019

Ms. Cheryl Sydzyik
Moody Nolan
434 Madison Avenue
Covington, Kentucky 41011

Re: Geotechnical Exploration
Northern Kentucky University
New Residence Hall
Highland Heights, Kentucky
Geotechnology Project No. J032441.01

Dear Ms. Sydzyik:

Presented in this report are the results of our geotechnical exploration completed for the Northern Kentucky University New Residence Hall in Highland Heights, Kentucky. Our services were performed in general accordance with the terms of our April 11, 2019 Subcontract Agreement with Moody Nolan, which references Geotechnology's January 23, 2019 Proposal No. J032441.01.

We appreciate the opportunity to provide the geotechnical services for this project. If you have any questions regarding this report, or if we may be of any additional service to you, please do not hesitate to contact us.

Respectfully submitted,
GEOTECHNOLOGY, INC.

John S. Nealon, PhD, PE, PG
Principal Engineer

STATE OF KENTUCKY
JOHN S. NEALON
20644
LICENSED PROFESSIONAL ENGINEER

Joseph D. Hauber, PE
Senior Project Manager

JSN/JDH

Copies submitted: Client (email/2 mail)
THP Limited, Inc. (email)

TABLE OF CONTENTS

LIST OF TABLES

GEOTECHNICAL EXPLORATION
NORTHERN KENTUCKY UNIVERSITY
NEW RESIDENCE HALL
HIGHLAND HEIGHTS, KENTUCKY
August 7, 2019 | Geotechnology Project No. J032441.01

1.0 INTRODUCTION

Geotechnology, Inc. (Geotechnology) has prepared this geotechnical exploration report for Moody Nolan for the proposed Northern Kentucky University (NKU) New Residence Hall to be located on the eastern corner of Kenton Drive and Carroll Drive in Highland Heights, Kentucky. Our services documented in this report were provided in general accordance with the terms of our April 11, 2019 Subcontract Agreement with Moody Nolan, which references Geotechnology's January 23, 2019 Proposal No. J032441.01.

At the outset of the project, the purposes of the geotechnical exploration were to 1) evaluate the general subsurface profile at the Campbell Drive site (not the Kenton and Carroll Drives site noted above) and the engineering properties of the soils and bedrock, and 2) to develop recommendations for the geotechnical aspects of the design and construction of the project, as defined in our proposal. Our original scope of services included geotechnical borings, laboratory testing, engineering analyses, and preparation of a geotechnical report for the Campbell Drive site. However, following completion of drilling and laboratory testing for the Campbell Drive site, a decision was made to move the residence hall location to the western corner of the existing Parking Lot F. This report is for the latter site and not the former.

2.0 PROJECT INFORMATION

As previously stated, a decision was made to move the residence hall location to the western corner of the existing Parking Lot F, following completion of drilling and laboratory testing for the Campbell Drive site. Parking Lot F is bordered to the northwest by Kenton Drive and to the southwest by Carroll Drive. The Lot F location had been previously explored and evaluated in 2016 by Thelen Associates, A Division of Geotechnology, Inc. (Geotechnology). The geotechnical report for the Lot F site was submitted by Geotechnology on December 16, 2016 (Geotechnology Project No. J028765.01).

The proposed residence hall footprint will cover approximately 15,600 square feet and will be a five-story, L-shaped, wood-frame or cold-formed-steel-frame, slab-on-grade building having its ground floor at El. 821.70.[1] In the wing of the building running parallel to Kenton Drive, the load-

[1] The elevations in this report are referenced to North American Vertical Datum of 1988 (NAVD 88) in units of feet, unless noted otherwise.

bearing walls will extend to foundation level, and will be supported by continuous footings where possible, and by structural grade beams spanning between drilled shafts where shallow foundation support is not feasible or economical. Along the wing running perpendicular to Kenton Drive, the first floor will be of steel-frame construction, and widely-spaced columns will be used to create a podium level that will provide an open common space for the students. Maximum column and wall loads will be 375 kips and 14.5 kips per lineal foot (klf), respectively.

Proposed site grades are not available at this time. However, based on comparison of boring elevations and the proposed ground floor elevation of 821.70, about 4 feet of cut and 5 feet of new fill will be required to establish the building pad and surrounding grades for the residence hall.

3.0 SITE CONDITIONS

The site location and pre-development topography of the project site are shown on the Site and Boring Plan (Sheet No.1) and the 1963 Topography Plan (Sheet No. 2) included in Appendix B. The 1963 Topography Plan was derived from 1963 topographic mapping that was published by the Northern Kentucky Area Planning Commission (NKAPC).

The existing site terrain slopes gently downward to the northeast in the vicinity of the proposed building pad, and slopes steeply downward beyond the northeast edge of Lot F. The building site is completely occupied by Lot F, a large, asphaltic concrete student parking lot. Past bulk grading activities to construct the parking lot area involved filling in a drainage valley (cf. the 1963 Topography Plan on Sheet No. 2) and cutting down the adjacent ridges. Maximum depths of previous cutting and filling are estimated to have been on the order of 40 feet and 30 feet, respectively. Approximately 15 feet of relief currently exists across the existing parking lot, and about 10 feet across the building area. An approximately 30-foot-high, 2.5-horizontal-to-1-vertical (2.5H:1V) fill slope bounds the east side of Lot F and descends to the northeast. The base of this 2.5H:1V slope steepens to approximately 1.5H:1V for the lowest 10 feet of grade change in the vicinity of an outlet for an existing storm sewer that roughly follows the now-buried valley alignment. An approximate 15- to 20-foot-high, 4H:1V slope extends upwards from the southeast edge of Lot F to an adjacent student parking lot (Lot I).

4.0 SUBSURFACE EXPLORATION

The subsurface exploration for Geotechnology Project No. J028765.01 consisted of eleven borings, numbered 1 through 11. The boring locations were selected by Geotechnology, and were staked in the field by a Geotechnology survey crew relative to a given benchmark elevation of EL. 813.75 at the rim of a storm sewer catch basin immediately southeast of the intersection of Campbell Drive and Kenton Drive. We note that the rim elevation of this catch basin is posted as 813.28 on the base plan provided by Moody Nolan on June 20, 2019. The locations of the borings are shown on our Site and Boring Plan, which is included in Appendix B.

The borings were drilled on November 23 and November 25, 2016 with a buggy-mounted drill rig advancing hollow-stem augers, as indicated on the boring logs presented in Appendix C. Sampling of the overburden soils and bedrock was accomplished ahead of the augers at the depths indicated on the boring logs, with either 2-inch-outside-diameter (O.D.) split-spoons or 3-inch-O.D., thin-walled Shelby tube samplers in general accordance with the procedures outlined by ASTM D1586 and ASTM D1587, respectively. Standard Penetration Tests (SPTs) were performed on the split-spoon samples to obtain the N-values[2] of the sampled materials.

Observations for groundwater were made in the borings during drilling, at the completion of drilling, and before backfilling the boreholes.

As each boring was advanced, the Drilling Foreman kept a field log of the subsurface profile noting the soil and bedrock types and stratifications, groundwater, SPT results, and other pertinent data. Representative portions of the split-spoon samples were placed in glass jars with lids to preserve the in-situ moisture contents of the samples. The Shelby tubes were capped and taped at their ends to preserve the in-situ moisture contents and densities of the samples, and the tubes were transported and stored in an upright position. The glass jars and Shelby tubes were marked and labeled in the field for identification when returned to our laboratory.

The boring logs were prepared by an Engineering Geologist on the basis of the field logs, visual classification of the soil and bedrock samples in the laboratory, and the laboratory test results. Soil and Rock Classification Sheets are also included in Appendix C, which describe the terms and symbols used on the boring logs. The dashed lines on the boring logs indicate an approximate change in strata as estimated between samples, whereas a solid line indicates that the change in strata occurred within a sample where a more precise measurement could be made. The transition between strata can be abrupt or gradual.

5.0 LABORATORY REVIEW AND TESTING

Upon completion of the fieldwork, the samples recovered from the borings were transported to our Soil Mechanics Laboratory, where they were visually reviewed and classified by the Project Engineering Geologist.

Laboratory testing was performed on selected soil and rock samples to estimate engineering and index properties. Laboratory testing of the selected soil samples included moisture content, Atterberg limits, and unconfined compression tests. The test results are summarized in the

[2] The Standard Penetration Test Value, or N-value, is defined as the number of blows required to drive the split-spoon sampler 12 inches with a 140-pound hammer falling 30 inches. Since the split spoon sampler is driven 18 inches or until refusal, the blows for the first 6 inches are for seating the sampler, and the number of blows for the final 12 inches is the N-value. Additionally, "refusal" of the split-spoon sampler occurs when the sampler is driven less than 6 inches with 50 blows of the hammer.

Tabulation of Laboratory Tests in Appendix D, along with the unconfined compressive strength test forms.

6.0 SUBSURFACE CONDITIONS

The borings revealed a general soil and bedrock profile consisting of variable depths of uncontrolled fill underlain by colluvial and/or residual soils and by the interbedded shale and limestone bedrock. More specific descriptions of the subsurface strata are provided below, and boring logs containing detailed material descriptions are located in Appendix C

6.1 Stratification

6.1.1 Pavement

Six of the borings were drilled through the existing pavement of parking Lot F, including Borings 2 through 5, 7, and 10. Four of the borings revealed approximately 5 to 6 inches of asphaltic concrete (AC) directly over the uncontrolled fill. Two of the borings revealed 6 to 9 inches of AC over 3 to 6 inches of granular base.

6.1.2 Topsoil

Topsoil was encountered at the ground surface in Borings 6, 9, and 11. The thickness of the topsoil in these borings varied from 0.2 to 0.3 feet.

6.1.3 Fill

Existing fill was encountered in each of the borings with the exception of Boring 2. The fill in the borings varied from 2 to 28 feet thick and was generally comprised of a mixture of clayey soils and shale and limestone from the bedrock, presumably from the previous bedrock cuts across the site. Additionally, the fill was described as medium stiff to very stiff with variable moisture contents and intermittent zones of nested shale fragments and limestone floaters. Because of the random moisture content and in-situ density of the fill, and because the fill was placed without compaction testing over intermittent low-density soils, the fill is considered to be uncontrolled.

Moisture content testing on the fill soils revealed a wide range in moisture content percentages ranging from 13.3 to 32.0 percent. Atterberg limits testing was performed on three samples of the fill. Two of the samples revealed liquid limits ranging from 39 to 46 percent and plasticity indices ranging from 18 to 23 percent, and were therefore classified as CL soils according to the Unified Soil Classification System (USCS). The remaining sample revealed a liquid limit of 61 percent and a plasticity index of 34 percent, and classified as a CH (i.e., highly plastic) soil per the USCS. Three unconfined compressive strength tests on the fill from Boring 4 yielded unconfined compressive strengths ranging from 3,030 to 4,650 pounds per square foot (psf).

6.1.4 Sediments

Sediments consist of recent, low-density alluvial soils that are deposited by fluvial or flowing water systems (e.g., swales, streams, rivers, etc.). Sediment was encountered beneath the fill in Boring 10 between the depths of 22 and 28.5 feet. The sediment was described as a dark gray, moist to wet, soft, silty clay with silt seams. A moisture content test on the sediment indicated 34.3 percent moisture.

6.1.5 Colluvium

Colluvial soils form on hillsides by the downslope transport of soil and rock material under the influence of gravity. Colluvium was encountered beneath the fill in Borings 3, 5, 7, 9, and 11. The colluvium in these borings was described as brown and gray, moist to wet, medium stiff to very stiff, silty clay with shale and limestone fragments.

Moisture content testing on the colluvium revealed moisture contents ranging from 14.4 to 26.8 percent.

6.1.6 Residuum

Residual soils form by in-situ weathering of the underlying bedrock into a soil. Occasionally, bedrock remnants (i.e., shale or limestone layers) may be encountered within the residual soils. Residual soils were encountered in Borings 2 and 9, and were described as brown and gray, moist, stiff to very stiff, silty clay and clay with trace bedding planes.

Moisture content testing of the residuum yielded moisture contents ranging from 18.0 to 38.7 percent. An Atterberg limits test performed on a sample from Boring 2 yielded a liquid limit of 73 percent and a plasticity index of 44 percent, which classified the soil as a CH (i.e., highly plastic) soil per the USCS.

6.1.7 Bedrock

The overburden soils at the site are underlain by bedrock consisting of interbedded shale and limestone layers. Bedrock was encountered at depths ranging from 2.0 to 28.5 feet below the ground surface in the borings.

According to the 1962 USGS Geologic Map of the Newport Quadrangle, Newport, Kentucky, the bedrock immediately underlying the overburden soils belongs to the Ordovician-aged Fairview Formation. The referenced USGS Map indicates that the Fairview Formation is comprised of interbedded shale and limestone of approximately equal percentages. Limestone layers are regularly between 4 to 8 inches thick, but can be 14 inches thick or more in some locations.

Bedrock in the Northern Kentucky Area is typically categorized as highly weathered, weathered, or unweathered, based on the degree of weathering of the shale component. The highly weathered zone is typically the uppermost zone, wherein the shale is brown to olive brown in color and has almost weathered to a clay. In the intermediate weathered zone, the shale is typically olive brown with occasional gray and is stronger than the shale in the highly weathered zone. In the unweathered parent zone, the shale is gray and is stronger than the shale in the weathered zones. Each zone is interbedded with limestone. It is common for one or both of the weathered shale bedrock zones to be absent due to differential weathering, erosion, or prior excavation. The Rock Classification Sheet, which is included in Appendix C, describes the varying degrees of weathering along with the rock strength descriptions that are used on the appended boring logs.

Regarding the limestone, these layers are predominantly unweathered, and their strengths are estimated to range from medium strong to very strong (i.e., uniaxial compressive strengths ranging from 4,000 psi to upwards of 30,000 psi). Occasionally, layers are encountered within the bedrock profile where groundwater seepage is concentrated, and weathering of the limestone layers is more advanced.

Interbedded, highly weathered shale and limestone bedrock was encountered in Borings 2, 8, 10, and 11 at variable depths within the previous fill and cut areas. The thickness of the highly weathered to weathered bedrock, where penetrated, varied from 2.5 to 5.0 feet. The highly weathered shale was described as extremely weak. Moisture content testing on the highly weathered shale revealed moisture contents ranging from 8.8 to 18.9 percent.

