REPORT
ON
GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED ADDITION – STEWART HALL
SHIPPENSBURG UNIVERSITY
SHIPPENSBURG TOWNSHIP
CUMBERLAND COUNTY, PENNSYLVANIA

PREPARED
FOR
PENNONTI ASSOCIATES, INC.
MECHANICSBURG, PENNSYLVANIA

F. T. KITLINSKI & ASSOCIATES, INC.
Consulting Geotechnical Engineers
Harrisburg, Pennsylvania

January 2017
January 4, 2017

Penonni Associates, Inc.
1215 Manor Drive
Mechanicsburg, Pennsylvania 17055

Attn: Mr. Craig Raynor, PLA

Re: Geotechnical Engineering Investigation
Proposed Addition – Stewart Hall
Shippensburg University, Shippensburg Township
Cumberland County, Pennsylvania
FTK&A Project No. 16-12-10535

Ladies and Gentlemen:

In accordance with your request, we recently completed a geotechnical engineering investigation and study in connection with the referenced project. The bases for remuneration for these services are outlined in our November 15, 2016 letter proposal issued to your office. Authorization to proceed with this work was received in an email from Mr. Craig Raynor, PLA, of your office, on December 13, 2016.

PURPOSE AND SCOPE

This report presents the results of the subsurface explorations and laboratory soil testing, together with our analyses and conclusions relative to the site preparation, foundation design and foundation construction for the addition to Stewart Hall at Shippensburg University.
SITE LOCATION & DESCRIPTION

Shippensburg University is located in Shippensburg Township, Cumberland County, Pennsylvania which positions the campus of the school along the county's western boundary in the southcentral portion of the state. The geographic location of the University appears on the Project Location Plan, Plate No. 1, in Appendix A of this report. Stewart Hall is situated in the southern portion of the campus and on the north side of Old Main Drive. Old Main (the building) is located immediately to the west of Stewart Hall. The location of Stewart Hall is indicated on the Campus Map also included in Appendix A of this report.

The addition to Stewart Hall will extend from the north elevation of the existing building into an area currently mantled with a bituminous walkway along the building and a lawn area beyond the walkway. In terms of topography, the project site is essentially flat although a slight (less than 2 percent) downward slope exists in the south to north direction. Based on the elevations determined at the locations of the test borings, existing ground surface elevations are approximately two (2) to three (3) feet higher than the ground floor of Stewart Hall which will also be the ground floor elevation of the new addition.

PROPOSED CONSTRUCTION

Details associated with the addition were not available prior to the issue of this report. However, a preliminary plan developed by Ewing Cole Architects of Philadelphia, Pennsylvania indicates the main portion of the addition will have a rectangular-shaped footprint measuring 55 feet in the east-west direction by 30 feet in the transverse direction. The main portion of the addition will be attached to Stewart Hall via an eight (8) feet wide by 18 feet long connecting link. In the vertical dimension, the addition will be three (3) stories in height and its ground floor elevation will match the ground floor elevation (687.3) of Stewart Hall. Structurally, it is anticipated that the addition will have a steel frame including steel floor and roof joists. Maximum column loads are projected to be in the vicinity of 200 kips with bearing walls supporting maximum loads of 10 kips/lineal foot. These values suggest the exterior wall of the addition will, most likely, consist of some form of masonry construction similar to that of Stewart Hall.
INFORMATION MADE AVAILABLE

The following information was made available by Pennoni Associates, Inc. and used in the preparation of this report:

- Drawing No. TB developed by Ewing Cole Architects. This drawing shows the footprints of Stewart Hall and the proposed addition along with the locations of two (2) test borings to be drilled in connection with the subsurface investigation for the addition. Also appearing on this drawing are notes associated with the “Required Services of the Owner’s Geotechnical Engineer for the Preparation of the Geotechnical Investigation Report” and “Suggested Specification for Test Borings”.

- Email information received from Pennoni Associates indicating the ground floor of Stewart Hall is positioned at elevation 687.3 and the first floor of the new addition will be positioned at elevation 697.4.

