GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED IU HEALTH CENTRAL UTILITY PLANT
IU HEALTH ACADEMIC HEALTH CENTER OF THE FUTURE CAMPUS
WEST 13TH STREET AND NORTH SENATE AVENUE
INDIANAPOLIS, INDIANA

ATLAS PROJECT NO. 170GC01425

SEPTEMBER 15, 2022

PREPARED FOR:

INDIANA UNIVERSITY HEALTH
950 NORTH MERIDIAN STREET, SUITE 1100
INDIANAPOLIS, IN 46204

ATTENTION: MR. BRENT BOHAN
PROJECT DIRECTOR

September 15, 2022

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Mr. Brent Bohan
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Re: **Geotechnical Engineering Investigation**
Proposed IU Health Central Utility Plant
IU Health Academic Health Center of the Future Campus
West 13th Street and North Senate Avenue
Indianapolis, Indiana
Atlas Project No. 170GC01425

Dear Mr. Bohan:

Submitted herewith is the report for the geotechnical engineering investigation performed by Atlas Technical Consultants for the referenced project. This study was authorized in accordance with Atlas Proposal No. 22-00202 dated January 7, 2022, IU Health Work Order #4 dated June 16, 2022 with Capital ID No. 15-AHCA-900.

This report contains the results of the field and laboratory testing program, an engineering interpretation of this data with respect to the available project characteristics and recommendations to aid design and construction of the foundations and other earth-connected phases of this project. We wish to remind you that we will store the samples for 60 days after which time they will be discarded unless you request otherwise.

We appreciate the opportunity to be of service to you on this project. If we can be of any further assistance, or if you have any questions regarding this report, please do not hesitate to contact either of the undersigned.

Sincerely,

THOMAS JAMES STRUEWING
REGISTERED
No.
21494
STATE OF
INDIANA
PROFESSIONAL ENGINEER

David McIlwaine, P.E.
Senior Project Engineer

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Principal Engineer

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1 PURPOSE AND SCOPE

The purpose of this study was to characterize the general subsurface conditions at the project site by drilling a total of eleven soil test borings and to evaluate this data with respect to foundation concept and design for the proposed Indiana University Health (IU Health) Central Utility Plant (CUP) facility. In addition to the eleven soil test borings that were drilled specifically for this project, this study also includes five soil test borings that were drilled immediately north and immediately east of the proposed CUP building location for the IU Health AHC project. Included in this report is an evaluation of the site with respect to potential construction problems and recommendations dealing with quality control during construction.

2 PROJECT CHARACTERISTICS

IU Health is planning the construction of the Central Utility Plant facility (CUP) for the proposed IU Health AHC project at the existing IU Health Methodist Hospital campus on the near north side of Indianapolis, Indiana. The CUP will service the various facilities that are being constructed for the IU Health AHC campus. The general location of the project site is the western half of the property that is south of 13th Street, north of 12th Street, east of Senate Avenue and west of Capitol Avenue. The general location of the proposed CUP facility site is shown on the Vicinity Map (Figure 1 in the Appendix).

2.1 Site Characteristics

The proposed CUP project site is currently occupied mostly by asphalt paved surface parking lots with some lawn or landscaped areas. Based upon topographic mapping of the project site generated by Cripe, the general topography of the project site is characterized as relatively flat with an estimated topographic relief of about 3 ft. The current ground surface within the project site generally ranges from about El 718 to about El 721, with the majority of the project area at about El 719 to El 720.

Based on the urban location of the project site, in conjunction with available aerial photos, it is apparent that the majority of the project site was previously occupied by buildings that have been razed. Furthermore, based upon past experience in the vicinity of the project site, it is assumed that some of the previous buildings had basements as well as other below-grade features such as crawl spaces, wells, cisterns, pits, tanks, vaults, utilities, etc. that have been filled.

2.2 Proposed Project Characteristics

The proposed CUP building will have plan dimensions of approximately 266 ft (north/south) by approximately 140 ft (east/west). It is our understanding that the proposed building will be a two-story structure that will have a slab-on-grade ground floor over the majority of the building area. The proposed finish first floor level will reportedly be at El 718.83, which is at or near the existing ground surface at the site. The general layout of the proposed CUP project is shown on Figure 2 in the Appendix.

The CUP building will include a small basement level at the northwest corner of the building. The basement finish floor level will be at El 698.83 and the bottom of the basement foundations will bear at about El 697.3. The basement will connect to a utility tunnel that will extend northward beneath 13th Street to the IU Health South Support Building. The tunnel will slope downward from south to north with the tunnel floor at approximately El 698.8 at the south end and about El 690.4 at the north end. The bottom of the tunnel foundation will bear at approximately El 697.3 at the south end and El 688.9 at the north end of the tunnel.

An underground water storage tank will be located beneath the western portion of the building. The water storage tank will be a cast-in-place concrete structure with a base mat bearing at approximately El 701.3.

The foundations for the non-basement portion of the building that are in the immediate vicinity of the below-grade components of the building will bear at the same level as the below-grade component foundations.

2.3 Structural Loading Conditions

It is our understanding that the anticipated column loads (unfactored service loads including dead load and live load) will be in the range of about 200 kips/column to 360 kips/column. The maximum wall loads for the proposed structure are expected to be approximately 6 kips/lin.ft. It is our understanding that the slab-on-grade floor loads may be in the range of about 100 lbs/sq.ft to 400 lbs/sq.ft. No unusual loading conditions or settlement restrictions have been specified.

3 GENERAL SUBSURFACE CONDITIONS

The general subsurface conditions for the proposed CUP facility were investigated by drilling 11 test borings (Borings B-401 through B-411). The test borings were drilled to depths of 25.0 ft to 50.0 ft at the approximate locations shown on the Boring Plan (Figure 2 in the Appendix). The test borings were marked in the field by representatives of Atlas and the ground surface elevations at the test boring locations were estimated based upon topographic mapping provided by IU Health (topographic mapping generated by Cripe). In addition to the 11 soil test borings that were drilled specifically for this project, this study also includes five soil test borings that were drilled immediately north and immediately east of the proposed CUP building location for the IU Health AHC project (Boring Nos. B-56, B-57, B-58, B-121 and B-124).