Interbedded, weathered shale and limestone bedrock was encountered in Borings 1, 2, 3, 7 and 9. The thickness, where penetrated, was approximately 2.5 feet. The weathered shale was described as extremely weak. Moisture contents of two samples of the weathered shale were 5.9 and 15.1 percent.

Interbedded, unweathered shale and limestone bedrock was encountered in Borings 1, 3 through 9, and 11. The depth to the top of the unweathered bedrock, where encountered, ranged from 2 feet to 29.5 feet from the ground surface in Borings 6 and 11, respectively. The unweathered shale was described as extremely weak. Moisture contents of four samples of the unweathered shale varied from 5.1 to 13.5 percent.

6.2 Groundwater Conditions

As mentioned in Section 4.0, groundwater observations were made in the borings during drilling, at the completion of drilling, and before backfilling the boring holes.

In general, groundwater was first encountered in Borings 5, 9, 10, and 11 at the soil/bedrock interface; however groundwater levels rose in each of these four test borings and established artesian conditions (i.e., groundwater under head pressure) over a 24-hour period. The maximum increase in groundwater level over the 24-hour period was 17.5 feet in Boring B-11, in which the artesian head rose to within 7 feet of the existing ground surface. The borings in which groundwater was encountered are centered in the now-buried valley that had been filled with uncontrolled fill.

Based on the groundwater observations and our local experience, groundwater seepage is anticipated along the soil/bedrock interface and in saturated zones of fill or native soils that are within the perched groundwater zones, or that are below the groundwater table. Locally concentrated flow may occur due to saturated layers of fill or native soils or along fractures in the bedrock. Additionally, groundwater levels and seepage amounts are expected to vary with time, location, season of the year, and amounts of precipitation.

7.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the boring logs, visual examination of the recovered samples, the laboratory test results, our understanding of the proposed project, our engineering analyses, and our experience as Consulting Soil and Foundation Engineers in the Northern Kentucky Area, we have reached the following conclusions and make the following recommendations of this report.

7.1 Subsurface Conditions

As discussed in Section 3.0, the project site is an existing large, gently-sloped parking lot that was established by cutting of nearby ridgetops and filling of a pre-existing, northeast-trending drainage valley. The ground surface or pavement in the project area is underlain by variable depths of uncontrolled fill soils over intermittent native sediment, colluvial, and residual soils over the interbedded shale and limestone bedrock. Refer to Section 6.0 and the boring logs in Appendix C for additional information on the subsurface strata.

As discussed in Section 6.0, four of the borings encountered an artesian groundwater condition at the soil/bedrock interface. Presumably, this has resulted from the filling of the aforementioned drainage valley without providing subsurface drainage. The groundwater seepage that typically flows along the soil/bedrock interface, within layers of the bedrock, and along the former valley bottom has been restrained by the valley fill, which is acting as an aquitard. We anticipate that the groundwater at the soil/bedrock interface discharges into the valley to the northeast of the project site in the proximity of the storm sewer outlet discussed in Section 3.0 of this report.

7.2 Excavation Support

Excavation support should be the responsibility of the Contractor. Excavation support should be designed and implemented such that excavations are adequately ventilated and braced, shored, and/or sloped in order to protect and ensure the safety of workers within and near the excavations and to protect adjacent ground, slopes, structures, and infrastructure. Federal, state, and local safety regulations should be satisfied. The analyses, discussions, conclusions, and recommendations throughout this report are not to be interpreted as pre-engineering compliance with any safety regulation.

7.3 Site Preparation and Earthwork

As stated in Section 2.0, earthwork for this project will involve cuts and fills up to approximately 5 feet.

The initial preparation of the site for grading should include the removal of vegetation, heavy root systems, topsoil, and existing pavement from the proposed cut, fill, pavement, and structure areas. The topsoil may be stockpiled for future use on the completed cut and fill slopes or in landscaped areas, if permitted by specification, whereas the vegetation, including the heavy root systems should be disposed of off-site in accordance with applicable regulations.

Existing pavements within the grading and proposed structure limits should be demolished and removed. Asphaltic concrete, rubble, and debris associated with the pavement removal should be disposed of off-site, unless there are provisions in the specifications for on-site reclamation of these materials. We should review these provisions to evaluate their impact on the recommendations of this report. Pavements outside of the footprints of the proposed structures may temporarily be left in place prior to removal and/or replacement to provide a stable base for construction equipment.

Experience indicates that the overburden soils and the highly weathered and weathered zones of the bedrock can be excavated with dozers and scrapers, although ripping is necessary to loosen the bedrock so that it can be picked up by the scrapers. Excavations that extend into the unweathered gray shale and limestone bedrock become more difficult with depth, and more ripping may be required to loosen the bedrock.

Historically, structures at NKU having floor slabs supported on fresh exposures of gray, unweathered shale have experienced issues with floor slab heave as the shale has absorbed moisture following construction. We also note that highly plastic soils were encountered on this project site in the uncontrolled fill and in the residuum (with measured plasticity indices of 34 and 44 percent). Therefore, after clearing, grubbing, and making the required excavations in cut areas, we recommend that the exposed subgrade in areas that will require less than 2 feet of fill first be reviewed by the Project Geotechnical Engineer for the presence of highly plastic clays and unweathered bedrock, which have the potential of swelling with increases in moisture content, and which can result in heave of building foundations and floor slabs. We recommend that test pits be excavated to evaluate the extents of these materials across the building subgrade. If highly plastic clays or unweathered bedrock are encountered, we recommend that they be undercut to a depth of at least 2 feet below proposed floor slab subgrade elevations, and then be replaced with lean clay soils having plasticity indices of 22 percent or less, or with free-draining granular soils as is discussed subsequently. This recommendation was implemented successfully in the NKU Student Union structure, which, to our knowledge, has not experienced floor slab heave issues.

The base of the undercuts, as well as the remaining cut areas outside of the undercuts, should then be thoroughly proofrolled using a heavily loaded piece of equipment under the review of the Project Geotechnical Engineer, or a representative thereof. Soft or yielding soils observed during the proofrolling should be undercut to stiff non-yielding cohesive soils. If cohesionless soils are used for backfilling undercuts, the base of the undercut should be graded to drain towards a gravity outlet that will provide permanent subsurface drainage of the granular-filled undercut. The Project Geotechnical Engineer should also provide recommendations for the permanent subsurface drainage system(s) based on site conditions at the time of undercutting. The cohesionless soil should consist of free-draining granular material containing less than 3 percent fines, and should be separated from overlying and underlying cohesive soils with a non-woven filtration geotextile (such as Mirafi 140N or approved equivalent) to mitigate the migration of fines into the cohesionless soil over time.

Preferably, the undercuts should be backfilled with new compacted clayey fill satisfying the material and compaction requirements presented in this section. The undercut soils may be reused provided that they conform to the recommendations contained in this report regarding acceptable fill materials. We recommend that the Contract Documents include a bid item for the recommended undercutting, as deemed necessary, and the replacement with new compacted and tested fill on a "per cubic yard of in-place compacted fill" basis.

Fill materials should consist of approved on-site lean clayey soils, bedrock, or approved lean clay borrow materials that are relatively free of topsoil, vegetation, trash, construction or demolition debris, frozen materials, particles over 6 inches in maximum dimension, or other deleterious materials. Moderately to highly plastic clays that are obtained from cut areas or from utility excavations may be reused as new fill in paving or landscaping areas outside of the building limits, and even then should be restricted to at least 2 feet below pavement subgrade elevations.

The shale and limestone bedrock may be incorporated into the fill provided that the gray, unweathered shale is first pulverized to a soil-like consistency and then moisture-conditioned, and provided that the limestone is broken up and dispersed so as not to cause nesting or retard compaction. The maximum dimension of the broken-up limestone floaters in the fills should be limited to 18 inches, with a maximum thickness of 6 inches. Thicker layers or larger pieces of limestone, if not capable of being broken up, should be wasted off site. Additionally, limestone floaters should be restricted from the fill in the upper 2 feet below floor slab subgrade elevations within and up to 5 feet outside of the structure footprint. In pavement areas, we recommend that limestone floaters be restricted within 1 foot of pavement subgrade elevations.

The fill should be placed in shallow level lifts (or layers), 6 to 8 inches in loose thickness. Each lift should be moisture-conditioned to within the acceptable moisture content range provided in Table 1 (on the following page), and then compacted with a sheepsfoot roller or self-propelled compactor to at least the minimum percent compaction indicated in the same table. Moisture-conditioning may include aeration and drying of wetter soils, wetting of drier soils, and/or thorough mixing of wetter and drier soils into a uniform mixture. Additionally, if shale is used in the fill, water will likely need to be blended with the shale to moisture-condition it. Where free-draining granular backfill is used to backfill undercuts, it should be compacted to at least the minimum relative densities indicated in Table 2 (on the following page).

Where fill is placed on sloping terrain that is steeper than 4H:1V, the fill should be placed on continuous horizontal benches up the sloping terrain, with the initial bench having a minimum width of 15 feet and all subsequent benches being at least 5 feet wide. The initial 15-foot wide bench should be located at the toe of the proposed fill, unless noted otherwise. The benching operations should remove surficial medium stiff or softer soils and expose stiff native soils or undisturbed, intact bedrock on the surfaces of the benches. The benches should not be made until the fill is ready to be placed. If groundwater seepage is noted on the benches, the Project Geotechnical Engineer should be contacted for underdrainage recommendations before the soils are replaced and compacted.

Table 1. Percent compaction and moisture-conditioning requirements for fill and backfill.

Area	Minimum Percent Compaction[a,b]	Acceptable Moisture Content Range[c]
Structural[d]	98% SPMDD	-2% to +3% of OMC
Non-structural	95% SPMDD	±3% of OMC
Floor slab subgrade	98% SPMDD	0% to +3% of OMC
Pavement subgrade ≤ 12 inches below subgrade	100% SPMDD	0% to +2% of OMC

[a] SPMDD = standard Proctor maximum dry density determined from ASTM D698.
[b] For granular soils that do not exhibit a well-defined moisture-density relationship, refer to Table 2 for minimum relative density requirements.
[c] OMC = optimum moisture content determined from ASTM D698.
[d] Structural fill and backfill for foundations are defined as fill and backfill located within the zones of influence of existing and proposed structures. The zone of influence of a structure is defined as the area below the footprint of the structure and 2H:1V downward and outward projections from the bearing elevation of the structure.

Table 2. Relative density compaction requirements for granular fill and backfill.

Area	Minimum Relative Density[a,b]
Structural[c]	80%
Non-structural	75%
Floor slab and pavement subbase	80%

[a] Relative density evaluated on the basis of the maximum and minimum index densities determined from ASTM D4253 and D4254, respectively.
[b] For granular soils that exhibit a well-defined moisture-density relationship, refer to Table 1 for minimum percent compaction and moisture-conditioning requirements.
[c] Structural fill and backfill for foundations are defined as fill and backfill located within the zones of influence of existing and proposed structures. The zone of influence of a structure is defined as the area below the footprint of the structure and 2H:1V downward and outward projections from the bearing elevation of the structure.

We recommend that the permanent cut and fill slopes for this project be designed not steeper than 3H:1V. Gentler slopes should be used whenever possible for ease of maintenance. Additionally, we recommend that the fill slopes be slightly overbuilt and then trimmed back to the design slope to achieve a well-compacted surface. Silt and/or sand soils should also be excluded from the face of the fill slopes, as these materials are more susceptible to erosion.

Topsoil should be track-compacted on the proposed cut and fill slopes. We recommend that a maximum of 6 inches of topsoil be placed on the slopes. We note that bedrock exposures at proposed grades may not consistently hold the topsoil layer, and small pop-outs may occur, especially at points of seepage.

Groundwater is not expected to have an adverse effect on the proposed earthwork construction. However, the Contractor must be prepared to remove seepage that accumulates during excavation on fill surfaces or at subgrade levels.

Maintaining the moisture content of cohesive bearing and subgrade soils within the acceptable range provided in Table 1 is very important during and after construction for the proposed structure. The clayey bearing and subgrade soils should not be allowed to become excessively wet or dried during or after construction, and measures should be taken to prevent water from ponding on these soils and to prevent these soils from desiccating during dry weather.

Positive drainage should be established to promote the rapid drainage of surface water away from the structure, and to prevent the ponding of water adjacent to the structure. Finish grading in grass and landscaped areas should be sloped down and away from the structure at a gradient of at least 10 percent for at least 10 feet, and then at a gradient of at least 2 percent beyond the initial 10 feet from the structure. Proposed pavements should drain away from the structure at a minimum of 2 percent. The final grades should direct the surface water to storm water collection systems.

Deep-rooted vegetation should not be planted within 1.5 times their projected mature foliage radius from foundations, as the roots of such vegetation can extract moisture from plastic and low-plastic soils alike, causing them to shrink, which can potentially create foundation and floor slab settlement issues. Additionally, smaller bushes or flowerbeds adjacent to the proposed structure should not be watered by ponding water in the beds where the bushes or flowers may be growing, which could lead to swelling and heave of the foundation soils.

We recommend that the earthwork operations be carried out during the drier season of the year. In our experience, weather conditions are historically more favorable for earthwork during the months of May through October in the Northern Kentucky Area. Regardless of the time of year, asphalt, concrete, or fill should not be placed over frozen or saturated soils, and frozen or saturated soils should not be used as compacted fill or backfill.

Best management practices (BMPs) should be implemented to reduce the effects of erosion and the siltation of adjacent properties. Upon completion of earthwork, disturbed areas should be stabilized. It is also recommended that riprap and/or suitable armoring be used at the outlets of storm sewers and headwalls to reduce flow velocities and protect against erosion.

7.4 Site Classification and Seismic Design Category

Based on the borings and our interpretation of the 2018 Edition of the Kentucky Building Code (2018 KBC), it is our opinion that the site class and seismic parameters in Table 3 are applicable for this project.

Table 3. Site class and seismic design category per the 2018 Kentucky Building Code (2018 International Building Code).