EXHIBITS ACCOMPANYING THIS REPORT

Appendix A

Plate No. 1 – Project Location Plan & Campus Location Plan
Plate No. 2 – Boring Location Plan

Appendix B

Logs of Standard Drive-Sample/Core Borings
(Boring Nos. B-1 & B-2)

Appendix C

Laboratory Test Results – Covering Grain Size Analyses (sieve and hydrometer), Atterberg Limit Determinations, and Natural Moisture Content Determinations.
SUBSURFACE INVESTIGATIONS

Two (2) standard drive-sample core borings, Nos. B-1 and B-2, were performed at the site on December 19, 2016 by CGC Geoservices, LLC of Carlisle, Pennsylvania. The number and locations of the test borings drilled at the site were jointly determined by Ewing Cole Architects and F. T. Kitlinski & Associates, Inc. Kitlinski Associates established the soil sampling and rock coring criteria employed during the advance of the borings as well as the final depth of each boring. Kitlinski Associates also staked the locations of the boring points in the field and determined the ground surface elevation at each point. The ground surface elevations are based on the ground floor of Stewart Hall having an elevation of 687.3. The locations of the borings are shown on the Boring Location Plan, Plate No. 2 in Appendix A of this report and ground surface elevations appear on the typed test boring logs in Appendix B.

The borings drilled at the project site were scheduled for depths of 30.0 feet each. In the event bedrock was encountered prior to reaching a depth of 30.0 feet, the drilling criteria called for advancing the boring not less than five (5) feet into the rock with the minimum boring depth established as 15.0 feet. As the subsurface exploration program developed, the borings were advanced to depths of 25.0 and 30.0 feet for a total of 55.0 lineal feet of drilling completed at the site.

The borings were advanced through the overburden following the standard penetration test (SPT) procedure to recover split spoon soil samples in accordance with ASTM D 1586. Briefly, the SPT consists of driving a 2.00 inch O.D. by 1.375-inch I.D. split-barreled sampler into the soil using a 140 pound hammer freely falling through a distance of 30 inches. As the sampler is advanced, the number of blows for each of four(4), six(6) inch increments of penetration are recorded with the sum of the blows for the second and third increments forming the SPT “N” value. The “N” value is subsequently employed, in part, to evaluate the relative density, bearing capacity, and settlement potential of the unconsolidated deposits. Spoon samples were taken continuously until bedrock was contacted. Furthermore, the borings were cased using 3.25 inch (I.D.) hollow stem augers. In all cases, the standard penetration testing was performed in undisturbed soil material positioned below the bottom of the augers. When the bedrock surface
was contacted, as marked by absolute split spoon refusal, NQ2 size and type diamond core drilling tools were utilized to continue the boring and recover rock core specimens measuring two (2) inches in diameter. Bedrock was encountered only at the location of Boring No. B-2. After contacting the bedrock, this boring was advanced five (5) feet into the rock.

During the conduct of the test borings, observations were made to detect the presence and level of any ground water. These observations were made as the borings progressed and at the completion of each boring. The ground water data, together with the results of the standard penetration testing and rock coring, are presented on the test boring logs in Appendix B of this report.

In addition to the subsurface explorations, a site reconnaissance was performed on December 14, 2016 by Mr. Blair C. Kitlinski, P.E./geotechnical engineer of this office. The soil samples and rock core specimens recovered from the borings were reviewed by Mr. Kitlinski upon their return to the laboratory of F. T. Kitlinski & Associates, Inc. where they are now stored.

LABORATORY TESTING

General classification tests were performed on a representative soil sample of the overburden to determine its basic engineering properties and classification according to the Unified Soil Classification System (USCS). The tests performed included two (2) grain size analyses (sieve and hydrometer), two (2) Atterberg Limits Determinations, and two (2) natural moisture content determinations. All tests were conducted in accordance with the latest standards set forth by the American Society for Testing and Materials (ASTM). The laboratory test results are presented in Appendix C of this report.
GENERAL GEOLOGY

The project site lies in the Great Valley (also known as the Cumberland Valley west of the Susquehanna River) section of the Ridge and Valley physiographic province and forms the southernmost part of the province in Pennsylvania. The Great Valley arcs across Pennsylvania from its southern border with Maryland in Franklin County to its eastern border with New Jersey in Northampton County. Overall, the Great Valley ranges from eight (8) to ten (10) miles wide.