The subsurface conditions disclosed by the field investigation are summarized in Sections 3.2 and 3.3 of this report. Detailed descriptions of the subsurface conditions encountered in each test boring are presented on the “Test Boring Logs” in the Appendix. The letters in parentheses following the soil descriptions are the soil classifications in general accordance with the Unified Soil Classification System (ASTM D 2487). It should be noted that the stratification lines shown on the soil boring logs represent approximate transitions between material types. In-situ stratum changes could occur gradually or at slightly different depths.

3.1 Regional and Site Geology

The City of Indianapolis is located near the western boundary of the Indiana Physiographic unit known as the New Castle Till Plains and Drainageways, which is part of the Central Till Plain Region. This unit is typified by nearly flat to gently rolling terrain that is dissected by generally southwest trending valleys. Naturally occurring surface features in Indianapolis result from the most recent glaciation (i.e., Wisconsinan Age), which is believed to have crossed Indiana approximately 20,000 years ago. While most of the Indianapolis area is covered by a relatively thick layer of glacial till soil, major valleys, such as those associated with White River and Fall Creek, were formed by meltwater flows during glacial recession. Glacial outwash deposits within these meltwater valleys, which generally coincide with the current stream channels but are much wider, are composed predominately of coarse-grained granular soils consisting of sand and gravel, sometimes containing cobbles and boulders and often overlying layers consisting predominantly of cobbles and boulders at greater depths. The project site is located about 1.2-miles east of White River and about 0.7-miles east of Fall Creek.

Most of the near-surface soils consist of man-made fills of various types. The majority of the natural unconsolidated deposits in the immediate vicinity of the site consist of glacial outwash sand and gravel that was deposited by glacial meltwaters. The natural glacial outwash is often covered by a thin layer of cohesive alluvium and occasionally the glacial outwash soils are interrupted by layers of glacial till that vary in thickness and appear to be random in their occurrence. Geologic mapping indicates that the upper bedrock in this area is dolomitic limestone that was deposited on the order of about 400,000 years ago during the Middle Devonian Age. Published geologic mapping indicates that the bedrock surface underlying this portion of Indianapolis varies from about El 600 to about El 630. The current surface topography within the project site is the result of urban development.

The only mapped fault underlying Marion County is the Fortville Fault, which trends approximately northeast to southwest in the eastern part of the county. This is a high angle dip-slip fault of post-Mississippian and pre-Pleistocene age that cuts the upper bedrock surface but does not extend into the overlying glacial till. There have been no recorded earthquakes associated with the Fortville Fault. Any ground shaking in Indianapolis from earthquakes would likely result from fault movement within with the New Madrid seismic zone, which is located in southeastern Missouri, or the Wabash Valley fault system located in southwestern Indiana and southeastern Illinois. No significant earthquake activity is expected from any of the other faults located in Indiana.

3.2 General Subsurface Soil Conditions

It is evident that the project site has experienced several generations of previous urban development based on available aerial imagery. The project site is currently mostly paved asphalt parking lots with some lawn or landscaped areas. Refer to the individual Test Boring Logs in the Appendix for specific pavement section thicknesses or topsoil thicknesses at the test boring locations. Underlying the pavement section or topsoil, the majority of the test borings encountered miscellaneous uncontrolled fill that includes sandy silty clay, silty clay and sand fill containing various amounts of gravel, bricks, cinders and asphalt fragments to depths ranging from approximately 3.5 ft to 11.0 ft below the existing ground surface. An apparent concrete slab was encountered in Boring B-401 at a depth of about 9.5 ft below the existing ground surface. Boring B-404 encountered a zone of bricks to a depth of 4 ft below the existing ground surface and Boring B-403 encountered a zone of gravel between depths of about 5.5 ft and 11.0 ft.

Underlying the fill materials, some of the test borings revealed natural, very soft to medium stiff, silty clay (CL) and sandy silty clay (CL) to depths ranging from about 6.0 ft to 13.5 ft below the existing ground surface. Several borings also included zones of very loose to loose clayey sand (SC) within this upper zone. Underlying the miscellaneous fill materials, the natural cohesive soils or the very loose to loose clayey sand; the test borings generally revealed medium dense to dense glacial outwash sand (SP, SP-SM, SW, SW-SP, SM) that contains varying amounts of silt and gravel to the boring termination depths. The consistencies of the natural cohesive soils and densities of the natural granular soils as described above and on the test boring logs were estimated based on the results of the standard penetration test (ASTM D1586).

Our experience indicates that cobbles and boulders are often present within the White River and Fall Creek glacial outwash soils that underlie this site. Therefore, it is important to understand that cobbles and boulders may be encountered at various depths and locations at this site (multiple test borings appeared to have encountered cobbles/boulders within the glacial outwash soils, but auger refusal did not occur in any test boring due to boulders).

Due to the urban location of the project site and past generations of urban development, large obstructions and various types of debris, rubble and remnants from previous structures are often encountered within miscellaneous uncontrolled fill materials such as those encountered in the upper 3.5 ft to 11.0 ft of the test borings. It should be anticipated that remnants from various types of structures will be encountered at this site, possibly including basement walls, basement floors, footings, pits, wells, cisterns, etc.

3.3 Ground Water Conditions

Ground water level observations were made during the drilling operations by noting the depth of free ground water on the drilling tools during the drilling operations. Free ground water was noted in the test borings at depths ranging from approximately 33 ft to 37 ft below the existing ground surface, which corresponds to ground water levels at about El 684 to El 685 at the times that the test borings were drilled.

Short-term ground water level readings made in relatively clean granular glacial outwash soils are generally considered to be a reasonably reliable indication of the ground water level at the time the test borings are drilled. However, fluctuations in the level of the ground water should be expected due to variations in rainfall, pumping from the aquifer, the flow level in nearby Fall Creek and White River and other factors not evident at the time of this study. Furthermore, ground water levels reported in test borings drilled previously at the IU Health Methodist Hospital campus by Atlas (formerly ATC and ATEC) typically revealed ground water levels at about El 690. The data from the recently performed test borings, in conjunction with previously drilled test borings over a period of many years at the IU Health Methodist Hospital campus, indicates that the normal ground water level in the vicinity of the project area is generally at, or perhaps slightly below, about El 690 much of the time. Based upon the available data, it appears likely that the ground water level during most of the life of the proposed structure will fluctuate within a range of about El 685 to El 692.