Category/ Parameter	Designation/ Value	Notes
S_S	0.158 g	Campbell County, Kentucky, per Table 1613.3.1 of the 2018 KBC
S_1	0.081 g	
Site Class	D	Per Chapter 20 of ASCE 7
F_a	1.2	Per Table 11.4-1 of ASCE 7
F_v	1.7	Per Table 11.4-2 of ASCE 7
S_{MS}	0.176 g	Per Equation 11.4-1 of ASCE 7
S_{M1}	0.134 g	Per Equation 11.4-2 of ASCE 7
S_{DS}	0.118 g	Per Equation 11.4-3 of ASCE 7
S_{D1}	0.090 g	Per Equation 11.4-4 of ASCE 7

7.5 Foundation Design and Construction

Relatively heavy column and wall loads are planned for this project. Variable thicknesses of uncontrolled fill with localized low-density sediment soils were encountered in some of the borings, and shallow bedrock was encountered within 1 foot of the proposed finish floor elevation in Boring 8. In our opinion, intolerable differential settlements may be expected if the proposed structure is supported on shallow spread footings bearing in or over the undocumented fill. In addition, differential settlements would be exacerbated in areas where shallow foundations would bear on the bedrock and settlements would be negligible. Therefore, we recommend that the structure be supported either on combination of drilled shafts and shallow spread footings extended to or into the bedrock, or on shallow spread footings bearing on the bedrock or on fill soils improved by rammed aggregate piers (RAPs). The elevations of the bedrock surface can be estimated from the approximate bedrock surface contours depicted on Sheet 1 in Appendix B. Continuous footings may be used to support wall loads to a depth of roughly 5 feet below subgrade levels existing at the time of excavation. Narrow trenches excavated to extend shallow foundation bearing depths to more than about 5 feet become difficult to keep open and to keep free of sloughed soils prior to concrete placement. Where it may be necessary to extend foundations to depths of 5 feet or more to reach bedrock, drilled shafts or RAPs should be used.

Section 7.5.1 discusses the shallow foundation components. Sections 7.5.2 and 7.5.3 discuss the drilled shaft and RAPs, respectively.

7.5.1 Shallow Foundations on Bedrock

Shallow foundations can be used when suitable bearing materials are encountered at shallow bearing elevations. Shallow foundations should consist of continuous wall footings and isolated column pads bearing on or in undisturbed, intact bedrock. Footings bearing on highly weathered, weathered, and unweathered bedrock may be proportioned for respective maximum net allowable bearing pressures of 6,000, 10,000, and 30,000 pounds per square foot (psf), full dead and full live load. We recommend that the minimum lateral dimensions for continuous wall footings and isolated column footings be at least 18 and 24 inches, respectively.

Exterior footings and footings in unheated interior areas should bear at least 30 inches below the lowest adjacent exterior/unheated grade for protection from frost penetration. Additionally, the foundation bearing elevations should not be located higher than a relationship of 2H:1V above proposed adjacent foundations or the inverts of nearby existing or proposed utilities that parallel or nearly parallel the foundations, without a site-specific evaluation of the conditions by the Project Geotechnical Engineer.

We recommend that foundation excavations be cut to neat lines and grades so that concrete may be placed directly against the banks of the excavations without forming. Loose, soft, wet, frozen, or otherwise disturbed materials should be removed from the bearing surfaces of the foundations prior to the placement of reinforcing steel and concrete. If a limestone layer is exposed in the bottom of the footing excavation, we recommend that the excavation be deepened to penetrate the limestone layer, unless it can be determined that there is no softening of the shale beneath the limestone. Additionally, disturbed or loosened beds of limestone should be removed from the bearing surfaces. If a crusted or saturated surface develops at a foundation bearing surface, we recommend that it be skimmed to expose a fresh surface before reinforcing steel and concrete are placed. Foundation concrete should be placed the same day as the excavation is made to prevent saturation or desiccation of the bearing surfaces.

Concrete mud mats may be placed over the bearing surfaces to protect the bearing materials from desiccation or softening via saturation. If concrete mud mats are utilized, the concrete should have a minimum compressive strength of 1,500 psi and a minimum thickness of 2 inches. The excavated bearing surface should be lowered at least the thickness of the mud mat, and the top of the mud mat should be at or below the design bearing elevation of the foundation. Prior to the placement of the concrete mud mat, the bearing surfaces should be cleaned of loose, soft, wet, frozen, or otherwise disturbed material.

Water should not be allowed to pond on top of the bedrock within footing excavations, or on or around completed footings, in order to mitigate potential softening or swelling of the bearing materials.

We recommend that foundation steps have a maximum height of 2 feet and a corresponding minimum length of 4 feet. Reinforcing steel and concrete should remain continuous through the foundation steps.

We recommend that foundation excavations be reviewed by the Project Geotechnical Engineer or his representative prior to placing concrete in order to confirm that the bearing materials and surfaces are consistent with the design recommendations of this report.

Refer to Section 7.6 for a discussion of lateral earth pressures.

7.5.2 Drilled Shafts

Axial load capacity for drilled shafts may be provided by the allowable base resistance values provided in Table 4. We recommend that drilled shafts bear at least 3 times the shaft diameter below the ground surface and grade beams, where applicable. We recommend that drilled shafts be spaced at least 2 times the shaft diameter on-center, unless an accounting is made for group effects. If the drilled shafts need to be designed to resist uplift loads, it is our opinion that some uplift resistance can be provided by the adhesion between the drilled shaft concrete and the soil and bedrock strata that they penetrate. Table 4 also summarizes recommended allowable adhesion values for the overburden and bedrock strata, which may be used in combination with an appropriate safety factor.

Table 4. Drilled shaft base and adhesion parameters.

Soil or Bedrock Type	Maximum Net Allowable Base Resistance, $\sigma_{b,all}$ (psf)	Minimum Embedment Depth Below Top of Stratum	Allowable Adhesion (psf)[a]
New Structural Fill	--	--	320
Uncontrolled Fill	--	--	100
Stiff to Very Stiff Native Soils	--	--	400
Highly weathered shale bedrock	6,000	6 inches	750
Weathered shale bedrock	10,000	6 inches	1,200
Unweathered shale bedrock	30,000	6 inches	1,500
Unweathered shale bedrock	80,000	2 feet	1,500

[a] Side resistance should be ignored within 5 feet of the proposed ground surface grades.

Where the drilled shafts will be supporting lateral loads, the drilled shafts should be designed using a p-y approach. Table 5 provides cohesive soil parameters for p-y analyses of laterally loaded deep foundation elements, while Table 6 provides p-y parameters for the shale bedrock.

Lateral resistance for deep foundations should be ignored above the frost line (i.e., above a depth of 30 inches from the ground surface).

Where the spacing of laterally loaded deep foundations will be close enough that their areas of resistance overlap (i.e., less than 5 times their shaft diameter), we recommend that an appropriate p-multiplier be applied in the analyses to account for the overlap and reduction in lateral resistance. We recommend that the p-multiplier be estimated per Section 10.7.2.4-1 from the *AASHTO LRFD Bridge Design Specifications* (AASHTO 2012).

Table 5. Cohesive soil parameters for p-y analyses of laterally loaded deep foundation elements.

Soil Description from Boring Log	p-y Curve Model/ Material Type	Unit Weight, γ (pcf)	Saturated Unit Weight, γ_{sat} (pcf)	Cohesion, c (psf)	Friction Angle, ϕ (°)	Strain, ε_{50}	Initial Horizontal Subgrade Reaction, k (pci)	
							Static	Cyclic
Uncontrolled Fill Soils	Soft Clay	120	125	750	-	0.01	100	-
Stiff Native Soils or New Compacted Fill	Stiff Clay	125	130	1,500	-	0.007	500	200
Highly Weathered Shale Bedrock	Stiff Clay	140	140	4,500	-	0.002	-	-

[a] Highly weathered shale bedrock should be modeled with a stiff clay model. See Table 6 for p-y parameters for other bedrock types.

Table 6. Bedrock parameters for p-y analyses of laterally loaded deep foundation elements.

Bedrock Description from Boring Log	p-y Curve Model/ Material Type	Unit Weight, γ (pcf)	Uniaxial Compressive Strength, q_u (psf)	Initial Modulus of Rock Mass, E_m (psi)	Strain, ε_{50} or Strain Factor, k_{rm}
Highly Weathered Shale Bedrock	See Table 5.				
Weathered Shale Bedrock	Weak Rock	140	12,000	4,165	0.0005
Unweathered Shale Bedrock	Weak Rock	150	30,000	10,400	0.0005

Drilled shaft excavations should be made straight and plumb with level bottoms, using dry construction methods. Loose, soft, wet, or otherwise disturbed materials should be removed from the bearing surfaces to expose undisturbed bedrock before the reinforcing steel and concrete are placed. Concrete should not be placed through more than 3 inches of water in the bottom of any shaft, and the rate of inflow of groundwater should be less than 12 inches per hour, unless wet construction methods are implemented. We recommend that each drilled shaft excavation be reviewed by the Project Geotechnical Engineer or his representative to confirm that the soil and bedrock conditions encountered within the drilled shaft are consistent with those encountered in the borings and with the design recommendations of this report.

Considering the groundwater conditions encountered in the borings, full-depth temporary casing from the ground surface to the top of bedrock may be needed to control groundwater and/or caving overburden soils. We recommend that the Contract Documents include a bid item for casing shafts as recommended by the Project Geotechnical Engineer or his representative on a cost-per-cased-shaft basis.

Bottoms of grade beams should extend 30 inches below proposed exterior grades. Similar to the shallow foundations on bedrock, grade beams between drilled shafts should be excavated to neat lines and grades so that concrete may be placed directly against the banks of the excavations without forming. If the excavation becomes desiccated prior to placement of concrete, the sides and bottoms of the excavation should be trimmed to expose fresh, moist soils to reduce the potential of the desiccated soils absorbing water and swelling, resulting in uplift pressures on the grade beams.

7.5.3 Rammed Aggregate Piers

The proposed structure can also be supported by spread footings bearing on new and existing fill and native soils reinforced by rammed aggregate pier (RAP) elements. In this case, the piers would be constructed by augering 24- to 36-inch diameter holes to the bedrock surface and then backfilling the holes with thin lifts of compacted aggregate. Compaction is achieved using high-frequency impact hammers that deliver vertical ramming energy that densifies the aggregate and forces it laterally into the sidewalls of the hole. This action increases the lateral stresses in the surrounding soil, further stiffening the stabilized composite soil mass. The result of the RAP installation is to strengthen and stiffen the subsurface soils that then support the footing and floor slab loads.

RAP construction may reduce time and cost for foundation placement as compared to other deep foundation systems because 1) conventional spread and wall foundations are placed directly on the reinforced soil mass, and 2) there is no set up time for the aggregate pier elements.

Typically, a net allowable bearing pressure for shallow spread footings of 4,000 to 6,000 psf is available where RAP elements are used to reinforce the subsurface soil profile.

If a RAP-enhanced foundation system is selected, we recommend that the following issues be considered prior to construction:

- Specifications for rammed aggregate pier foundation systems should be prepared by a design/build RAP contractor.

- At a minimum, the design/build RAP contractor should evaluate the composite soil mass following installation of the RAPs to confirm the allowable shallow foundation bearing capacity and to estimate maximum total and differential settlements. (Typically, settlements of the aggregate-pier supported footings are estimated to be on the order of 1 inch or less.)

- All of the RAP element installations, as well as the post-installation evaluation testing of the composite soil mass, should be reviewed by the Project Geotechnical Engineer or his representative to verify proper installation procedures and to document observed changes in the explored soil conditions.

Exterior shallow foundations bearing on the RAP-reinforced soil should bear at least 30 inches below the lowest adjacent exterior/unheated grade for protection from frost penetration unless moderately to highly plastic clays are encountered at bearing elevations. In this case, the bearing elevation should be lowered to a depth of 42 inches below final exterior grade to further mitigate potential shrinkage or heave of the plastic soils.

After the foundation soils have been reinforced with RAP elements, adequate protection of the reinforced ground is required, including proper drainage to mitigate ponding water and maintenance of minimum excavation distances from the installed piers, as per the design/build RAP contractor's recommendations. Prior to foundation installation, the reinforced ground surface should be cleared and cleaned to the satisfaction of the Project Geotechnical Engineer or his representative.

Lateral loads can be resisted by passive soil pressures in accordance with Table 7. Additionally, a friction coefficient of 0.5 between the concrete footings and underlying aggregate pier-enhanced soil can be used in combination with passive earth pressures to resist lateral loads. The coefficient of friction should be applied to dead normal loads only.

7.6 Lateral Loads and Earth Pressures

Lateral load resistance for drilled shaft foundations using a p-y approach was discussed in Section 7.5.2. Where shallow foundations will be subjected to lateral loads, sliding resistance may be provided by a combination of friction and passive resistance. Frictional resistance can be estimated using an ultimate static friction coefficient between cast-in-place concrete and bedrock or cast-in-place concrete and RAP-enhanced soils of 0.50, in combination with an appropriate safety factor. The ultimate static friction coefficient should be applied to dead normal loads only.

Lateral loads may also be resisted by passive soil pressures acting against the portions of footings and grade beams below the 30-inch frost depth in accordance with Table 7.

Table 7. Lateral earth pressures for level (horizontal) ground surfaces.

	Active[a]	At-Rest[a]	Passive[a,b]
Lateral earth pressure coefficient, K	0.39	0.56	2.56
Drained equivalent fluid weight, EFW (pcf)	49	70	320
Undrained equivalent fluid weight, EFW_u (pcf)[c]	87	98	222

[a] Parameters are based on level ground surfaces, a soil unit weight (γ) of 125 pcf, and a soil internal angle of friction (ϕ) of 26 degrees.

[b] Passive resistance may be considered where concrete is cast against free-standing vertical faces of soil or bedrock, but should be ignored in the upper 30 inches below proposed grade due to seasonal variations in moisture and frost penetration. If the ground is sloping down and away from the foundation in the area of passive resistance, we should be contacted to provide site-specific recommendations.

[b] Includes hydrostatic pressure of 62.4 pcf.