Ordovician and Cambrian-age rocks, which are formed of sediments that were deposited by an inland sea, form the bedrock beneath the Great Valley. Shale, siltstone and sandstone are the predominate types in the northern third of the Valley whereas carbonate rocks in the form of limestone and dolomite predominate in the southern part of the Valley.

According to published geologic maps, the project site is underlain by the Rockdale Run Formation of Ordovician Age. This formation characteristically consists of very light grey, finely laminated limestone having some pink to brown lenses of chert and a few dolomite beds. The Formation is medium to thickly bedded and moderately resistant to weathering although the interface between the bedrock and the overburden is often irregular with the bedrock surface typically being pinnacled and/or consisting of ridges and troughs. As is the case with most carbonate rocks in Pennsylvania, the Rockdale Run Formation is susceptible to dissolution. Dissolution is a major factor in the development of sinkholes under certain conditions. Therefore, precautionary construction and storm water management measures should be followed as part of the development of project sites underlain by this formation.

ANALYSIS OF SUBSURFACE CONDITIONS

Overburden

The overburden was completely penetrated in only one (1) of the two (2) test borings drilled at the project site. Therefore, the overall thickness of the overburden is not able to be established based on the test boring results. The boring which completely penetrated the overburden, No. B-2, encountered the durable bedrock surface at a depth of 20.0 feet below the existing ground
surface elevation. In this case, the bedrock surface has been defined as the level where hollow stem auger refusal occurred and diamond coring tools were required to continue the boring. At the location of Boring No. B-1, the soil sampling tools were advanced to the scheduled depth (30.0 feet) of the boring where it terminated in unconsolidated deposits. However, there is some indication in Boring No. B-1 that a highly weathered limestone ledge of bedrock may have been encountered between the depths of nine (9) and 14.0 feet. The soil samples retrieved from this zone contained an increased amount of durable limestone fragments which were not present in any of the other samples. Hence, bedrock may be present at a shallower depth than indicated by the final depth of this boring.

Beneath the topsoil which mantles the ground surface, the overburden consists of an upper layer of fill material and at greater depth residual soils produced from the in situ weathering and decomposition of the underlying bedrock. The fill material is three (3) feet thick at the locations of both borings and consists of brown clayey silt mixed with varying amounts of topsoil. In terms of in situ density, the fill is in medium stiff to stiff state of consistency which indicates it may not have been formally compacted, however, most likely has been in place for some time. The nature of the fill suggests it may have originated from the excavation required for the construction of the ground floor of Stewart Hall. Taking into account the thickness of the fill and the vertical position of the addition, the presence of the fill should not impact the new construction.

The residual soil deposits positioned beneath the fill material are 17.0 feet thick at the location of Boring No. B-2 and not less than 27.0 feet thick at the location of Boring No. B-1 which terminated in unconsolidated deposits at its scheduled depth of 30.0 feet. It was previously mentioned, however, that the soil samples retrieved from between the depths of nine (9) and 14 feet in this boring contained increased amounts of weathered limestone gravel. Although it would be somewhat unusual, the presence of this gravel may be associated with a ledge of highly weathered bedrock.

The vast majority of the residuum consists of brown silty clays and clayey silts containing varying amounts of sand and in some cases fine gravel. Laboratory tests conducted on
representative samples of the residuum yielded Unified Soil Classification System (USCS) designations of “CH” and “ML”. In addition to these soil types a percentage of the soil visually classifies as a “CL” soil type. The USCS describes “CH” soil types as highly plastic “fat” clays. “ML” soil types are described as silt, very fine sands and silty or clayey fine sands with slight plasticity. Finally, “CL” soil types are defined by the USCS as inorganic sandy, silty and gravelly clays and silty clays having low to medium plasticity. The highly plastic nature of the “CH” soil type is indicated by the Atterberg Limits Determinations performed on the soil sample. This testing produced a plastic limit of 27 and a liquid limit of 55 which result in a plastic index of 28. The testing performed on the more moderately plastic “ML” soil type resulted in a plastic index of 13 based on plastic and liquid limits of 27 and 40, respectively. Of these three (3) soil types, the larger portion of the residuum would fall into classifications of “CL” and “ML”.