It is not possible to accurately predict future ground water levels with complete certainty; however, it is reasonable and prudent to expect that ground water levels above the levels measured during this investigation will occur in the future, possibly within the life-span of the facility. Based upon our experience, as well as data generated from other studies in the White River and Fall Creek outwash terraces in downtown Indianapolis, it appears unlikely that the ground water level will rise above about El 694 during the life of the facility. Although a higher ground water level due to rare or unforeseen events, or a combination of rare events (e.g., an extended period of heavy or above normal rainfall, cessation of pumping from the aquifer in nearby wells, extended periods of flooding of Fall Creek and White River, etc.) cannot be ruled out with complete certainty, it appears that a ground water level higher than El 694 would be unlikely during the life of this structure. It is even more unlikely that such an event would occur rapidly, but rather would occur over an extended period of time, allowing emergency measures to be taken, if necessary.

4 DESIGN RECOMMENDATIONS

The following design recommendations have been developed on the basis of the previously described project characteristics (Section 2) and subsurface conditions (Section 3). If there are any changes in the project criteria, including the proposed structure location, loading conditions, the finish floor elevations, foundation bearing elevations, structure type, etc., a review should be made by this office.

The design recommendations presented herein are contingent upon the assumption that continuous field observations, testing and evaluations of all of the soil related aspects of the project as described in Sections 4 and 5 of this report will be performed by a representative of Atlas during construction to confirm that the earth related elements of the project are compatible and consistent with the conditions upon which the design recommendations are based and that all unsuitable materials are identified and remediated as described herein. The careful and thorough field testing, observations and evaluations of the soil related aspects of the project are a critical and essential component of the design recommendations.

4.1 Seismic Parameters

Based on geologic mapping, the results of the test borings and measured shear wave velocities in similar White River glacial outwash soil deposits in downtown Indianapolis; it is our opinion that the subsurface conditions at this site meet the criteria for Site Class “C” based on Section 1613.3.2 of the 2012 International Building Code (Chapter 20 of ASCE 7-10 “Minimum Design Loads for Buildings and Other Structures”). The recommended seismic design parameters for this project are summarized in the following table (Table No. 1):

Table No. 1 – Recommended Seismic Design Parameters

Seismic Design Parameter	Recommended Class/Value
Seismic Site Class*	C
Site Modified Peak Ground Acceleration, PGA_M	0.09g
Design Spectral Response Acceleration at Short Periods, S_{DS}**	0.13g
Design Spectral Response Acceleration at 1-Second Period, S_{D1}**	0.10g

*Based upon Chapter 20 of ASCE 7-10 "Minimum Design Loads for Buildings and Other Structures"
**Based upon Section 1613 of the 2012 International Building Code

4.2 General Foundation Concepts

Based upon the results of the test borings that were drilled for this project, it is evident that materials that are not suitable for the reliable support of the proposed structure using conventional spread footings underlie the entire project site. The unsuitable materials consist primarily of miscellaneous, uncontrolled fill and remnants from previous structures and facilities (e.g., underground utilities, etc.); and softer natural cohesive soils and looser natural granular soils. These unsuitable materials were typically encountered in the test borings to depths ranging from as little as about 3.5 ft to as much as about 13.5 ft below the existing ground surface, which corresponds to estimated bottom of unsuitable materials mostly ranging from approximately El 713 to El 705. It is important to note that although a significant number of test borings were drilled in order to characterize the subsurface conditions at this site, and in particular to determine the depths of unsuitable materials, it is possible that miscellaneous, uncontrolled fill, rubble and debris, remnants from previous structures and facilities, cohesive soils or looser granular soils may extend deeper at some isolated locations on the project site since this condition has been encountered on other similar urban project sites in and near downtown Indianapolis. The natural, medium dense to dense glacial outwash sand and gravel soils that were typically encountered underlying the unsuitable materials described above (i.e., below the miscellaneous uncontrolled fill materials, remnants of previous construction, cohesive soils and/or looser granular soils) are considered to be suitable for the reliable support of conventional spread footings for the proposed building.

Table No. 2 summarizes the approximate depths and the estimated approximate elevations at which suitable bearing soils were encountered in the test borings drilled for this project. Due to the urban location and the previous generations of urban development of the project site, it is possible that miscellaneous uncontrolled fill materials and remnants from previous structures (e.g., walls, floors, basements, pits, wells, vaults, tanks, footings, utilities, etc.) may extend deeper at some isolated locations since such cases have been encountered in the past on other similar project sites in and near the IU Health Methodist Hospital campus and in downtown Indianapolis.

Table No. 2 – Estimated Depths and Elevations to Suitable Bearing Soils

Boring No.	Estimated Ground Surface Elevation*	Estimated Depth to Suitable Bearing Soils, ft	Estimated Elevation at Top of Suitable Bearing Soils*
B-401	718	11.0	707
B-402	719	8.5	710
B-403	719	11.0	708
B-404	718	11.0	707
B-405	720	8.5	711
B-406	719	6.0	713
B-407	719	13.5	705
B-408	719	6.0	713
B-409	719	6.0	713
B-410	718	3.5	714
B-411	719	3.5	715
B-56	719	13.5	704
B-57	718	11.0	707
B-58	719	8.5	710
B-121	720	8.5	711
B-124	722	7.0	715

* Ground surface elevations estimated from topographic map provided by IU Health.