Where foundation and retaining walls for this project will be subjected to unbalanced lateral earth pressures, we recommend that the lateral earth pressures be computed on the basis of equivalent fluid weights of the backfill, plus surcharges for pavement loads, sloping backfill, etc. Table 7 provides the recommended equivalent fluid weights for soil under drained and undrained conditions, and also the recommended earth pressure coefficients for proposed surcharges. Unless a site-specific analysis is performed, we recommend that surcharges be modeled as a uniform horizontal pressure equal to the vertical intensity of the surcharge multiplied by the recommended lateral earth pressure coefficient.

The values provided in Table 7 assume that the ground surface above the top of the wall is level and not sloping toward the wall. For ground sloping behind the wall on its active or at-rest side, we recommend that it be accounted for as a surcharge on the wall, as discussed above, unless site-specific equivalent fluid weights are computed on the basis of the backfill slope.

The decision to use active or at-rest earth pressures should be based on the ability of the wall to deflect as a result of the lateral earth pressures. In cohesionless granular backfill, active earth pressures are assumed to be applicable if the top of the wall is able to deflect a minimum of 0.002 times the height of the wall. In cohesive clayey backfill, the minimum deflection at the top of the wall for active earth pressures to develop is 0.02 times the height of the wall. If these minimum horizontal deflections at the top of the wall are restrained from occurring or are unacceptable to the structure, at-rest earth pressures are applicable.

Undrained equivalent fluid weights should be used in computing the lateral loads on the wall wherever the backfill is unable to be drained by a drainage system (discussed below). For the drained equivalent fluid weights to be applicable, a drainage system should be incorporated along the backfilled face of the wall (i.e., the high side of the wall) consisting of either a prefabricated drainage board or an approximately 18-inch width of free-draining gravel with less than 3 percent fines wrapped with a non-woven drainage geotextile. At the base of the drainage board or free-draining gravel should be a minimum 12-inch-thick by 12-inch-wide gravel zone wrapped with a non-woven drainage geotextile. Within the wrapped gravel at its base should be a 4-inch-diameter rigid perforated plastic pipe. The plastic pipe should be connected to a suitable gravity outlet (e.g., the proposed storm sewer system). The granular backfill should be compacted to at least 75 percent relative density per ASTM D4253 and D4254. We recommend that the drainage system extend to subgrade elevation beneath pavements or floor slabs; otherwise the drainage system should extend to within 2 feet of finished grade and be capped with at least 2 feet of compacted clayey soils to reduce the infiltration of surface water behind the wall. Clayey backfill should be compacted per the requirements presented in Table 1. The drainage system should not connect to interior drainage systems below floor slabs. These interior drainage systems should have separate, independent outlets.

In the case of exterior retaining walls that are subject to freezing temperatures, clayey backfill will be subject to freezing that may result in frost heave pressures against the wall. This can be mitigated either by using free-draining granular backfill against the exterior wall in lieu of clayey backfill and a manufactured drainage mat, or by installing a minimum 1.5-inch thickness of rigid, polystyrene foamboard insulation between the backfilled wall face and the manufactured drainage mat.

7.7 Utility Construction

Excavation difficulty in utility trenches will vary with location, depth of utility, and depth of previous cuts made during original bulk grading of the native topography. The combined depths of previous bulk grading cuts and planned utility trenches will likely extend into the weathered and unweathered bedrock in certain areas of the site, based on the borings and on comparison of existing and pre-development topography. Because of the anticipated limestone percentages that will be encountered in the bedrock, there will be excavation difficulties within the utility trenches. The difficulty of making the trench excavations in the highly weathered to weathered bedrock arises because of the need to shear limestone layers from the bottoms and sides of the trenches. The excavation difficulties will substantially increase in the trenches that penetrate into the unweathered bedrock. Excavations in the unweathered bedrock will necessitate the use of large trackhoes with ripping teeth and/or the use of rock saws or hoe rams.

Section 7.3 discussed ongoing floor slab heave issues that have historically affected structures at NKU having floor slabs supported on fresh exposures of gray, unweathered shale as it has absorbed moisture following construction. Backfilling of utility trenches with granular soils or poorly-compacted clayey soils has contributed to these problems as subsurface groundwater seepage has accessed highly plastic overburden soils and/or the unweathered shale exposed in the trench sidewalls. We recommend that utility trenches within and up to 5 feet outside of the new structure limits be backfilled full height with flowable fill having a minimum design strength of 30 psi and a maximum design strength of 100 psi for future excavatability. This recommendation was implemented successfully in the NKU Student Union structure, which again, to our knowledge, has not experienced floor slab heave issues.

Prior to placing the bedding and/or utilities within the utility trench, soft, saturated, and compressible material should be removed from the bottom of the trench, exposing moist stiff soils or undisturbed bedrock.

We anticipate that select granular backfill will be used as pipe bedding and pipe zone backfill for the utility trenches occurring more than 5 feet outside of the structure limits. We recommend that the granular backfill be limited to the pipe bedding and minimum required pipe/utility cover. The remainder of the utility trenches should be backfilled with flowable fill or compacted clayey soils up to design subgrade elevation to reduce the potential for water collecting in these trenches and being absorbed by the surrounding clays and causing pavement heave.

Granular bedding and backfill that exhibits a well-defined moisture-density relationship should be compacted and moisture-conditioned per the requirements presented in Table 1; otherwise, the granular material should be compacted to at least the minimum relative densities indicated in Table 2. Utility trench backfill should be placed in 6- to 8-inch thick lifts with each lift compacted to at least the specified degree of compaction. Under no circumstances should the backfill be flushed in an attempt to obtain compaction.

7.8 Floor Slabs

We anticipate that the floor slabs for the building will be designed as slab-on-grade concrete. The concrete floor slab thicknesses should be designed based on the native or compacted and tested, stiff soils at this site providing a modulus of subgrade reaction (k) of 75 pounds per cubic inch (pci).

As discussed in Section 6.1 of this report, we encountered CH (highly plastic clay) soils and unweathered bedrock within our borings. Where these materials are encountered at proposed floor slab subgrade elevation, refer to Section 7.3 for additional floor slab recommendations.

We recommend that the floor slab be underlain by a minimum 4-inch-thick granular subbase layer and by a plastic vapor barrier to serve as a capillary break and to reduce the potential for groundwater rising beneath and into the floor slab from the clayey subgrade via capillary action. The use of a plastic vapor barrier is especially important in areas that will have glued floor coverings. We understand that current flooring practice is to use water-based floor tile glues, which in our experience, can unset with time as subsurface vapor penetrates the concrete floor slab and raises its moisture content. The effects of the vapor barrier on curling of the concrete floor slab should be considered in the mix design and placement of the concrete floor slab.

Immediately prior to placement of the granular base, we recommend that the top 8 inches of clayey floor slab subgrade be compacted and moisture-conditioned per the requirements presented in Table 1. The granular subbase should be compacted per the requirements presented in Table 1 or Table 2, whichever is applicable.

Care should be taken during slab-on-grade construction to not allow the subgrade to become desiccated or saturated. Additionally, consideration should be given to the timing of construction relative to the time of year and weather. If the slab construction is performed during relatively cold and wet weather, the use of lime- or cement-treatment of the subgrade may be beneficial to maintain progress during construction; otherwise, the subgrade is likely to be weakened by softening from saturation of rain weather, leading to delays in reworking the subgrade to prepare it back to its pre-softened condition. A cost-benefit analysis may need to be performed to evaluate the need for lime- or cement-treatment.

We recommend that control joints be provided within the concrete slab-on-grade floors. These joints should be sealed to mitigate surface water infiltration until the building is enclosed. We recommend that the floor slab be structurally separated from walls, columns, footings, and penetrations to allow independent movement of the floor.

7.9 Pavement Design and Construction

Pavements for this project should be designed in accordance with expected axle loads, frequency of loading, and subgrade properties. The subgrade properties should be evaluated by field California Bearing Ratio (CBR) or plate load tests after final grading is completed, or by the correlation of field density tests to laboratory CBR tests.

Proposed pavement subgrades should be proofrolled with a heavily loaded piece of equipment under the review of the Project Geotechnical Engineer, or representative thereof. Soft or yielding soils observed during the proofrolling should be undercut to stiff, non-yielding soils; however, the depth of undercut below final subgrade elevations may be limited to 3 feet in light-duty traffic areas and 4 feet in heavy-duty traffic areas. The undercut should be backfilled with new compacted fill satisfying the material and compaction requirements presented in Section 7.3. We recommend that the Contract Documents include an item for undercutting unsuitable soils and replacing them with new compacted and tested fill on a “per cubic yard of compacted replacement fill” basis.

If soft or yielding soils are encountered at the maximum undercut depths specified above (i.e., 3 feet for light-duty traffic and 4 feet for heavy-duty traffic), the soft or yielding subgrade may be stabilized at those depths using an approved biaxial or triaxial geogrid (e.g., Tensar BX-1200 or TriAx TX160) and an 8-inch lift of compacted crushed stone. The remainder of the undercut should be backfilled with dense-graded aggregate or with clayey soils satisfying the material and compaction requirements presented in Section 7.3. If clayey soils are used, an approved separation geotextile should be provided at the interface between the crushed stone and the clayey soils.

Prior to the placement of pavement or an aggregate base, where provided, we recommend that the top 8 inches of clayey subgrade be scarified and recompacted per the requirements presented in Section 7.3.

If the proposed pavement section includes an aggregate base, we recommend that caution be exercised so that the proposed aggregate base does not become saturated during or after construction. Water trapped in the granular base is capable of freezing, causing it to expand within the voids it occupies. Consequently, ice lenses may form and potentially heave the pavement. Furthermore, the thawing process can soften underlying cohesive subgrades, which reduces the pavement support provided by the subgrade, giving rise to “pumping” of the pavements under loads. Preferably, the aggregate base should be a free-draining material with provisions for draining the base through a system of underdrains.

Surface drainage should be directed away from the edges of proposed or existing pavements so that water does not pond next to pavements or flow onto pavements from unpaved areas. Such ponding or flow can cause deterioration of pavement subgrades and premature failure of pavements. If drainage ditches are used to intercept surface water before it reaches the pavements, the ditches should have an invert at least 6 inches below the pavement subgrade, and have a sufficient longitudinal gradient to rapidly drain the ditches and prevent ponding of water. In those areas where exterior grades do not fully slope away from the edges of the proposed pavement, we recommend that edge drains be installed along the perimeter of the pavement.

If dumpsters are utilized at the project site, we recommend that the dumpster be supported on reinforced concrete slabs and that the slabs be sized to accommodate the loading wheels of the dumpster truck. The access lane to the dumpster should also be designed for the heavier wheel loads associated with dumpster trucks.

8.0 RECOMMENDED ADDITIONAL SERVICES

The conclusions and recommendations given in this report are based on Geotechnology’s understanding of the proposed design and construction, as outlined in this report; site observations; interpretation of the exploration data; and our experience. Since the intent of the design recommendations is best understood by Geotechnology, we recommend that Geotechnology be included in the final design and construction process, and be retained to review the project plans and specifications to confirm that the recommendations given in this report have been correctly implemented. We recommend that Geotechnology be retained to participate in prebid and preconstruction conferences to reduce the risk of misinterpretation of the conclusions and recommendations in this report relative to the proposed construction of the NKU Residence Hall.

Since actual subsurface conditions between boring locations may vary from those encountered in the borings, our design recommendations are subject to adjustment in the field based on the subsurface conditions encountered during construction. Therefore, we recommend that

Geotechnology be retained to provide construction observation services as a continuation of the design process to confirm the recommendations in this report and to revise them accordingly to accommodate differing subsurface conditions. Construction observation is intended to enhance compliance with project plans and specifications. It is not insurance, nor does it constitute a warranty or guarantee of any type. Regardless of construction observation, contractors, suppliers, and others are solely responsible for the quality of their work and for adhering to plans and specifications.

9.0 LIMITATIONS

This report has been prepared on behalf of, and for the exclusive use of, the client for specific application to the named project as described herein. If this report is provided to other parties, it should be provided in its entirety with all supplementary information. In addition, the client should make it clear that the information is provided for factual data only, and not as a warranty of subsurface conditions presented in this report.

Geotechnology has attempted to conduct the services reported herein in a manner consistent with that level of care and skill ordinarily exercised by members of the profession currently practicing in the same locality and under similar conditions. The recommendations and conclusions contained in this report are professional opinions. The report is not a bidding document and should not be used for that purpose.

Our scope for this phase of the project did not include any environmental assessment or investigation for the presence or absence of wetlands or hazardous or toxic materials in the soil, surface water, groundwater, or air, on or below or around this site. Our scope did not include an assessment of the effects of flooding and erosion of creeks adjacent to or on the project site.

The analyses, conclusions, and recommendations contained in this report are based on the data obtained from the subsurface exploration. The field exploration methods used indicate subsurface conditions only at the specific locations where samples were obtained, only at the time they were obtained, and only to the depths penetrated. Consequently, subsurface conditions may vary gradually, abruptly, and/or nonlinearly between sample locations and/or intervals.

The conclusions or recommendations presented in this report should not be used without Geotechnology's review and assessment if the nature, design, or location of the facilities is changed, if there is a substantial lapse in time between the submittal of this report and the start of work at the site, or if there is a substantial interruption or delay during work at the site. If changes are contemplated or delays occur, Geotechnology must be allowed to review them to assess their impact on the findings, conclusions, and/or design recommendations given in this report. Geotechnology will not be responsible for any claims, damages, or liability associated with any other party's interpretations of the subsurface data, or with reuse of the subsurface data or engineering analyses in this report.

The recommendations included in this report have been based in part on assumptions about variations in site stratigraphy that may be evaluated further during earthwork and foundation construction. Geotechnology should be retained to perform construction observation and continue its geotechnical engineering service using observational methods. Geotechnology cannot assume liability for the adequacy of its recommendations when they are used in the field without Geotechnology being retained to observe construction.

A copy of "Important Information about This Geotechnical-Engineering Report" that is published by the Geotechnical Business Council (GBC) of the Geoprofessional Business Association (GBA) is included in Appendix A for your review. The publication discusses some other limitations, as well as ways to manage risk associated with subsurface conditions.