In terms of in situ density, the results of the 23 standard penetration tests completed in the residuum produced “N” values ranging from 4 to 22, averaging 11. On a boring by boring basis, the average “N” value is 10 in Boring No. B-1 and 13 in Boring No. B-2. This range of “N” values indicate the consistency of the residuum varies from soft to very stiff with the median condition being medium stiff to stiff in Boring No. B-1 and stiff to very stiff in Boring No. B-2. On an overall basis, the upper 14 feet of the overburden penetrated in Boring No. B-1 and all of the soil penetrated in Boring No. B-2 are judged to be in a stiff to very stiff state of consistency. These states of consistency are based on “N” values which general range from 10 to 20 and average 15. The lower density material encountered in Boring No. B-1 is positioned between the depths of 14 and 30 feet. “N” values in this zone vary from 4 to 10 and average just over 8. This lower density soil may also indicate the presence of bedrock. It is not unusual for low density material to be positioned immediately above or adjacent to the bedrock surface – particularly when it is oriented at a very steep angle.

Taking into account the in situ density and plasticity levels of the residuum, a bearing capacity of 2,700 p.s.f. is judged to be appropriate for the dimensioning of individual square spread footings and 2,000 p.s.f. for continuous spread footings. These values are based on limiting
differential settlement to less than 0.5 inch and maintaining a safety factor of at least three (3) with respect to base shear failure.

**Bedrock**

As previously mentioned, Boring No. B-1 terminated in unconsolidated deposits at its scheduled depth of 30.0 feet. In the case of Boring No. B-2, the bedrock underlying the site was positively contacted at a depth of 20.0 feet. In Boring No. B-1, however, the nature of the soil suggests that the bedrock surface could be positioned as high as 9.0 to 14.0 feet at the location of this boring. Taking into consideration all of the information, the general profile of the bedrock surface is not able to be determined. The variation in the position of the bedrock surface is the result of differential weathering of the rock. Earlier in this report it was established that the type of bedrock underlying the site often has a very irregular surface — often of a pinnacled nature and/or consisting of series of ridges and troughs. Therefore, it would not be unusual for the bedrock surface to be positioned at levels which are above and/or below the level where it was contacted in Boring No. B-2.

After contacting the bedrock, Boring No. B-2 was advanced 5.0 feet into the rock. The rock cores retrieved from the borings consist of light grey limestone. This confirms geologic mapping of the area which indicate the site is underlain by the Rockdale Run Formation of Ordovician Age. The limited amount of core specimens retrieved from the borings suggests the upper portion of the bedrock is in a fairly durable/solid condition although there are some signs of weathering and solution activity. The recovery and RQD percentages for the single core run completed in the bedrock are 90 and 68, respectively. Should the bedrock be used for support of the addition, the bearing capacity of the rock can conservatively be established as 30 k.s.f. Although a much higher bearing pressure would be applicable for the solid portions of the bedrock, it is appropriate to employ a lower bearing capacity when considering the irregularities believed to be present in the bedrock mass.

Based on the results of the test borings, the bedrock surface is positioned at a depth which would require some form of deep foundation system, such as caissons or micropiles to transfer the
structure loads to the bedrock. The caissons would develop their reaction in end bearing or through a combination of end bearing and side/skin friction when socketed into the rock. Micropiles on the other hand would rely solely on side/skin friction developed in small diameter sockets drilled into the bedrock. For the caissons a value of 50 p.s.i. would be appropriate for determining the contribution of side friction to the overall capacity of the foundation. This value could be increased to 80 p.s.i. for the smaller diameter micropile sockets.

Ground Water

During the test boring operations, ground water readings were taken only at the completion of both of the test borings. Therefore, the readings are considered to be short term. Nevertheless they provide sufficient information to determine if the ground water conditions at the site will impact foundation design and construction.

The ground water readings recorded in the borings suggest the phreatic surface is positioned no higher than elevation 665 at the time (December, 2016) the subsurface investigation program was completed. Anticipating the addition will be supported on conventional spread footings, ground water should not play a role in their design and construction. It would be premature to discuss the impact that ground water may have on deep foundations since the bearing elevations of the foundations are not able to be established based on the results of the test borings.