The natural, medium dense to dense, glacial outwash sand and gravel soils that were typically encountered beneath the unsuitable materials as summarized above, are generally considered suitable for reliable support of conventional spread footings. It is anticipated that suitable bearing soils will be encountered at the nominal spread footing bearing elevation beneath the proposed basement area and the water tank area (i.e., foundations bearing at or below El 701). However, in non-basement areas, it will be necessary to remove all unsuitable materials at the spread footing locations and either have the spread footings extend to bear deeper than otherwise would nominally be required in order to bear on the natural, medium dense to dense, glacial outwash sand and gravel; or to replace the unsuitable materials with either lean concrete (minimum compressive strength of 2,500 lbs/sq.in.) to re-establish the nominal non-basement area spread footing bearing elevation, or to enlarge the undercut excavation, backfill the undercut excavation with compacted INDOT No. 53 crushed limestone (or a suitably and similarly graded sand and gravel material). It is recommended that only well-graded granular material such as INDOT No. 53 crushed limestone be used to backfill the undercut excavations for non-basement footings (if lean concrete is not used for backfilling). The lateral dimensions at the bases of the undercut excavations beneath spread footings that are backfilled with compacted engineered fill materials must be enlarged 1 ft in each direction for each 2 feet of undercut depth below the design base of the footing, as depicted in Figure 5 in the Appendix, if compacted engineered fill material will be used to re-establish the nominal spread footing bearing elevation. The well-graded granular backfill materials should be placed and compacted as described in Section 5.3. Lean concrete (2,500 lbs/sq.in. minimum compressive strength) can also be used as backfill beneath spread footings, in which case the lateral dimensions at the base of any undercut excavation needed to remove unsuitable materials can be made the same lateral dimensions as the spread footing.

It appears that it will likely be more cost effective and expedient to use a proprietary intermediate foundation system or in-place soil modification/ground improvement technique such as aggregate columns or rigid inclusions to modify and improve the existing subsurface materials at the spread footing locations in non-basement areas so that spread footings can be used without the need for complete removal and replacement of the unsuitable materials as described above. This will eliminate the need for deep and variable undercutting and backfilling of unsuitable materials. This approach also results in more predictable foundation costs and scheduling since it eliminates the need for extensive undercutting and replacement of unsuitable soils to variable depths, which is difficult to quantify beforehand. In this case, consideration must be given relative to the use of aggregate columns or rigid inclusions near the existing buildings, tunnels, underground utilities, etc.; along with any other site elements. If aggregate columns or rigid inclusions are to be used, the specialty geotechnical contractor selected to improve the existing subsurface materials in-place must be consulted regarding the installation of such ground improvement elements adjacent to existing facilities such as buildings, pavements, tunnels, utilities, etc. to ensure that the existing features are not adversely affected due to the installation of the aggregate columns or rigid inclusions, including the serviceability of any existing operations, equipment or functions within the existing facilities due to vibrations from the installation process. Consideration must also be given to the sequencing of aggregate columns or rigid inclusions installation with respect to the construction of the basement and water tank such that the installation process does not adversely affect the new basement or water tank walls or foundations, or that excavation for the basement and water tank does not compromise the integrity of previously installed aggregate columns, depending upon the specific sequencing of the construction activities.

4.3 Spread Footings – Basement Level and Water Tank Level

As described in Section 4.2, the portions of the proposed structure that will include foundations bearing at or below the basement level and the water tank level can be supported on conventional spread footings that bear on the natural, medium dense to dense, glacial outwash sand and gravel that was typically encountered in the test borings at or below El 701 (i.e., the shallowest planned foundation bearing elevation in the basement and water tank areas). It may be necessary to remove old fill, remnants of previous construction, natural cohesive soils or looser sand at some spread footing locations and the spread footings must bear on the natural, medium dense to dense glacial outwash sand and gravel; or on suitable backfill materials placed over such soils after first removing any unsuitable materials to expose the suitable bearing soils. It is important to note that although soils considered suitable for support of the proposed structure using spread footings (i.e., the natural medium dense to dense glacial outwash sand and gravel) were typically encountered at or below approximately El 701 in the test borings, unsuitable materials could extend deeper at some isolated locations. Thus, it may be necessary to undercut unsuitable materials below El 701 in some isolated cases where identified by the footing inspections at the time of construction, and to backfill the undercut excavations with well-compacted engineered fill as described in Section 5.3, or backfilled with lean concrete fill (minimum compressive strength of lean concrete of 2,500 lbs/sq.in.).

Spread footings that bear on the natural, medium dense to dense glacial outwash sand and gravel at or below El 701 can be designed for an allowable soil bearing pressure of 6,000 lbs./sq.ft. A modulus of subgrade reaction value of 40 lbs/cu.in. can be used for the structural design of the base mat foundations for the water tank and the tunnel. It is essential that the soils exposed at the bases of all spread footing excavations should be carefully observed, tested and evaluated by a representative of Atlas to identify any unsuitable materials that must be removed, determine when suitable bearing soils are encountered and to confirm that all unsuitable materials have been identified and removed so that each spread footing will bear on suitable, competent soils as described above. Additional general spread footing recommendations are presented in Section 4.5.

4.4 Spread Footings – Non-Basement Areas

Unless special ground improvement measures are taken to improve or modify the existing unsuitable materials in-place as discussed in Sections 4.2 and 4.4.1 (e.g., aggregate columns, rigid inclusions, etc.), it will first be necessary to completely remove the existing unsuitable materials at all of the non-basement area spread footing locations and replace these materials with well-compacted engineered fill or lean concrete. It will be necessary to remove all existing uncontrolled fill materials, any natural cohesive soils and any looser natural granular soils from beneath the non-basement area spread footings to expose the natural, medium dense to dense, glacial outwash sand and gravel soils. The soils exposed at the base of the undercut excavations should then be carefully observed, tested and evaluated to confirm that the natural, medium dense to dense glacial outwash sand and gravel is exposed at each spread footing location and to identify any unsuitable materials where additional undercutting is necessary to expose the suitable bearing soils. Spread footings that bear on the natural, medium dense to dense glacial outwash sand and gravel, or on well-compacted engineered fill, or lean concrete, that is placed over such soils after first removing all unsuitable materials, can be designed for a maximum allowable soil bearing pressure 6,000 lbs/sq.ft.