APPENDIX A – IMPORTANT INFORMATION ABOUT THIS GEOTECHNICAL-ENGINEERING REPORT

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.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical-engineering study conducted for a civil engineer may not fulfill the needs of a constructor — a construction contractor — or even another civil engineer. Because each geotechnical- engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. No one except you should rely on 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 the Full Report

Serious problems have occurred because those relying on a geotechnical-engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

Geotechnical Engineers Base Each Report on a Unique Set of Project-Specific Factors

Geotechnical engineers consider many unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk-management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical-engineering report that was:

- not prepared for you;
- not prepared for your project;
- not prepared for the specific site explored; or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical-engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light-industrial plant to a refrigerated warehouse;
- 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. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical-engineering report is based on conditions that existed at the time the geotechnical engineer performed the study. *Do not rely on a geotechnical-engineering report whose adequacy may have been affected by*: the passage of time; man-made events, such as construction on or adjacent to the site; or natural events, such as floods, droughts, earthquakes, or groundwater fluctuations. *Contact the geotechnical engineer before applying this report to determine if it is still reliable.* A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ — sometimes significantly — from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide geotechnical-construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are Not Final

Do not overrely on the confirmation-dependent recommendations included in your report. *Confirmation-dependent recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations *only* by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's confirmation-dependent recommendations if that engineer does not perform the geotechnical-construction observation required to confirm the recommendations' applicability.*

A Geotechnical-Engineering Report Is Subject to Misinterpretation

Other design-team members' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Constructors can also misinterpret a geotechnical-engineering report. Confront that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing geotechnical construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical-engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make constructors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give constructors the complete geotechnical-engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise constructors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure constructors have sufficient time* to perform additional study. Only then might you be in a position to give constructors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and constructors fail to recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations," many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Environmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform an *environmental* study differ significantly from those used to perform a *geotechnical* 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 environmental problems have led to numerous project failures.* If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold-prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, many mold- prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical- engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; *none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.*

Rely, on Your GBC-Member Geotechnical Engineer for Additional Assistance

Membership in the Geotechnical Business Council of 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. Confer with you GBC-Member geotechnical engineer for more information.

GBC GEOTECHNICAL BUSINESS COUNCIL
of the Geoprofessional Business Association

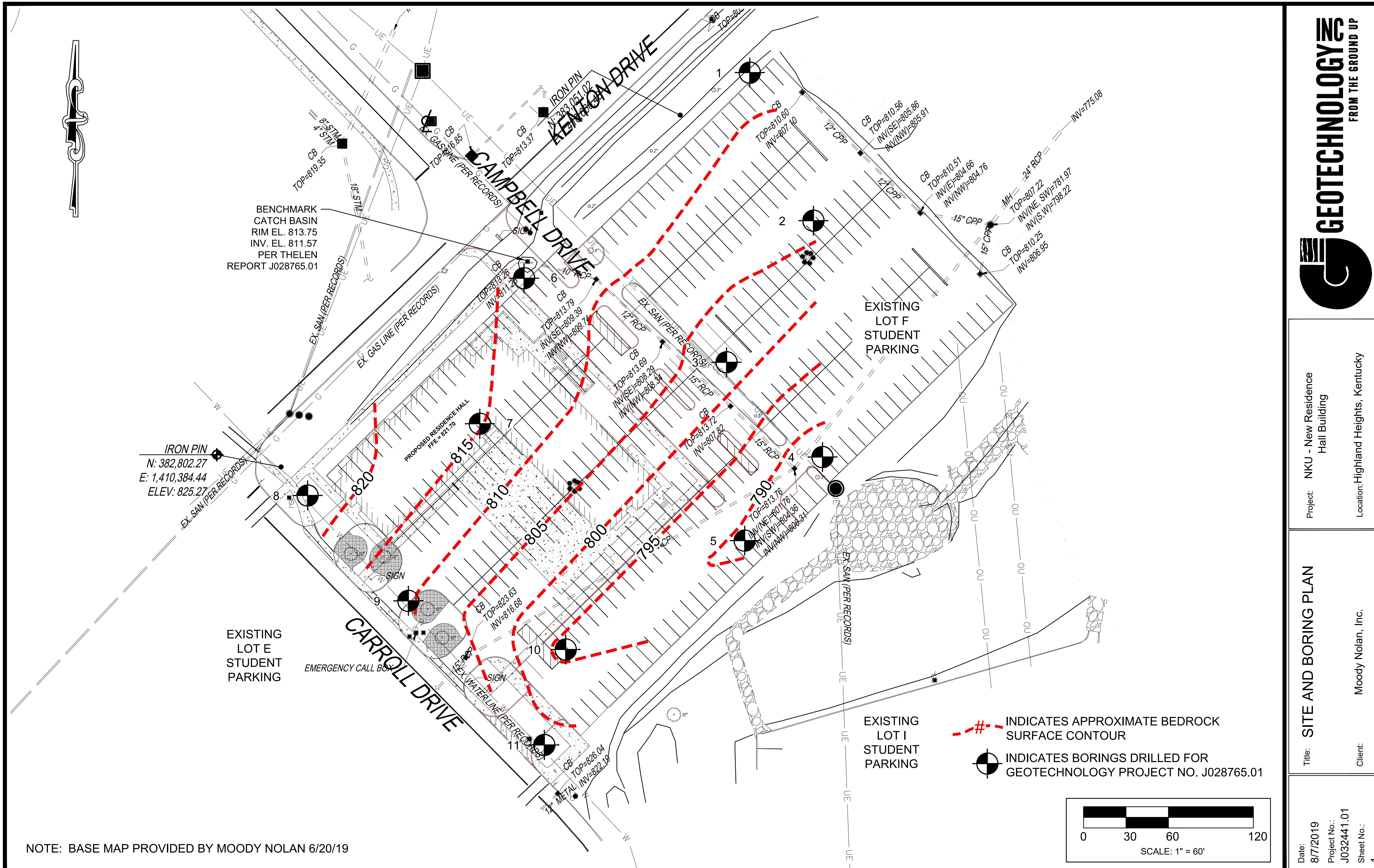
8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@geoprofessional.org www.geoprofessional.org

APPENDIX B – PLANS

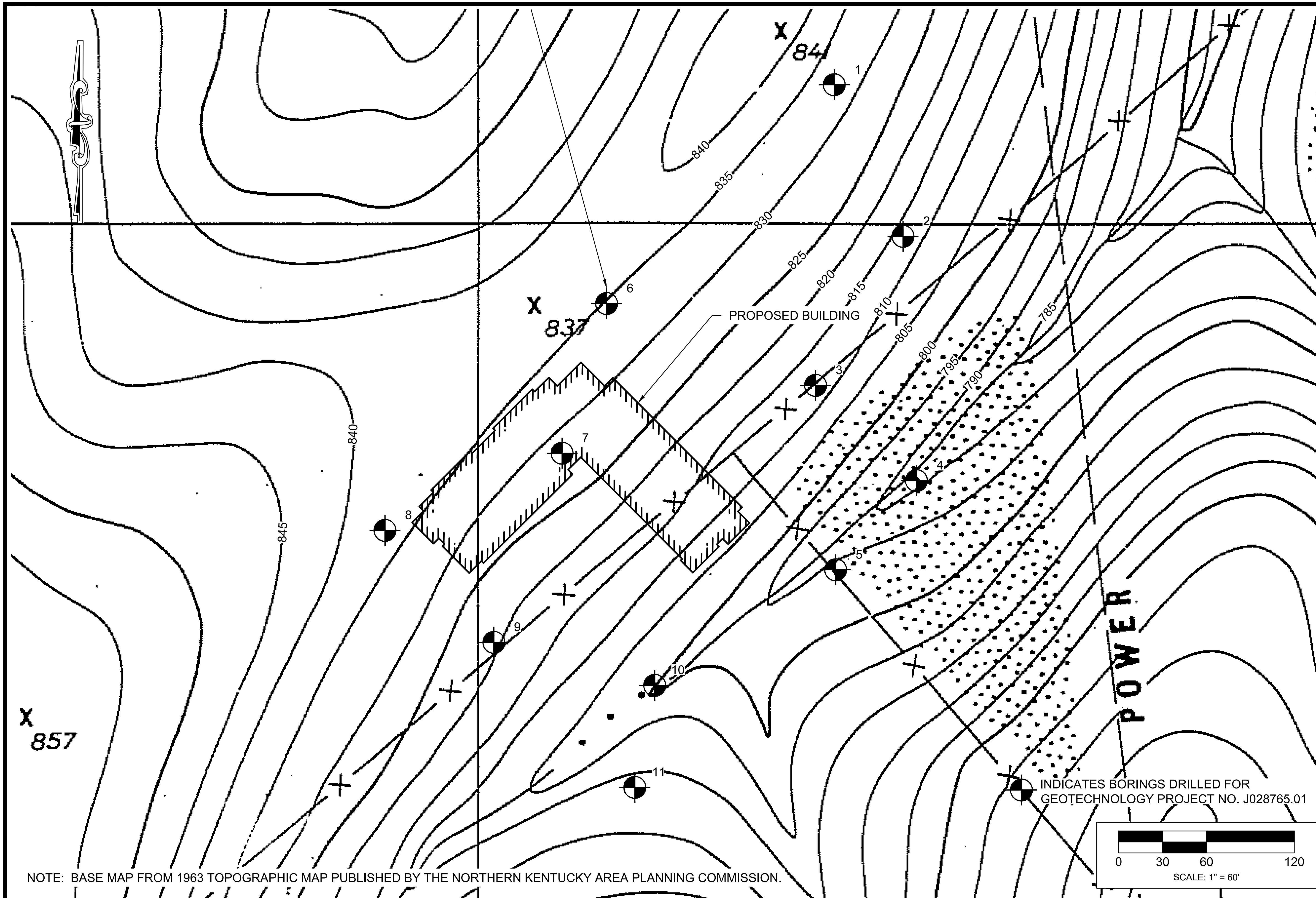
Boring Plan, Sheet 1

1963 Topography Plan, Sheet 2

Date Printed: 8/7/2019 8:07 AM Path: \\10.0.10.12\network storage\data\projects\J032\J032441.01-NKU - New Residence Hall building\Draw\J032441.01-Site and Boring plan.dwg



Date Printed: 8/7/2019 8:23 AM Path: \\10.0.10.12\network storage\data\projects\J032\J032441.01-NKU - New Residence Hall building\Draw\1963 topo with building.dwg



NOTE: BASE MAP FROM 1963 TOPOGRAPHIC MAP PUBLISHED BY THE NORTHERN KENTUCKY AREA PLANNING COMMISSION.

GEOTECHNOLOGY INC
FROM THE GROUND UP

Project: Geotechnical Exploration
NKU New Residence Hall Building

Location: Highland Heights, Kentucky

Title: 1963 TOPOGRAPHY PLAN

Client: Moody Nolan

Date: 8/7/2019

Project No.: J0320441.01

Sheet No.: 2

APPENDIX C – BORING INFORMATION

Boring Logs, Geotechnology Project No. J028765.01

Soil Classification Sheet

Rock Classification Sheet

THELEN ASSOCIATES
A DIVISION OF GEOTECHNOLOGY INC
1398 Cox Avenue, Erlanger, KY 41018-1002 / 859-746-9400 / Fax 859-746-9408
thelenassoc.com
geotechnology.com

LOG OF TEST BORING

CLIENT: American Campus Communities
PROJECT: Geotechnical Exploration, NKU Phase 1 Residence Hall
Highland Heights, Kentucky
LOCATION OF BORING: As shown on Boring Plan, Drawing 1

BORING #: 1
PROJECT #: J028765.01
PAGE #: 1 of 1

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Depth Scale (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
813.6	Ground Surface	0.0	0						
813.1	Mixed brown moist medium stiff FILL, topsoil and clay.	0.5							
	Mixed brown moist stiff FILL, silty clay with roots, some limestone floaters.			I	1A 1B	DS	3-4-5	18	100
811.6		2.0							
	Interbedded brown and gray slightly moist extremely weak weathered SHALE and gray medium strong to very strong LIMESTONE (bedrock).			I	2	DS	7-20-24	18	100
809.1		4.5							
808.1	Interbedded gray slightly moist extremely weak unweathered SHALE and gray medium strong to very strong LIMESTONE (bedrock).	5.5	5	I	3	DS	75/3"	2	67
	Bottom of test boring at 5.5 feet.								
			10						
			15						
			20						
			25						
			30						

Datum: NAVD 88
Surface Elevation: 813.6 ft.
Date Started: 11/23/2016
Date Completed: 11/23/2016
Hammer Weight: 140 lb.
Hammer Drop: 30 in.
Pipe Size: 2 in. O.D.
Hole Diameter: 8 in.
Rock Core Diameter: --
Boring Method: HSA-3.25
Drill Rig: CME 550 BD-1
Foreman: L. Wanstrath
Engineer: Mark A. Hushebeck

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted: None
At Completion: Dry
After: 24 hrs, Dry
Backfilled: 24 hrs.

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

THELEN ASSOCIATES

A DIVISION OF GEOTECHNOLOGY INC

1398 Cox Avenue, Erlanger, KY 41018-1002 / 859-746-9400 / Fax 859-746-9408

thelenassoc.com

geotechnology.com

LOG OF TEST BORING

CLIENT: American Campus Communities

BORING #: 2

PROJECT: Geotechnical Exploration, NKU Phase 1 Residence Hall
Highland Heights, Kentucky

PROJECT #: J028765.01

PAGE #: 1 of 1

LOCATION OF BORING: As shown on Boring Plan, Drawing 1

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Depth Scale (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
812.7	Ground Surface	0.0	0						
812.2	ASPHALT (6 inches)	0.5							
810.7	Brown, trace gray moist stiff SILTY CLAY, trace of rock fragments.	2.0		I	1	DS	4-5-20	18	100
808.2	Brown, trace gray moist stiff to very stiff SILTY CLAY, trace bedding planes (residuum).	4.5		I	2	DS	5-6-8	18	100
805.7	Brown, trace gray very moist very stiff CLAY, trace bedding planes (residuum) (CH).	7.0	5	I	3	DS	3-4-6	18	100
800.7	Interbedded brown moist extremely weak highly weathered SHALE and gray medium strong to very strong LIMESTONE, with clay layers (bedrock).	12.0	10	I	4	DS	10-15-33	18	100
				I	5	DS	50/3"	3	100
799.4	Interbedded gray and brown moist extremely weak weathered SHALE and gray medium strong to very strong LIMESTONE (bedrock).	13.3		I	6	DS	51-50/3"	6	67
	Bottom of test boring at 13.3 feet.		15						
			20						
			25						
			30						

Datum: NAVD 88	Hammer Weight: 140 lb.	Hole Diameter: 8 in.	Drill Rig: CME 550 BD-1
Surface Elevation: 812.7 ft.	Hammer Drop: 30 in.	Rock Core Diameter: --	Foreman: L. Wanstrath
Date Started: 11/23/2016	Pipe Size: 2 in. O.D.	Boring Method: HSA-3.25	Engineer: Mark A. Hushebeck
Date Completed: 11/23/2016			

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH

First Noted	None
At Completion	Dry
After	24 hrs, Dry
Backfilled	24 hrs.