CONCLUSIONS & RECOMMENDATIONS

Based on the evaluation of all data obtained as a part of this investigation and study, it is concluded that the proposed site is suitable for the intended construction and that a satisfactory foundation system can be developed provided the recommendations presented hereinafter are incorporated into the design, plans and specifications for the project.

The proposed addition will be a moderately loaded structure with fair bearing material available for support. Accordingly, employing a deep foundation system for its support is not considered necessary or economical when taking into account the subsurface conditions and anticipated
structural loads. Therefore, conventional, shallow depth, spread foundation systems consisting of individual column footings and continuous wall footings can be used for its support. The dimensions of the spread foundations will be larger than is usually the case in order to control the effects of settlement. However, the use of spread footings will eliminate a large amount of the uncertainties typically associated with deep foundations installed in a carbonate bedrock environment.

The following specific recommendations are presented for the site preparation, foundation design and foundation construction for the proposed addition to Stewart Hall on the campus of Shippensburg University which is located in Shippensburg Township, Cumberland County, Pennsylvania:

1. The foundation system for the addition should consist of conventional spread column footings and/or continuous wall footings bearing on undisturbed overburden and, if required, freshly-placed structural fill material. Stairwells and elevator shafts should be supported on small mat foundations.

2. A net allowable bearing capacity of 2,000 p.s.f. should be employed for the design of continuous spread foundations and 2,700 p.s.f. for the design of individual square spread foundations. Furthermore, these bearing capacities may be used for dimensioning the foundations bearing on undisturbed overburden as well as freshly placed structural fill material if required.

3. Individual spread column footings should be a minimum of 36 inches square and continuous wall footings should be at least 24 inches wide regardless of how low the resulting bearing pressures may be.

4. Footings which will be exposed to frost penetration should have a minimum cover of 36 inches over their bases.
5. In the event stepped foundations are required, the bearing elevation of the steps should not be positioned above an imaginary line which projects upward at a 45 degree angle from the bearing elevation of the step located at the lowest bearing elevation.

6. The floor slab of the structure should be designed employing a modulus of sub-grade reaction (k) equal to 125 p.c.i. (pounds per cubic inch).

7. All floor slabs should be isolated from the interior footings by a minimum of four (4) inches of PA No. 57 aggregate.

8. With regard to seismic considerations, an *International Building Code* (IBC) Site Class “D” should be employed for design purposes.

9. The design of walls acting as retaining structures (assuming an on-site material will be used as backfill material) should be based on fluid pressure distributions using the following parameters:

   - Angle of Internal Friction = 28 degrees
   - Moist Unit Soil Weight = 130 p.c.f.
   - At-Rest Earth Pressure Coefficient = 0.53
   - Active Earth Pressure Coefficient = 0.36
   - Passive Earth Pressure Coefficient = 2.76

   These values assume the fill material is compacted in accordance with the recommendations outlined hereinafter.

10. Subsequent to reaching foundation bearing elevation, all footing sub-grades should be thoroughly compacted in order to densify the limited depth of material loosened by the excavation process. Hand-operated vibratory compactors should be used for this purpose.

11. In the event the overburden at plan bearing elevation does not meet the design capacity, such soil should be excavated to suitable bearing material and plan bearing elevation re-established with structural fill as outlined hereinafter. The width of the
excavation associated with the removal of the unsuitable soil should increase by six (6) inches [symmetrical about the centerline(s) of the foundation] for each foot of increased depth of over-excavation.

12. After the footprint (plus a five foot wide buffer zone) of the structure has been reduced to floor/pavement sub-grade elevation, the sub-grade should be proof-rolled with at least four (4) complete passes of the compaction equipment outlined in Recommendation No. 15.

13. All soft or weak areas disclosed by pumping, weaving, rutting, cracking, etc. of the sub-grade should be over-excavated to a firm base and backfilled with PA No. 2A aggregate as specified hereinafter.

14. All fill and backfill placed within the footprint of the proposed structure should be compacted to at least 95 percent of the maximum dry density within two (2) percent, plus or minus, of the optimum moisture content as determined by the Standard Compaction Test ASTM 698 The minimum level of compaction may be reduced to 92 percent in non-structural areas.