The undercut excavations beneath the spread footings in non-basement areas can be backfilled with lean concrete, or with well-compacted engineered fill consisting well-graded granular material such as INDOT No. 53 crushed limestone to re-establish the nominal spread footing design bearing elevation. The backfill material used shall be tested, evaluated and approved by the geotechnical consultant to confirm that the material meets the required backfill characteristics. For the case where compacted engineered fill is used beneath the spread footings, the lateral dimensions at the bases of the undercut excavations beneath the spread footings must be enlarged 1 ft in each direction for each 2 feet of undercut depth below the design base of the spread footing as depicted in Figure 5 in the Appendix. The well-graded granular backfill materials should be placed and compacted as described in Section 5.3.

4.4.1 In-Place Ground Improvements for Non-Basement Areas

It appears that it will likely be more cost effective and expedient to use a proprietary intermediate foundation system or in-place soil modification/ground improvement technique such as aggregate columns or rigid inclusions to modify and improve the existing subsurface materials at the spread footing locations in the non-basement areas so that spread footings can be used without the need for complete removal and replacement of the existing unsuitable materials as described previously in Sections 4.2 and 4.4. This will eliminate the need for deep and variable undercutting and backfilling of unsuitable materials. This approach also results in more predictable foundation costs and scheduling since it eliminates the need for extensive undercutting and replacement of unsuitable materials to variable depths, which is difficult to quantify beforehand. In this case, consideration must be given relative to the use of aggregate columns or rigid inclusions near the existing buildings, tunnels, underground utilities, etc.; along with any other site elements. If aggregate columns or rigid inclusions are to be used, the specialty geotechnical contractor selected to improve the existing subsurface materials in-place must be consulted regarding the installation of such ground improvement elements adjacent to existing facilities such as buildings, pavements, tunnels, utilities, etc. to ensure that the existing features are not adversely affected due to the installation of the aggregate columns or rigid inclusions including serviceability of any existing operations, equipment or functions within the existing facilities due to vibrations from the installation process. Consideration must also be given to the sequencing of aggregate columns or rigid inclusions installation with respect to the construction of the basement and water tank so that the installation process does not adversely affect the new basement or tank walls or foundations, or that excavation for the basement does not compromise the integrity of previously installed aggregate columns or rigid inclusions, depending upon the specific sequence of construction activities.

It is recommended that a specialty geotechnical contractor be consulted to confirm the compatibility of the proprietary ground improvement system (i.e., aggregate columns, rigid inclusions, etc.) with the subsurface conditions and the project requirements (e.g., loading conditions, settlement criteria, structure types, existing facilities, etc.). Due to the variability in the type and condition of the existing subsurface materials at this site, which includes miscellaneous uncontrolled fill, zones of weaker cohesive soils, looser granular soils and remnants from previous construction that extend to varying depths below the existing ground surface and at various locations; the ground improvement system selected must be able to suitably improve the existing subsurface materials within the depth zone required for proper bearing and settlement control of spread footings. The specialty geotechnical contractor must be aware of, and design their system taking into account, the variability in the depth to the stronger, more reliable, natural glacial outwash sand and gravel soils over relatively short lateral distances, and thus uncertainty of the condition of the existing subsurface materials at any specific foundation location. The miscellaneous fill, softer natural cohesive soils and looser granular soils extend to depths as described previously in Section 4.2 and as summarized in Table No. 2. Therefore, it is recommended that the specialty geotechnical contractor consider appropriate depths of modification to enhance the reliability of the ground improvement measures.

Aggregate columns is a common proprietary ground improvement technique whereby dense-graded crushed limestone is placed in holes in thin lifts and densified using a specially designed dynamic energy source. The result is a pre-stressing of the existing material around the aggregate “columns”, inclusion of stiff reinforcement elements within the existing matrix materials and a partial transfer of foundation loads to the deeper, more competent stratum. Rigid inclusions is another common type of intermediate proprietary foundation system where cementitious grout is injected into the matrix soils during withdrawal of probes after extending the probes to the prescribed depths. After the “in-place” proprietary ground improvement measures are installed, spread footings can be used without the need for undercutting and replacement of the existing unsuitable materials. If such a system is to be used, consideration must be given by the specialty geotechnical contractor to potential issues regarding ground vibrations during installation of the aggregate columns or rigid inclusions and the potential impact on adjacent structures, operations and functions; as well as potential obstructions that may exist within the existing fill materials. It may be necessary to remove abandoned foundations, utilities, large debris, etc. within the existing fill to prevent obstruction of the aggregate columns. The specialty geotechnical contractor should be consulted regarding the type of equipment and method of ground installation techniques used to determine the magnitude of ground vibrations and potential adverse impacts on the existing facilities and operations within the facilities.

Intermediate foundation systems or ground improvement techniques such as aggregate columns and rigid inclusions are proprietary specialty geotechnical design/build procedures that are designed by a registered engineer retained by or working for the specialty geotechnical foundation contractor and installed by the specialty geotechnical contractor. Therefore, the specialty geotechnical contractor should be contacted regarding specific applicability to this project, development of the specific program to meet the project requirements including bearing capacity and settlement limitations. Spread footings that bear on intermediate foundation systems consisting of modified or improved subsurface materials as described above can usually be designed for an allowable bearing pressure in the range of about 5 to 8 kips/sq.ft while limiting settlement within required project tolerances without the need for undercutting and replacing the existing subsurface materials or the use of deep foundations. The actual design bearing pressure must be determined by the specialty geotechnical contractor based on the specific criteria of the system, the expected loading conditions and required settlement tolerances. Since aggregate columns and rigid inclusions intermediate foundation systems are proprietary specialty geotechnical systems that result in modified foundation soils, the ground improvement plan and final spread footing design criteria shall be developed and prepared by an engineer registered in the State of Indiana working for or retained by the specialty geotechnical contractor who shall be entirely responsible for the design, installation, performance and warranty of the intermediate foundation system. It is recommended that the ground improvement system be designed to achieve a minimum allowable bearing pressure of 6,000 lbs/sq.ft and to limit the maximum total foundation settlement to 1 in., or less, and the maximum differential foundation settlement to ¾ in. or less, or as otherwise prescribed by the structural engineer.