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

THELEN ASSOCIATES
A DIVISION OF GEOTECHNOLOGY INC
1398 Cox Avenue, Erlanger, KY 41018-1002 / 859-746-9400 / Fax 859-746-9408
thelenassoc.com
geotechnology.com

LOG OF TEST BORING

CLIENT: American Campus Communities
PROJECT: Geotechnical Exploration, NKU Phase 1 Residence Hall
Highland Heights, Kentucky
LOCATION OF BORING: As shown on Boring Plan, Drawing 1

BORING #: 3
PROJECT #: J028765.01

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
814.7	Ground Surface	0.0						
814.3	ASPHALT (5 inches)	0.4						
810.2	Mixed brown, trace of gray moist stiff to very stiff FILL, silty clay and shale and limestone fragments.	4.5	I	1	DS	7-6-5	18	100
			I	2	DS	6-7-5	18	100
807.7	Mixed brown and gray moist to very moist medium stiff to stiff FILL, silty clay and shale and limestone fragments.	7.0	I	3	DS	7-8-20	6	33
802.7	Brown and gray moist very stiff SILTY CLAY, some oxide stains, trace shale fragments (colluvium).	12.0	I	4	DS	4-4-4	18	100
			I	5	DS	6-5-8	18	100
800.2	Interbedded brown to olive brown, trace of gray moist extremely weak weathered SHALE and gray medium strong to very strong LIMESTONE with clay layers (bedrock).	14.5	I	6	DS	58-52-50	18	100
798.2	Interbedded olive brown moist extremely weak weathered SHALE and gray medium strong to very strong LIMESTONE (bedrock).	16.5	I	7	DS	17-26-16	18	100
	Bottom of test boring at 16.5 feet.							

Depth Scale (feet): 0, 5, 10, 15, 20, 25, 30

Datum: NAVD 88
Surface Elevation: 814.7 ft.
Date Started: 11/23/2016
Date Completed: 11/23/2016
Hammer Weight: 140 lb.
Hammer Drop: 30 in.
Pipe Size: 2 in. O.D.
Hole Diameter: 8 in.
Rock Core Diameter: --
Boring Method: HSA-3.25
Drill Rig: CME 550 BD-1
Foreman: L. Wanstrath
Engineer: Mark A. Hushebeck

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH

First Noted	None
At Completion	Dry
After	24 hrs, Dry
Backfilled	24 hrs.

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

THELEN ASSOCIATES
A DIVISION OF GEOTECHNOLOGY INC
1398 Cox Avenue, Erlanger, KY 41018-1002 / 859-746-9400 / Fax 859-746-9408
thelenassoc.com
geotechnology.com

LOG OF TEST BORING

CLIENT: American Campus Communities
BORING #: 4
PROJECT: Geotechnical Exploration, NKU Phase 1 Residence Hall
Highland Heights, Kentucky
PROJECT #: J028765.01
PAGE #: 1 of 1
LOCATION OF BORING: As shown on Boring Plan, Drawing 1

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Depth Scale (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
814.3	Ground Surface	0.0	0						
813.8	ASPHALT (6 inches)	0.5							
813.3	GRANULAR BASE (6 inches)	1.0							
	Mixed brown and gray moist stiff to very stiff FILL, silty clay and shale and limestone fragments, some nested zones of fragments.			I	1	DS	2-6-7	18	100
810.3		4.0		I	2	DS	9-8-6	12	67
	Mixed brown moist stiff to very stiff FILL, silty clay, some shale and limestone fragments.		5	U	3	PT		16	67
808.3		6.0							
	Mixed brown and gray moist to very moist medium stiff FILL, silty clay, limestone fragments, and shale fragments.			I	4	DS	5-5-5	18	100
806.8		7.5							
	Mixed brown and gray moist to very moist stiff to very stiff FILL, clay, some limestone and shale fragments, some gray shale fragment layers (CH).			I	5	DS	3-5-4	18	100
			10	I	6	DS	3-6-6	18	100
				I	7	DS	2-3-6	18	100
			15	I	8	DS	7-5-4	18	100
				U	9	PT		21	88
794.8		19.5							
	Mixed brown and gray moist to very moist stiff FILL, silty clay, little shale and limestone fragments.		20	I	10	DS	7-6-8	18	100
790.3		24.0							
	Mixed gray and brown moist stiff to very stiff FILL, silty clay and shale fragments and limestone floaters.		25	I	11	DS	9-16-10	18	100
786.3		28.0							
	Interbedded gray slightly moist extremely weak unweathered SHALE and gray medium strong to very strong LIMESTONE (bedrock).								
784.0		30.3	30	I	12	DS	75/3"	3	100
	Bottom of test boring at 30.3 feet.								
			35						

Datum:	NAVD 88	Hammer Weight:	140 lb.	Hole Diameter:	8 in.	Drill Rig:	CME 550 BD-1
Surface Elevation:	814.3 ft.	Hammer Drop:	30 in.	Rock Core Diameter:	--	Foreman:	L. Wanstrath
Date Started:	11/23/2016	Pipe Size:	2 in. O.D.	Boring Method:	HSA-3.25	Engineer:	Mark A. Hushebeck
Date Completed:	11/23/2016						

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted: None
At Completion: Dry
After: 24 hrs, Dry Cave @ 24 ft.
Backfilled: 24 hrs.

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

THELEN ASSOCIATES
A DIVISION OF GEOTECHNOLOGY INC
1398 Cox Avenue, Erlanger, KY 41018-1002 / 859-746-9400 / Fax 859-746-9408
thelenassoc.com
geotechnology.com

LOG OF TEST BORING

CLIENT: American Campus Communities
BORING #: 5
PROJECT: Geotechnical Exploration, NKU Phase 1 Residence Hall
Highland Heights, Kentucky
PROJECT #: J028765.01

LOCATION OF BORING: As shown on Boring Plan, Drawing 1

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)
816.7	Ground Surface	0.0
815.9	ASPHALT (9 inches)	0.8
815.7	GRANULAR BASE (3 inches)	1.0
804.7	Mixed brown and gray very moist medium stiff to stiff FILL, silty clay with gravel and shale and limestone fragments.	12.0
802.2	Mixed brown, trace gray very moist stiff to very stiff FILL, silty clay and clay, trace of rock fragments and gravel.	14.5
799.7	Mixed brown and gray very moist stiff FILL, silty clay, trace limestone floaters.	17.0
797.2	Mixed dark brown and gray very moist soft to medium stiff FILL, silty clay and topsoil with shale and limestone, some organic matter.	19.5
788.7	Brown and gray very moist to wet stiff to medium stiff SILTY CLAY, trace shale and limestone fragments (possible colluvium).	28.0
786.4	Interbedded gray wet extremely weak unweathered SHALE and gray medium strong to very strong LIMESTONE (bedrock).	30.3
	Bottom of test boring at 30.3 feet.	

Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
I	1	DS	2-8-12	8	44
I	2	DS	3-4-5	9	50
I	3	DS	3-2-3	8	44
I	4	DS	2-3-4	18	100
I	5	DS	3-4-4	18	100
I	6	DS	4-6-8	12	67
I	7	DS	5-3-6	18	100
I	8	DS	4-8-25	18	100
I	9	DS	6-6-7	18	100
I	10	DS	6-34-12	12	67
I	11	DS	75/3"	3	100

Depth Scale (feet): 0, 5, 10, 15, 20, 25, 30, 35

Datum: NAVD 88
Hammer Weight: 140 lb.
Hole Diameter: 8 in.
Drill Rig: CME 550 BD-1
Surface Elevation: 816.7 ft.
Hammer Drop: 30 in.
Rock Core Diameter: --
Foreman: L. Wanstrath
Date Started: 11/23/2016
Pipe Size: 2 in. O.D.
Boring Method: HSA-3.25
Engineer: Mark A. Hushebeck
Date Completed: 11/23/2016

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted: 30 ft.
At Completion: Dry
After: 24 hrs, 16.0 ft.
Backfilled: 24 hrs.

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

THELEN ASSOCIATES
A DIVISION OF GEOTECHNOLOGY INC
1398 Cox Avenue, Erlanger, KY 41018-1002 / 859-746-9400 / Fax 859-746-9408
thelenassoc.com
geotechnology.com

LOG OF TEST BORING

CLIENT: American Campus Communities
PROJECT: Geotechnical Exploration, NKU Phase 1 Residence Hall
Highland Heights, Kentucky
LOCATION OF BORING: As shown on Boring Plan, Drawing 1

BORING #: 6
PROJECT #: J028765.01
PAGE #: 1 of 1

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Depth Scale (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
815.4	Ground Surface	0.0	0						
815.1	TOPSOIL (4 inches)	0.3							
813.4	Mixed brown and olive brown very moist medium stiff FILL, silty clay with shale and limestone fragments.	2.0		I	1A 1B	DS	4-11-20	12	67
812.6	Interbedded gray slightly moist extremely weak unweathered SHALE and gray medium strong to very strong LIMESTONE (bedrock).	2.8		I	2	DS	100/3"	3	100
	Bottom of test boring at 2.8 feet.								
			5						
			10						
			15						
			20						
			25						
			30						

Datum: NAVD 88
Surface Elevation: 815.4 ft.
Date Started: 11/25/2016
Date Completed: 11/25/2016
Hammer Weight: 140 lb.
Hammer Drop: 30 in.
Pipe Size: 2 in. O.D.
Hole Diameter: 8 in.
Rock Core Diameter: --
Boring Method: HSA-3.25
Drill Rig: CME 550 BD-1
Foreman: L. Wanstrath
Engineer: Mark A. Hushebeck

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH

First Noted	None
At Completion	Dry
After	24 hrs, Dry
Backfilled	24 hrs.

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

THELEN ASSOCIATES

A DIVISION OF GEOTECHNOLOGY INC

1398 Cox Avenue, Erlanger, KY 41018-1002 / 859-746-9400 / Fax 859-746-9408

thelenassoc.com

geotechnology.com

LOG OF TEST BORING

CLIENT: American Campus Communities

PROJECT: Geotechnical Exploration, NKU Phase 1 Residence Hall, Highland Heights, Kentucky

LOCATION OF BORING: As shown on Boring Plan, Drawing 1

BORING #: 7

PROJECT #: J028765.01

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
819.8	Ground Surface	0.0						
819.3	ASPHALT (6 inches)	0.5						
817.8	Mixed brown and gray very moist stiff FILL, silty clay and shale and fragments.	2.0	I	1	DS	5-7-7	12	67
815.3	Olive brown moist stiff to very stiff SILTY CLAY, limestone fragments (colluvium).	4.5	I	2	DS	5-11-20	18	100
812.8	Interbedded brown to olive brown moist extremely weak weathered SHALE and gray medium strong to very strong LIMESTONE (bedrock).	7.0	I	3	DS	75/6"	6	100
811.8	Interbedded gray slightly moist extremely weak unweathered SHALE and gray medium strong to very strong LIMESTONE (bedrock).	8.0	I	4	DS	75/6"	6	100
	Bottom of test boring at 8.0 feet.							

Depth Scale (feet): 0, 5, 10, 15, 20, 25, 30

Datum:	NAVD 88	Hammer Weight:	140 lb.	Hole Diameter:	8 in.	Drill Rig:	CME 550 BD-1
Surface Elevation:	819.8 ft.	Hammer Drop:	30 in.	Rock Core Diameter:	--	Foreman:	L. Wanstrath
Date Started:	11/25/2016	Pipe Size:	2 in. O.D.	Boring Method:	HSA-3.25	Engineer:	Mark A. Hushebeck
Date Completed:	11/25/2016						

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH

First Noted	None
At Completion	Dry
After	24 hrs, Dry
Backfilled	24 hrs.

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

THELEN ASSOCIATES
A DIVISION OF GEOTECHNOLOGY INC
1398 Cox Avenue, Erlanger, KY 41018-1002 / 859-746-9400 / Fax 859-746-9408
thelenassoc.com
geotechnology.com

LOG OF TEST BORING

CLIENT: American Campus Communities
PROJECT: Geotechnical Exploration, NKU Phase 1 Residence Hall
Highland Heights, Kentucky
LOCATION OF BORING: As shown on Boring Plan, Drawing 1

BORING #: 8
PROJECT #: J028765.01
PAGE #: 1 of 1

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Depth Scale (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
825.2	Ground Surface	0.0	0						
824.2	Mixed brown moist very stiff FILL, silty clay, limestone fragments, trace of roots.	1.0		I	1A 1B	DS	6-7-18	18	100
823.2	Mixed brown moist stiff FILL, silty clay with shale and limestone fragments.	2.0							
820.7	Interbedded brown moist extremely weak highly weathered SHALE and gray medium strong to very strong LIMESTONE (bedrock).	4.5		I	2	DS	58-50/3"	3	33
819.7	Interbedded gray slightly moist extremely weak unweathered SHALE and gray medium strong to very strong LIMESTONE (bedrock).	5.5	5	I	3	DS	75/3"	3	100
	Bottom of test boring at 5.5 feet.		10						
			15						
			20						
			25						
			30						

Datum: NAVD 88	Hammer Weight: 140 lb.	Hole Diameter: 8 in.	Drill Rig: CME 550 BD-1
Surface Elevation: 825.2 ft.	Hammer Drop: 30 in.	Rock Core Diameter: --	Foreman: L. Wanstrath
Date Started: 11/25/2016	Pipe Size: 2 in. O.D.	Boring Method: HSA-3.25	Engineer: Mark A. Hushebeck
Date Completed: 11/25/2016			

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted: None
At Completion: Dry
After: 24 hrs, Dry
Backfilled: 24 hrs.