15. All fill/backfill should be placed in loose lifts not exceeding eight (8) inches in thickness. Where hand held equipment, such as wacker type tampers and “walk behind” rollers are employed for compaction, the loose lift thickness should be reduced to a maximum of six (6) inches.

16. The compaction equipment to be used for soil compaction and proof-rolling sub-grades should consist of a smooth drum vibratory roller having a total static weight of at least 3,000 pounds. The vibratory roller should be operated in a frequency range of 1100 to 1300 vibrations per minute (v.p.m.). At 1300 v.p.m., the dynamic force should be at least 5,000 pounds and the total applied force (static weight plus dynamic force) should be at least 17,000 pounds.
17. Off-site borrow used for the construction of structural fill sections should be free of organic and deleterious content and durable rock fragments measuring over four (4) inches in maximum dimension and should meet the requirements for GC, GW, GP, GM, SP, SM, and SC soil types as specified under ASTM D 2487-90. The on-site overburden will not fall into one of these classifications. It is recommended preference be given to materials which are predominately granular in nature (silty sand and gravel or shale borrow) over materials which are predominately cohesive in nature (siltts and clays) in the selection of a borrow source. It would be prudent to have all borrow materials approved prior to delivery to the site.

18. All foundation work, including but not limited to proof-rolling, footing preparation and fill/backfill placement, should be monitored by a qualified representative of a professional geotechnical engineering firm to ensure the specified bearing capacity is available at the footing sub-grade elevations and the specified degree of compaction is obtained on a layer by layer basis.

In the event design considerations change or unusual conditions are disclosed during construction, F. T. Kitlinski & Associates, Inc. should be notified immediately so additional and/or revised recommendations can be developed as required.

If you have any questions regarding the intent of these recommendations, please feel free to contact our office.

Very truly yours,

F. T. KITLINSKI & ASSOCIATES, INC.

Blair C. Kitlinski, P.E.
APPENDIX A

Plate No. 1 - Project Location Plan

Plate No. 2 - Boring Location Plan
PROPOSED NEW 3-STORY ADDITION
F.F. ELEV. TB-2
MATCH EXIST.

MAXIMUM ASSUMED FOUNDATION LOADS
COLUMNS = 200 KIPS
BEARING WALL = 10 KLF

EXISTING 3-STORY STRUCTURE
F.F. ELEV. (-3'-6")
BELOW GRADE

BORING LOCATION PLAN

F. T. KITLINSKI & ASSOCIATES, INC.
CONSULTING GEO-TECHNICAL ENGINEERS
3636 North Progress Avenue
Harrisburg, Pennsylvania 17110
FSC NO. 3632-HS5

PROPOSED ADDITION – STEWART HALL
SHIPPENSBURG UNIVERSITY
SHIPPENSBURG TOWNSHIP
CUMBERLAND COUNTY, PENNSYLVANIA

PENNCONI ASSOCIATES, INC.
MECHANICSBURG, PENNSYLVANIA

SCALE: None
DATE: 1/17
PLATE NO. 1
APPENDIX B

Logs of Drive Sample Borings

Boring Nos. B-1 and B-2
Project Name & Location:
Proposed Addition – Stewart Hall
Shippensburg University, Shippensburg Township
Cumberland County, Pennsylvania

Project No.: 16-12-10535

Client:
Pennoni Associates, Inc.
Mechanicsburg, Pennsylvania

Boring No: B-1

Ground Surface Elevation: 689.5'

Sheet 1 of 1

Date Started: December 19, 2014
Date Completed: December 19, 2014

Total Boring Depth (ft): 30.0'

Driller & Drilling Co.: A. Chronister
Drilling Rig Model & No.: Acker Soil Max
Weather: Cold - Clear - 30°

Casing: HSA 3.25" I.D.
Casing Hammer Weight: Casing Hammer Drop:
Spoon Sampler O.D. X I.D.: 2.00" X 1.375"
Sampling Hammer Weight: 140 pounds
Sampling Hammer Drop: 30°
Core Bit Type & Size: NQ2 - 2"