4.4.2 Lightly Loaded, Non-Settlement Sensitive Spread Footings

Lightly loaded project elements that are not settlement sensitive, such as site retaining walls, lightly loaded canopies, signs, screen walls, decorative elements, etc., can be supported on shallow spread footings bearing on the existing soils at nominal depths, provided that the soils at the bases of these spread footing excavations are carefully observed and evaluated and any clearly unsuitable materials that are identified (i.e., fill that contains collapsible objects or degradable materials, concentrations of rubble and debris, old utilities such as sewers, etc. and any soft or loose soils) are first removed and replaced with well-compacted engineered fill. However, it must be must recognized that there is some risk of greater-than-normal settlement in this case since the undocumented fill materials, such as those noted in the upper approximately 3.5 ft to 11.0 ft at this site, are not as reliable as naturally deposited soils and the fill could contain compressible or collapsible materials not detected by the test borings or revealed by the field observations at the time of construction. If this risk is unacceptable, then these project elements should also be supported on spread footings bearing on firm natural soils in a similar manner as described in Sections 4.2, 4.3 and 4.4 for the main building, or other alternative foundation elements may be considered, such as auger-cast piles.

Provided that the risk of greater than normal settlement of the lightly loaded, non-settlement-sensitive project elements as described above is acceptable, spread footings that bear on firm existing soil at nominal depths, or on well-compacted engineered fill that is placed over firm existing soil, can be designed for a net allowable soil bearing pressure of 1,500 lbs/sq.ft for column (square type) and wall (strip type) footings. It should be anticipated that some undercutting of very soft or very loose soils, or concentrations of rubble and debris, will be required at some footing locations even in conjunction with the relatively light soil bearing pressure. The materials exposed at the bases of the spread footings should be carefully observed, tested and evaluated as described in Section 5.4 to determine whether the actual bearing materials are consistent with those upon which the design recommendations are based.

4.5 General Spread Footing Recommendations

All spread footings should be at least 3 ft wide for bearing capacity considerations. All exterior spread footings and spread footings in unheated areas should be located at a depth of at least 3 ft below the final exterior grade for frost protection. Although the Indiana Building Code requires only 2.5 ft of foundation embedment below the exterior grade in Marion County, our experience indicates that the actual frost depths in this region can occur deeper.

Uplift forces on the spread footings can be resisted by the weight of the spread footings and the soil material that is placed over the footings. It is recommended that the soil weight considered to resist uplift loads be limited to that immediately above and within the perimeter of the footings, unless a much higher factor of safety is used. A total soil unit weight of 115 lbs/cu.ft can be used for the backfill material placed above the footings, provided it is compacted as recommended in Section 5.4. It is also recommended that a factor of safety of at least 1.3 be used for calculating uplift resistance from the footings, provided only the weight of the footing and the soil immediately above it are used to resist uplift forces.

Lateral forces on a spread footing can be resisted by the passive lateral earth pressure against the side of the footing and by friction between the foundation soil and the base of the footing. An allowable passive pressure (“equivalent fluid pressure”) of 125 lbs/sq.ft per ft of depth below the ground surface can be used for that portion of the footing that is below a depth of 2.5 ft below the final exterior grade (no portion of the footing above this depth should be used for lateral resistance). An allowable coefficient of friction between the base of the footing and the underlying soil of 0.3 (based on a factor of safety of 1.5) can be used in conjunction with the minimum downward load on the base of the footing.

All footings should be located so that the least lateral clear distance between any two footings will be at least equal to the difference in their bearing elevations. It is extremely important to note that this does not define the slope at which excavations can safely be made, which is much flatter, but rather the geometric arrangement necessary to prohibit overstressing foundation soils due to stress interference between footings. If this distance cannot be maintained, the lower footing should be designed to account for the load imparted by the upper footing. If this condition occurs adjacent to a below-grade wall, the below-grade wall should be designed for the additional lateral surcharge load that will be imparted upon the wall by the upper footing. The actual slope of a temporary excavation will need to be made flatter and bracing, shoring or underpinning of existing footings may be necessary, depending upon the specific geometric arrangement of the footings and loading conditions on the footings, in order to protect the integrity of the existing footings and to prohibit undermining of soil from beneath spread footings.

Care must be exercised when excavating near the existing features and buildings as well as the surrounding streets, utilities, etc. to protect the integrity of the existing foundations and floors, as well as other existing features. Bracing, shoring or underpinning will be required where it is necessary to excavate below the bottom elevation of the existing footings, floor slabs, streets, utilities, etc.

4.6 Slab-on-Grade Floors

It appears that it will be possible to support the basement level slab-on-grade floors on the existing soils beneath the basement floor level. It will be necessary to remove any existing remnants from previous construction, rubble, debris and any softer natural soils from beneath the floor slab areas and to replace these materials with well-compacted granular fill. It is expected that relatively clean sand and gravel soils will be exposed beneath the basement level floor slab. In any basement floor slab subgrade areas where clean sand and gravel is not exposed at the slab subgrade level, it is recommended that the existing soils be removed to a minimum depth of 6 in. below the bottom of the slab and replaced with a 6 in. (minimum) thick layer of relatively clean granular material such as sand and gravel or crushed limestone. Provided that a minimum of 6 in. of granular material exists, or is placed below the slab, a modulus of subgrade reaction value (k_{30}) of 150 lbs/cu.in. can be used for design of the basement floor slabs.