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

THELEN ASSOCIATES
A DIVISION OF GEOTECHNOLOGY INC
1398 Cox Avenue, Erlanger, KY 41018-1002 / 859-746-9400 / Fax 859-746-9408
thelenassoc.com
geotechnology.com

LOG OF TEST BORING

CLIENT: American Campus Communities
BORING #: 9
PROJECT: Geotechnical Exploration, NKU Phase 1 Residence Hall
Highland Heights, Kentucky
PROJECT #: J028765.01
PAGE #: 1 of 1
LOCATION OF BORING: As shown on Boring Plan, Drawing 1

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Depth Scale (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
824.9	Ground Surface	0.0	0						
824.7	TOPSOIL (3 inches)	0.2							
	Mixed brown and gray moist stiff FILL, clay, some shale and limestone fragments.			I	1A 1B	DS	4-4-2	18	100
				I	2	DS	2-3-4	18	100
			5	I	3	DS	3-4-4	18	100
817.9		7.0							
	Brown and gray very moist stiff to very stiff SILTY CLAY with shale and limestone fragments (possible colluvium).			I	4	DS	3-4-7	18	100
815.4		9.5							
	Brown and gray moist very stiff SILTY CLAY with shale and limestone, trace bedding planes (residuum).		10	I	5	DS	5-6-11	18	100
				I	6	DS	16-18-14	18	100
810.4		14.5							
	Interbedded olive brown moist extremely weak weathered SHALE and gray medium strong to very strong LIMESTONE (bedrock).		15	I	7	DS	24-25-25	12	67
807.9		17.0							
	Interbedded gray slightly moist extremely weak unweathered SHALE and gray medium strong to very strong LIMESTONE (bedrock).			I	8	DS	75/6"	6	100
806.9		18.0							
	Bottom of test boring at 18.0 feet.		20						
			25						
			30						

Datum: NAVD 88
Surface Elevation: 824.9 ft.
Date Started: 11/25/2016
Date Completed: 11/25/2016
Hammer Weight: 140 lb.
Hammer Drop: 30 in.
Pipe Size: 2 in. O.D.
Hole Diameter: 8 in.
Rock Core Diameter: --
Boring Method: HSA-3.25
Drill Rig: CME 550 BD-1
Foreman: L. Wanstrath
Engineer: Mark A. Hushebeck

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted: None
At Completion: Dry
After: 24 hrs, 9.0 ft.
Backfilled: 24 hrs.

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

THELEN ASSOCIATES
A DIVISION OF GEOTECHNOLOGY INC
1398 Cox Avenue, Erlanger, KY 41018-1002 / 859-746-9400 / Fax 859-746-9408
thelenassoc.com
geotechnology.com

LOG OF TEST BORING

CLIENT: American Campus Communities
PROJECT: Geotechnical Exploration, NKU Phase 1 Residence Hall
Highland Heights, Kentucky
LOCATION OF BORING: As shown on Boring Plan, Drawing 1

BORING #: 10
PROJECT #: J028765.01

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
822.4	Ground Surface	0.0						
821.9	ASPHALT (6 inches)	0.5						
816.4	Mixed brown, trace gray very moist medium stiff to stiff FILL, silty clay, some shale and limestone fragments, oxide stains (CL).	6.0	I	1	DS	2-3-3	18	100
			U	2	PT		18	75
			I	3	DS	6-8-11	12	67
812.9	Mixed brown, trace gray very moist stiff FILL, clay, some shale and limestone fragments.	9.5	I	4	DS	3-5-5	18	100
805.4	Mixed brown and gray moist stiff to very stiff FILL, silty clay, some limestone and shale fragments.	17.0	I	5	DS	4-5-6	18	100
			I	6	DS	5-7-10	18	100
			I	7	DS	5-7-7	18	100
802.4	Mixed gray moist medium stiff to stiff FILL, silty clay and shale fragments, with limestone floaters, some nested zones.	20.0	I	8	DS	5-6-9	18	100
800.4	Mixed gray moist soft to medium stiff FILL, silty clay and shale fragments, with limestone floaters, some nested zones.	22.0	U	9	PT		21	88
793.9	Dark gray moist to wet soft SILTY CLAY with silt seams (sediment).	28.5	I	10	DS	2-2-3	18	100
			I	11	DS	2-2-2	18	100
789.4	Brown and gray moist extremely weak highly weathered SHALE and gray medium strong to very strong LIMESTONE (bedrock).	33.0	I	12	DS	8-75/3"	7	78
	Auger refusal and bottom of test boring at 33.0 feet.							

Datum: NAVD 88
Surface Elevation: 822.4 ft.
Date Started: 11/25/2016
Date Completed: 11/25/2016
Hammer Weight: 140 lb.
Hammer Drop: 30 in.
Pipe Size: 2 in. O.D.
Hole Diameter: 8 in.
Rock Core Diameter: --
Boring Method: HSA-3.25
Drill Rig: CME 550 BD-1
Foreman: L. Wanstrath
Engineer: Mark A. Hushebeck

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted: None
At Completion: Dry
After: 24 hrs, 13.0 ft.
Backfilled: 24 hrs.

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

THELEN ASSOCIATES
A DIVISION OF GEOTECHNOLOGY INC
1398 Cox Avenue, Erlanger, KY 41018-1002 / 859-746-9400 / Fax 859-746-9408
thelenassoc.com
geotechnology.com

LOG OF TEST BORING

CLIENT: American Campus Communities
PROJECT: Geotechnical Exploration, NKU Phase 1 Residence Hall
Highland Heights, Kentucky
LOCATION OF BORING: As shown on Boring Plan, Drawing 1

BORING #: 11
PROJECT #: J028765.01
PAGE #: 1 of 1

ELEV.	COLOR, MOISTURE, DENSITY, PLASTICITY, SIZE, PROPORTIONS DESCRIPTION	Strata Depth (feet)	Depth Scale (feet)	Sample Condition	Sample Number	Sample Type	SPT* Blows/6" / Rock Core RQD (%)	Recovery (in.)	Recovery (%)
825.7	Ground Surface	0.0	0						
825.5	TOPSOIL (3 inches)	0.2		I	1A 1B	DS	4-4-5	18	100
	Mixed brown moist stiff to very stiff FILL, silty clay, limestone fragments, trace gravel (CL).			I	2	DS	3-3-3	18	100
821.2		4.5	5						
	Mixed dark brown wet medium stiff FILL, topsoil and limestone fragments.			I	3	DS	17-4-3	3	17
818.7		7.0							
	Mixed brown and gray moist very stiff FILL, silty clay, trace limestone fragments.			I	4	DS	2-5-5	18	100
816.2		9.5	10						
813.7	Mixed brown and gray very moist soft to medium stiff FILL, silty clay and clay, some shale and limestone fragments.	12.0		I	5	DS	4-5-6	18	100
811.2	Mixed brown and gray moist stiff FILL, silty clay and clay, trace shale and limestone fragments, trace gravel.	14.5	15	I	6	DS	6-5-3	18	100
808.7	Mixed brown, trace gray moist stiff FILL, silty clay, oxide stains, little shale and limestone fragments.	17.0		I	7	DS	6-6-7	18	100
806.2	Mixed brown and gray moist stiff FILL, silty clay with shale and limestone fragments (fill).	19.5	20	I	8	DS	3-4-6	6	33
	Brown, trace gray very moist stiff SILTY CLAY, with oxide stains, trace shale fragments (possible colluvium).			I	9	DS	4-5-6	18	100
801.2		24.5	25						
	Interbedded brown very moist extremely weak highly weathered SHALE and gray medium strong to very strong LIMESTONE with clay layers (bedrock).			I	10	DS	12-38-40	18	100
796.2		29.5	30						
795.2	Interbedded gray moist to wet extremely weak unweathered SHALE and gray medium strong to very strong LIMESTONE (bedrock).	30.5		I	11	DS	75/6"	6	100
	Bottom of test boring at 30.5 feet.		35						

Datum: NAVD 88
Surface Elevation: 825.7 ft.
Date Started: 11/25/2016
Date Completed: 11/25/2016
Hammer Weight: 140 lb.
Hammer Drop: 30 in.
Pipe Size: 2 in. O.D.
Hole Diameter: 8 in.
Rock Core Diameter: --
Boring Method: HSA-3.25
Drill Rig: CME 550 BD-1
Foreman: L. Wanstrath
Engineer: Mark A. Hushebeck

BORING METHOD
HSA = Hollow Stem Augers
CFA = Continuous Flight Augers
DC = Driving Casing
MD = Mud Drilling

SAMPLE TYPE
PC = Pavement Core
CA = Continuous Flight Auger
DS = Driven Split Spoon
PT = Pressed Shelby Tube
RC = Rock Core

SAMPLE CONDITIONS
D = Disintegrated
I = Intact
U = Undisturbed
L = Lost

GROUNDWATER DEPTH
First Noted 24.5 ft.
At Completion 26.0 ft.
After 24 hrs, 7.0 ft.
Backfilled 24 hrs.

* SPT = Standard Penetration Test - Driving 2" O.D. Sampler 18" with 140-Pound Hammer Falling 30"; Count Made at 6" Intervals

SOIL CLASSIFICATION SHEET

NON COHESIVE SOILS
(Silt, Sand, Gravel and Combinations)

Density

Very Loose	- 5 blows/ft. or less
Loose	- 6 to 10 blows/ft.
Medium Dense	- 11 to 30 blows/ft.
Dense	- 31 to 50 blows/ft.
Very Dense	- 51 blows/ft. or more

Relative Properties

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

Particle Size Identification

Boulders	- 8 inch diameter or more	
Cobbles	- 3 to 8 inch diameter	
Gravel	- Coarse	- 3/4 to 3 inches
	- Fine	- 3/16 to 3/4 inches
Sand	- Coarse	- 2mm to 5mm (dia. of pencil lead)
	- Medium	- 0.45mm to 2mm (dia. of broom straw)
	- Fine	- 0.075mm to 0.45mm (dia. of human hair)
Silt		- 0.005mm to 0.075mm (Cannot see particles)

COHESIVE SOILS
(Clay, Silt and Combinations)

Consistency	Field Identification	Unconfined Compressive Strength (tons/sq. ft.)
Very Soft	Easily penetrated several inches by fist	Less than 0.25
Soft	Easily penetrated several inches by thumb	0.25 – 0.5
Medium Stiff	Can be penetrated several inches by thumb with moderate effort	0.5 – 1.0
Stiff	Readily indented by thumb but penetrated only with great effort	1.0 – 2.0
Very Stiff	Readily indented by thumbnail	2.0 – 4.0
Hard	Indented with difficulty by thumbnail	Over 4.0

Classification on logs are made by visual inspection.

Standard Penetration Test – Driving a 2.0" O.D., 1 3/8" I.D., sampler a distance of 1.0 foot into undisturbed soil with a 140 pound hammer free falling a distance of 30 inches. It is customary to drive the spoon 6 inches to seat into undisturbed soil, then perform the test. The number of hammer blows for seating the spoon and making the tests are recorded for each 6 inches of penetration on the drill log (Example – 6/8/9). The standard penetration test results can be obtained by adding the last two figures (i.e. 8+9=17 blows/ft.). Refusal is defined as greater than 50 blows for 6 inches or less penetration.

Strata Changes – In the column "Soil Descriptions" on the drill log, the horizontal lines represent strata changes. A solid line (————) represents an actually observed change; a dashed line (— — — —) represents an estimated change.

Groundwater observations were made at the times indicated. Porosity of soil strata, weather conditions, site topography, etc., may cause changes in the water levels indicated on the logs.

GEOTECHNOLOGY INC
FROM THE GROUND UP

ROCK CLASSIFICATION SHEET

ROCK WEATHERING

Descriptions	Field Identification
Unweathered	No visible sign of rock material weathering, perhaps slight discoloration on major discontinuity surfaces.
Weathered	Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discolored by weathering and may be somewhat weaker externally than it its fresh condition.
Highly Weathered	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a discontinuous framework or as corestones.
Residual Soil	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact with bedding planes visible, and the soil has not been significantly transported.

ROCK STRENGTH

Descriptions	Field Identification	Uniaxial Compressive Strength (psi)
Extremely Weak	Indented by thumbnail	40-150
Very Weak	Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife.	150-700
Weak	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer.	700-4,000
Medium Strong	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with a single blow of a geological hammer.	4,000-7,000
Strong	Specimen requires more than one blow of a geological hammer to fracture.	7,000-15,000
Very Strong	Specimen requires many blows with a geological hammer to fracture.	15,000-36,000
Extremely Strong	Specimen can only be chipped with geological hammer.	>36,000

BEDDING

Descriptive Term	Bed Thickness
Massive	> 4 ft.
Thick	2 to 4 ft.
Medium	2 in. to 2 ft.
Thin	< 2 in.