Drilling Progress & Ground Water Data

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Soil Sampling Data

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<th>Run No.</th>
<th>Depth Interval (Feet)</th>
<th>Recovery (Feet)</th>
<th>RQD (Feet)</th>
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<td>0.0'- 2.0'</td>
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<td>2.0'- 4.0'</td>
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<tr>
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<td>4.0'- 6.0'</td>
<td>6-7-7-8</td>
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<tr>
<td></td>
<td>12</td>
<td>24.0'- 26.0'</td>
<td>4-4-5-5</td>
<td></td>
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<td></td>
<td></td>
<td>26.0'- 28.0'</td>
<td>3-5-4-3</td>
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<td></td>
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</tr>
<tr>
<td></td>
<td>13</td>
<td>28.0'- 30.0'</td>
<td>5-5-5-6</td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>End of boring</td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>@ 30.0'</td>
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</tbody>
</table>

Rock Coring Data

<table>
<thead>
<tr>
<th>Recovery (Feet)</th>
<th>RQD (Feet)</th>
<th>NOTES</th>
</tr>
</thead>
<tbody>
<tr>
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</tbody>
</table>
**Project Name & Location:**
Proposed Addition – Stewart Hall
Shippensburg University, Shippensburg Township
Cumberland County, Pennsylvania

**Boring No:** B-2
**Ground Surface Elevation:** 690.2'

**Project No.:** 16-12-10535

**Client:**
Pennoni Associates, Inc.
Mechanicsburg, Pennsylvania

**Date Started:** December 19, 2014
**Casing:** HSA 3.25' I.D.
**Date Completed:** December 19, 2014
**Casing Hammer Weight:**
**Total Boring Depth (ft):** 25.0'
**Casing Hammer Drop:**
**Driller & Drilling Co.:** A. Chronister
**Spoon Sampler O.D. X I.D.:** 2.00" X 1.375"'
**Drilling Rig Model & No.:** Acker Soil Max
**Sampling Hammer Weight:** 140 pounds
**Weather: Cold – Clear -30°**
**Sampling Hammer Drop:** 30°
**Core Bit Type & Size:** NQ2 - 2"'s

### Drilling Progress & Ground Water Data

<table>
<thead>
<tr>
<th>Date</th>
<th>Depth Reached</th>
<th>Depth to Water</th>
<th>Hour</th>
</tr>
</thead>
<tbody>
<tr>
<td>12/19/16</td>
<td>25.0'</td>
<td>Dry 0</td>
<td></td>
</tr>
</tbody>
</table>

### Soil Sampling Data

<table>
<thead>
<tr>
<th>Material Description &amp; Remarks</th>
<th>N o.</th>
<th>Depth Interval (Feet)</th>
<th>Spoon Blows Per 6 Inches</th>
<th>Run No.</th>
<th>Depth Interval (Feet)</th>
<th>Recovery (Feet)</th>
<th>RQD (Feet)</th>
<th>NOTES</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fill: Brown clayey silt w/some sand &amp; topsoil – moist – stiff to v. stiff</td>
<td>1</td>
<td>0.0' - 2.0'</td>
<td>11-8-8.5</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>2.0' - 4.0'</td>
<td>4-5-6-4</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Brown clayey silt &amp; silty clay w/some sand – moist – med. stiff to stiff</td>
<td>3</td>
<td>4.0' - 6.0'</td>
<td>11-11-12-11</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>6.0' - 8.0'</td>
<td>6-5-6-8</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>8.0' - 10.0'</td>
<td>9-10-9.9</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>6</td>
<td>10.0' - 12.0'</td>
<td>3-4-9-11</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>12.0' - 14.0'</td>
<td>7-11-9-11</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>14.0' - 16.0'</td>
<td>10-11-9-7</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>9</td>
<td>16.0' - 18.0'</td>
<td>9-7-7-8</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>18.0' - 20.0'</td>
<td>3-5-7-9</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Light gray limestone – few thin weathered zones – slightly broken to sound - hard</td>
<td>1</td>
<td>20.0'</td>
<td>25.0' 4.0'</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Rock Coring Data