It appears that it may be possible to support lightly loaded and non-settlement-sensitive slab-on-grade floors on the existing soils in the non-basement areas, provided the slab subgrade is prepared and observed as described in Section 5.2 of this report and any clearly unsuitable fill materials (i.e., fill that contains collapsible objects or degradable materials, concentrations of rubble and debris, old utilities such as sewers, cisterns, wells, etc. and soft or loose soils) are removed and replaced with well-compacted engineered fill. The cost of complete removal and replacement of the existing miscellaneous uncontrolled fill materials beneath any lightly loaded and non-settlement-sensitive slab-on-grade floors in non-basement floor slab areas (or the use of other in-place ground improvement measures), may not be justified in order to eliminate the risk of greater-than-normal floor slab settlement that could occur at some locations if the existing fill materials are not completely removed. However, the owner must recognize that there is some risk of greater-than-normal floor slab settlement in this case since uncontrolled fill materials are not as reliable as naturally deposited soils and the fill could contain compressible or collapsible materials not detected by the test borings or revealed by the field observations at the time of construction. It is recommended that the slab-on-grade floors be supported on a 6 in. thick (minimum) layer of relatively clean granular material such as sand and gravel or crushed limestone and a modulus of subgrade reaction (k_{30}) value of 100 lbs/cu.in. can be used for design of the floor slabs. Alternatively, if it is desired to completely eliminate the risk of greater than normal floor slab settlement due to the existing uncontrolled fill materials and to mitigate the potential of unacceptable settlement, ground improvement measures should be implemented beneath the non-basement area floor slabs in a fashion similar to those described in Section 4.4.1 in order to mitigate and limit settlement of the floor slabs.

For any slab-on-grade floors that will be more heavily loaded, or that will support settlement sensitive equipment or devices, it is recommended that special ground improvement measures be taken to mitigate the risk of greater than normal settlement. This could include complete removal and replacement of the unsuitable materials beneath the slab-on-grade floors (refer to Section 4.2 and Table No. 2 regarding depths of unsuitable materials encountered in the test borings), or the implementation of in-place ground improvement measures beneath the slab-on-grade floors. If a special proprietary ground improvement technique is to be used to improve the existing subsurface conditions in-place, it is recommended that a specialty geotechnical contractor be engaged in a similar fashion as described in Section 4.4.1 to develop the appropriate in-place ground improvement program. In addition to the ground improvement elements, it is likely that a load transfer platform consisting of a thick layer of compacted dense-graded crushed limestone constructed beneath the floor slab and over the ground improvement elements will be required by the specialty geotechnical contractor in this case.

If any floor finishes, flooring adhesives or floor coverings are to be installed that are sensitive to moisture, or if there are any functions or uses that could be adversely affected by moisture vapors (such as stored goods in contact with the floor or climate/humidity controlled conditions), a vapor barrier should be included beneath the floor slabs in those areas of the building that will receive the moisture sensitive floor finish, floor covering or otherwise would require a vapor barrier as described above. It is recommended that where vapor barriers are used the vapor barrier should be installed in accordance with ACI Manual of Concrete Practice 302.1R, “Guide to Concrete Floor and Slab Construction”.

4.7 Ground Water

At the time of the subsurface investigation, the ground water level in the test borings generally appeared to be in the range of about El 685, or slightly lower. Data from previous subsurface investigations at the IU Health campus indicates the potential for a normal ground water level at about El 690. Furthermore, it is reasonable to expect that higher ground water levels will occur during the life of the proposed structure.

A review of published literature, as well as previous Atlas (formerly ATC and ATEC) studies in downtown Indianapolis, indicate that the ground water level in the glacial outwash aquifer beneath the project site responds primarily to three variables: pumping from the aquifer, varying climatic conditions (i.e., above average precipitation over an extended period of time) and changes in the level of White River and Fall Creek (i.e., flooding or permanently raising the pool levels with dams). There has been pumping from the outwash sand and gravel aquifer to serve industrial and municipal sources in downtown Indianapolis for many years. While the cessation of pumping from these wells could result in an increase in the ground water level, the trend in the future would be expected to be increasing ground water pumping rather than decreasing pumping. Consequently, it is not expected that a ground water level rise within the life of the proposed structures would likely occur due to cessation of ground water pumping.

Flooding of White River and Fall Creek could have an influence on the ground water level fluctuation at the CUP building site, although due to the distance between these waterways and the project site, and the vast storage volumes available within the outwash sand and gravel aquifer, the effect of flooding does not appear to be as significant at this site as it would be for sites located closer to these waterways and their associated floodplains. Raising the normal pool levels in these waterways by constructing dams, which has been discussed in the past but does not appear to be likely now, could raise the ground water level below this site to some level commensurate with the raising of the normal pool level in the waterways.

It appears that the most likely contributing factor that would produce a significant increase in the ground water level at the project site is a several-year period of above average precipitation. Based on the observed response of the ground water level at several downtown Indianapolis locations correlated with periods of above average precipitation, it appears that a cumulative surplus of precipitation on the order of about 16 to 18 inches of rain over a three or four-year period could result in an increase in the ground water table on the order of about 5 ft.

Since future precipitation patterns and the frequency and magnitude of flooding events cannot be predicted with certainty or accuracy, neither can the highest ground water level that will occur below the project site in the next 75 to 100 years be predicted with certainty or accuracy. However, based on the factors discussed in the previous paragraphs, it is considered unlikely that the ground water level at this site would rise above about El 694 during the life of the structure. Although a higher ground water level under a combination of rare events (e.g., an extended period of above normal precipitation in combination with unusual flooding of White River and Fall Creek, the cessation of pumping from a number of nearby wells and/or raising the pool level of White River and Fall Creek) cannot be ruled out with complete certainty (and the probability of such occurrences appears to be relatively low), it appears unlikely that a ground water level higher than about El 694 would occur during the life of this structure. It is even more unlikely that such an event would occur rapidly but rather the ground water level would rise over an extended period of time, allowing emergency measures to be taken, if necessary. Therefore, it is recommended that a design high ground water level no lower than El 694 should be used for the CUP facility. It is likely that the ground water level during most of the life of the proposed structure will fluctuate within a range of about El 685 to El 692.

The basement level and the water tank level will be at El 698.8 and El 702.8, respectively, which are well above the recommended design high ground water level of El 694.0. However, the north end of the tunnel will be at El 690.2, which is below the recommended design high ground water level of El 694.0. Therefore, some type of special measures will be required to prevent heaving of the tunnel base or floor slab and seepage of ground water into the tunnel. It appears that it will be most efficient for the tunnel to be made watertight and designed to resist buoyancy and hydrostatic pressure rather than installing permanent dewatering measures to maintain the ground water level below the tunnel floor level.