APPENDIX D – LABORATORY TEST DATA

Tabulation of Laboratory Tests, Geotechnology Project No. J028765.01

Unconfined Compressive Strength Test Forms, Geotechnology Project No. J028765.01

THELEN ASSOCIATES
A DIVISION OF GEOTECHNOLOGY INC

TABULATION OF LABORATORY TESTS

Boring No.	Sample No.	Depth (ft.) From	Depth (ft.) To	Moisture Content (%)	Natural Dry Density (pcf)	Atterberg Limits (%) LL	Atterberg Limits (%) PL	Atterberg Limits (%) PI	USCS Classification	Unconfined Compressive Strength (psf)
B-1	1	0.5	2.0	18.5						
B-1	2	2.5	4.0	5.9						
B-1	3	5.0	6.5	5.6						
B-2	2	2.5	4.0	18.0						
B-2	3	5.0	6.5	38.7		73	29	44	CH	
B-3	3	5.0	6.5	17.9						
B-4	2	2.5	4.0	19.9						
B-4	PT-3A	4.5	5.0	27.0	97.9					3,610
B-4	PT-3B	5.0	5.5	22.1	111.8					4,650
B-4	5	7.5	9.0	15.3						
B-4	7	12.5	14.0	23.9						
B-4	PT-9	18.0	18.5	28.3	98.3	61	27	34	CH	3,030
B-4	11	25.0	26.5	17.7						
B-4	12	30.0	31.5	8.8						
B-7	1	0.5	2.0	25.9						
B-7	2	2.5	4.0	14.4						
B-7	3	5.0	6.5	15.1						
B-7	4	7.5	8.0	5.1						
B-8	1	0.0	1.5	13.3						
B-8	2	2.5	4.0	10.5						
B-8	3	5.0	6.5	4.5						
B-10	1	0.5	2.0	29.5						
B-10	2	2.5	4.5	21.5		39	21	18	CL	
B-10	4	7.5	9.0	32.0						
B-10	6	12.5	14.0	21.1						
B-10	8	17.5	19.0	14.3						
B-10	PT-9	21.0	22.0	23.1	98.8					
B-10	10	22.0	23.5	28.6						
B-10	11	25.0	26.5	34.3						
B-10	12	30.0	31.5	18.9						
B-11	2	2.5	4.0	26.1		46	23	23	CL	
B-11	3	5.0	6.5	19.8						
B-11	5	10.0	11.5	24.5						
B-11	7	15.0	16.5	26.6						
B-11	9	20.0	21.5	26.8						
B-11	11	30.0	30.5	13.5						

THELEN ASSOCIATES
A DIVISION OF GEOTECHNOLOGY INC
thelenassoc.com
geotechnology.com
1398 Cox Avenue, Erlanger, Kentucky 41018-1002 / 859-746-9400 / Fax 859-746-9408

UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS
ASTM D2166

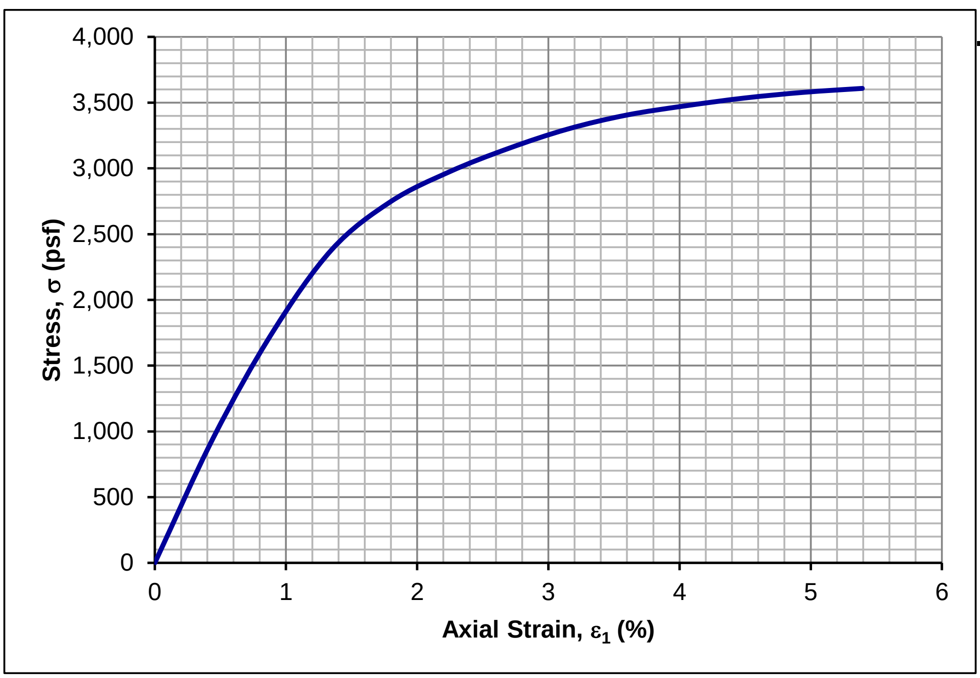
CLIENT : American Campus Communities — DATE: 12/1/2016
PROJECT NO.: J028765.01
PROJECT: NKU Phase 1 Residence Hall
LOCATION: Highland Heights, KY

BORING NO.: B-4 — SAMPLE NO.: PT-3A — DEPTH (ft.): 4.5-5.0
SAMPLE OBTAINED BY: Shelby Tube — CONDITION: Undisturbed
SAMPLE DESCRIPTION: Mixed brown very moist stiff FILL, silty clay, some shale and limestone fragments

LIQUID LIMIT (%): — PLASTIC LIMIT (%): — PLASTICITY INDEX (%): — USCS:
GRAVEL (%): — SAND (%): — SILT (%): — CLAY (%):
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed) — LOAD CELL NO.: 1059

SAMPLE DATA	
DIAMETER (in.):	2.85
HEIGHT (in.):	5.56
HEIGHT TO DIAMETER RATIO:	1.95
WET UNIT WEIGHT (pcf):	124.3
DRY UNIT WEIGHT (pcf):	97.9
VOID RATIO:	0.75
MOISTURE CONTENT (%)*:	27.0
DEGREE OF SATURATION (%):	99

FAILURE DATA	
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.1**
AXIAL STRAIN AT FAILURE (%):	**5.4**
TIME TO FAILURE (min.):	**5.8**
UNCONFINED COMPRESSIVE STRENGTH, q_u (psf):	**3,610**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**1,805**
SENSITIVITY, S_t:	**-**
STRAIN AT 50% OF UCS, ε_{50} (%):	**0.93**



FAILURE SHAPES

FRONT VIEW

SIDE VIEW

REMARKS :

*Moisture content determined after shear from entire sample.

THELEN ASSOCIATES
A DIVISION OF GEOTECHNOLOGY INC
thelenassoc.com
geotechnology.com
1398 Cox Avenue, Erlanger, Kentucky 41018-1002 / 859-746-9400 / Fax 859-746-9408

UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS
ASTM D2166

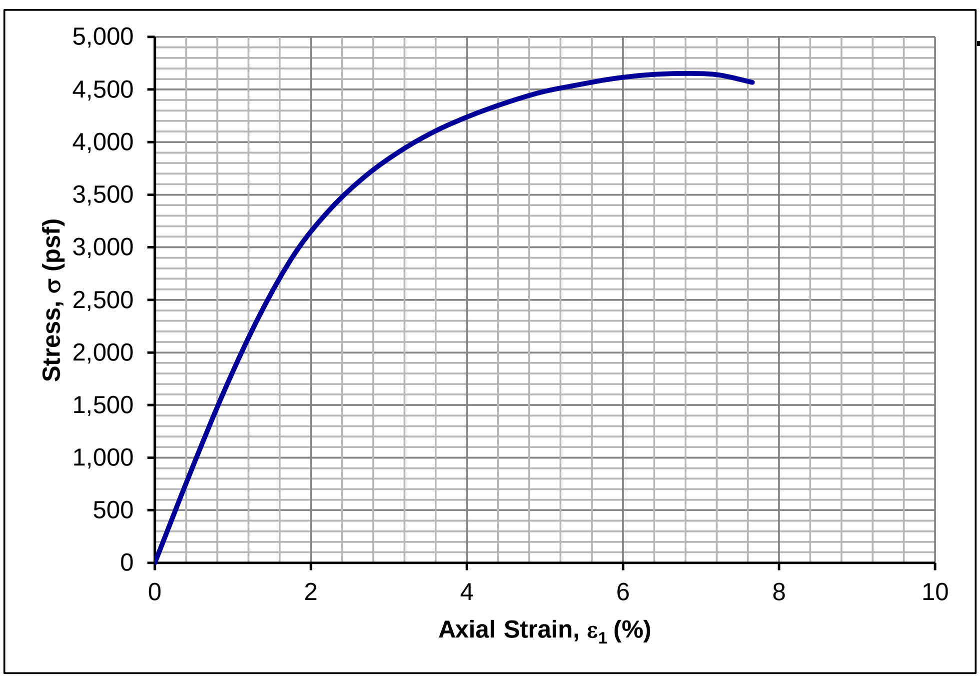
CLIENT : American Campus Communities — DATE: 12/1/2016
PROJECT NO.: J028765.01
PROJECT: NKU Phase 1 Residence Hall
LOCATION: Highland Heights, KY

BORING NO.: B-4 — SAMPLE NO.: PT-3B — DEPTH (ft.): 5.0-5.5
SAMPLE OBTAINED BY: Shelby Tube — CONDITION: Undisturbed
SAMPLE DESCRIPTION: Mixed brown moist very stiff FILL, silty clay, some shale and limestone fragments

LIQUID LIMIT (%): — PLASTIC LIMIT (%): — PLASTICITY INDEX (%): — USCS:
GRAVEL (%): — SAND (%): — SILT (%): — CLAY (%):
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed) — LOAD CELL NO.: 1059

SAMPLE DATA	
DIAMETER (in.):	2.84
HEIGHT (in.):	5.55
HEIGHT TO DIAMETER RATIO:	1.95
WET UNIT WEIGHT (pcf):	136.5
DRY UNIT WEIGHT (pcf):	111.8
VOID RATIO:	0.53
MOISTURE CONTENT (%)*:	22.1
DEGREE OF SATURATION (%):	100

FAILURE DATA	
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.1**
AXIAL STRAIN AT FAILURE (%):	**6.8**
TIME TO FAILURE (min.):	**7.2**
UNCONFINED COMPRESSIVE STRENGTH, q_u (psf):	**4,650**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**2,325**
SENSITIVITY, S_t:	**-**
STRAIN AT 50% OF UCS, ε_{50} (%):	**1.33**



FAILURE SHAPES

FRONT VIEW

SIDE VIEW

REMARKS :

*Moisture content determined after shear from sample cuttings.

THELEN ASSOCIATES
A DIVISION OF GEOTECHNOLOGY INC
thelenassoc.com
geotechnology.com
1398 Cox Avenue, Erlanger, Kentucky 41018-1002 / 859-746-9400 / Fax 859-746-9408

UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOILS
ASTM D2166

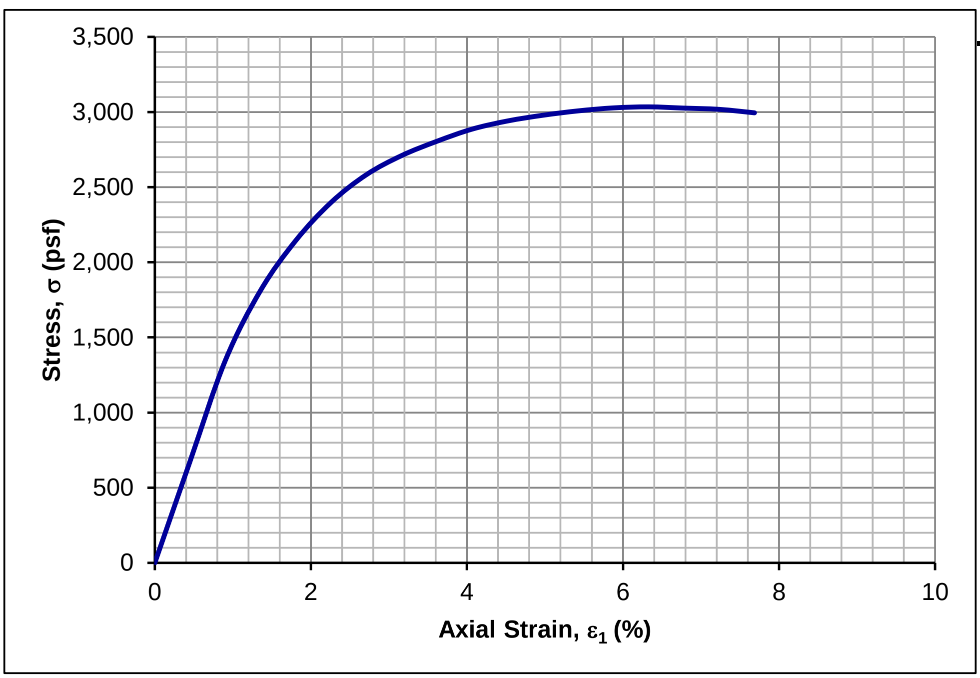
CLIENT : American Campus Communities DATE: 11/30/2016
PROJECT NO.: J028765.01
PROJECT: NKU Phase 1 Residence Hall
LOCATION: Highland Heights, KY

BORING NO.: B-4 SAMPLE NO.: PT-9 DEPTH (ft.): 18.0-18.5
SAMPLE OBTAINED BY: Shelby Tube CONDITION: Undisturbed
SAMPLE DESCRIPTION: Mixed brown and gray very moist stiff FILL, clay, trace limestone fragments

LIQUID LIMIT (%): 61 PLASTIC LIMIT (%): 27 PLASTICITY INDEX (%): 34 USCS: CH
GRAVEL (%): SAND (%): SILT (%): CLAY (%):
SPECIFIC GRAVITY OF SOLIDS: 2.75 (Assumed) LOAD CELL NO.: 1059

SAMPLE DATA	
DIAMETER (in.):	2.84
HEIGHT (in.):	5.53
HEIGHT TO DIAMETER RATIO:	1.95
WET UNIT WEIGHT (pcf):	126.1
DRY UNIT WEIGHT (pcf):	98.3
VOID RATIO:	0.75
MOISTURE CONTENT (%)*:	28.3
DEGREE OF SATURATION (%):	100

FAILURE DATA	
AVERAGE RATE OF AXIAL STRAIN TO FAILURE (%/min.):	**1.1**
AXIAL STRAIN AT FAILURE (%):	**6.3**
TIME TO FAILURE (min.):	**6.8**
UNCONFINED COMPRESSIVE STRENGTH, q_u (psf):	**3,030**
UNDRAINED SHEAR STRENGTH, s_u (psf):	**1,515**
SENSITIVITY, S_t:	**-**
STRAIN AT 50% OF UCS, ε_{50} (%):	**1.07**



FAILURE SHAPES

FRONT VIEW

SIDE VIEW

REMARKS :

*Moisture content determined after shear from entire sample.