- Continuous sampling of overburden
- No groundwater encountered
- Auger refusal @ 20.0'
- Started coring @ 20.0'
- Loss of drill water @ 20.0'
- End of boring @ 25.0'
APPENDIX C

Results of Laboratory Soil Tests

Grain Size Analysis

Atterberg Limits Determinations

Natural Moisture Content Determinations
Particle Size Distribution Report

GRAIN SIZE - mm

<table>
<thead>
<tr>
<th>Size</th>
<th>Percent Finer</th>
<th>Spec. Percent</th>
<th>Pass? (X=NO)</th>
</tr>
</thead>
<tbody>
<tr>
<td>.75</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>.50</td>
<td>97.9</td>
<td></td>
<td></td>
</tr>
<tr>
<td>.375</td>
<td>97.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>.25</td>
<td>94.8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>#4</td>
<td>94.3</td>
<td></td>
<td></td>
</tr>
<tr>
<td>#8</td>
<td>93.7</td>
<td></td>
<td></td>
</tr>
<tr>
<td>#10</td>
<td>93.5</td>
<td></td>
<td></td>
</tr>
<tr>
<td>#16</td>
<td>93.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>#30</td>
<td>92.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>#40</td>
<td>91.2</td>
<td></td>
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</tr>
<tr>
<td>#50</td>
<td>90.4</td>
<td></td>
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<tr>
<td>#100</td>
<td>88.8</td>
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</tr>
<tr>
<td>#200</td>
<td>86.9</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Source of Sample: Test Boring #1
Sample Number: 4

Material Description
Brown Silty Clay With A Trace Of Sand And Gravel

Atterberg Limits
PL = 27.0  LL = 55.0  PI = 28.0

Coefficients
D90 = 0.2533  D95 = 0.0559  D60 = 0.0060  C_u = \_
D50 = 0.0029  D30 = \_
D10 = \_
C_c = \_

Classification
USCS = CH  AASHTO = A-7-6(27)

Remarks
Natural Moisture Content = 26.9%

Date: 12/22/2016

Harrisburg, PA

Client: Pennoni Associates, Inc.
Project: Proposed Addition - Stewart Hall, Shippensburg University
Shippensburg Twp., Cumberland Co., PA
Project No: 16-12-10535
Figure 1
## Particle Size Distribution Report

### GRAIN SIZE - mm

<table>
<thead>
<tr>
<th>Size</th>
<th>% +3&quot;</th>
<th>% Gravel</th>
<th>% Sand</th>
<th>% Fines</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Coarse</td>
<td>Fine</td>
<td>Coarse</td>
<td>Medium</td>
</tr>
<tr>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>16.2</td>
</tr>
</tbody>
</table>

### SIEVE SIZE | PERCENT FINER | SPEC.* | PASS? (X=NO)
<table>
<thead>
<tr>
<th></th>
<th></th>
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</tr>
</thead>
<tbody>
<tr>
<td>#10</td>
<td>100.0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>#16</td>
<td>100.0</td>
<td></td>
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<tr>
<td>#30</td>
<td>94.5</td>
<td></td>
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</tr>
<tr>
<td>#40</td>
<td>83.8</td>
<td></td>
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<tr>
<td>#50</td>
<td>77.7</td>
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<tr>
<td>#100</td>
<td>71.7</td>
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</tr>
<tr>
<td>#200</td>
<td>66.6</td>
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</tbody>
</table>

*(no specification provided)*

**Material Description**
Brown Clayey Sandy Silt

**Atterberg Limits**
- PL = 27.0
- LL = 40.0
- Pl = 13.0

**Coefficients**
- D_90 = 0.5169
- D_60 = 0.4427
- D_30 = 0.0046
- D_10 =
- C_u =
- C_c =

**Classification**
- USCS = ML
- AASHTO = A-6(8)

**Remarks**
- Natural Moisture Content = 22.5%

**Source of Sample:** Test Boring #2
**Sample Number:** 5
**Depth:** 8.0' to 10.0'
**Date:** 12/22/16

**Client:** Pennoni Associates, Inc.
**Project:** Proposed Addition - Stewart Hall, Shippensburg University
Shippensburg Twp., Cumberland Co., PA
**Project No:** 16-12-10535
**Figure:** 2