This will require developing the structural capacity for the tunnel to resist uplift pressures, lateral pressures and using waterproofing materials sufficient to resist the hydrostatic pressure. Figure 4 in the Appendix can be used as a guide to evaluate the buoyant forces on the tunnel foundation. A total soil unit weight of 125 lbs/cu.ft can be used to determine the weight of soil above the foundation as shown in Figure 4. It is recommended that a factor of safety of at least 1.3 be used for buoyancy conditions. The tunnel walls will need to be made watertight and designed for hydrostatic pressures up to at least the design high ground water level in this case.

Since the basement will be above the design high ground water level, it is recommended that the basement walls be damp-proofed so that any surface water that infiltrates into the backfill or any perched ground water behind the basement walls cannot penetrate or seep through the walls. Refer to Section 4.8 for other recommendations regarding basement walls.

4.8 Below-Grade Walls

The magnitude of the lateral earth pressure against a below-grade wall (i.e., the basement walls, the water tank walls, the tunnel walls) is dependent on the method of backfill placement, the type of backfill materials used, drainage provisions and whether or not the wall is permitted to yield during and/or after placement of the backfill. When a wall is held rigidly against horizontal movement (such as basement walls, water tank walls and tunnel walls that are braced by the floors, structural framing, other walls, etc.), the lateral earth pressure against the wall is greater than the "active" lateral earth pressure that is typically used in the design of free-standing retaining walls. Therefore, braced walls (such as the basement walls, water tank walls and tunnel walls) must be designed for higher, "at-rest" lateral earth pressures using an at-rest lateral earth pressure coefficient, K_o, while free-standing retaining walls can be designed for active lateral earth pressures using an active lateral earth pressure coefficient, K_a. A design illustration to aid in computing lateral earth pressures against below-grade walls is included as Figure 3 in the Appendix. Figure 3 in the Appendix includes a hydrostatic pressure component for the case of a submerged watertight wall below the design high ground water level.

It is recommended that only well-graded, free-draining granular material be used for backfill behind the below-grade walls within a zone defined by a plane extending upward and outward on a 1 to 1 slope from the outside of the wall footing as shown in Figure 3. Provided that well-graded granular materials are used for backfill behind the basement walls, water tank walls and tunnel walls, it is recommended that a total soil unit weight of 125 lbs/cu.ft and a coefficient of lateral earth pressure at-rest (K_o) of 0.46 be used for the design of the basement walls using Figure 3 in the Appendix. A submerged soil unit weight of 68 lbs/cu.ft should be used below the design ground water level when calculating lateral earth pressures using Figure 3 in the Appendix. It is suggested that a 2 ft thick layer of relatively impervious, or low permeability, cohesive soils, such as silty clay, be included at the top of the backfill above the free-draining granular material to prevent excessive infiltration of surface water. The free-draining granular backfill soil should be covered with a layer of non-woven geotextile to prevent migration of the finer soils into the granular material.

If the on-site cohesive soils are used for backfill within any portion of the backfill zone described above, or if a temporary earth retention system is used that results in existing soils remaining within this zone, a coefficient of lateral earth pressure at-rest (K_o) of 0.55 should be used for calculating the lateral earth pressures using Figure 3 in the Appendix. If on-site cohesive soils are used for backfill within the zone defined above, or if the temporary earth retention system results in existing soils remaining within this zone, adequate drainage measures must be included to prevent accumulation of hydrostatic pressure behind the walls (e.g., pre-fabricated drainage materials or a 2 ft wide zone of INDOT No. 8 coarse aggregate placed against the walls). It is suggested that a 2 ft thick layer of relatively impervious, or low permeability, cohesive soils, such as silty clay, be included at the top of the backfill above the free-draining granular material to prevent excessive infiltration of surface water. The free-draining granular backfill soil should be covered with a layer of non-woven geotextile to prevent migration of the finer soils into the granular material.

It will be necessary to assume an area surcharge load to account for heavy construction equipment and the permanent traffic loads operating on the areas surrounding the buildings. The computational method depicted in Figure 3 in the Appendix includes criteria for the lateral earth pressure developed from normal surface area surcharge loads; however, if it is necessary to operate heavy equipment such as crawler cranes (or other such heavy equipment that impart concentrated loads greater than the general area surcharge load) immediately adjacent to the basement walls, an additional component of lateral earth pressure will result and must be added to the diagram in Figure 3 in the Appendix.

Only well-graded granular materials should be used to backfill the space between the basement walls and the temporary retention system (or between the walls and the open-cut slopes) and the backfill should be compacted to at least 95 percent of the standard Proctor maximum density (ASTM D 698).

4.8.1 Free-Standing Site Retaining Walls

Relatively short, free standing cantilever retaining walls (i.e., those walls that are free to rotate sufficiently to develop an active lateral earth pressure condition) and where the bases of the walls will be at relatively shallow depth below the existing site grade, can be designed using an active lateral earth pressure coefficient (K_a). Provided that well-graded granular material is used for backfill behind these walls, a total soil unit weight of 130 lbs/cu.ft and an active lateral earth pressure coefficient (K_a) of 0.33 (or an "equivalent fluid pressure" of 43 lbs/cu.ft) can be used for the design of free-standing retaining walls. It is recommended that a perforated drain pipe be placed along the base of the free-standing retaining walls to drain any surface water or ground water that might enter the backfill. The pipe should drain to a sump pit from which water can be pumped or drain to a suitable gravity outfall that is protected from clogging.

Lateral loads on the free-standing retaining walls can be resisted by the passive lateral earth pressure on the outside face of the wall foundation and by friction between the base of the foundation and the subgrade soils. Since significant displacement is required to mobilize passive resistance, a factor of safety of 3 has been used to determine the allowable equivalent fluid pressure for the passive condition in order to minimize the potential for excessive displacement. An allowable passive earth pressure of 110 lbs/cu.ft per ft of depth (using a factor of safety of 3 relative to the full passive pressure) below the ground surface can be used on that portion of the foundation located below a depth of 2.5 ft below the exterior grade (no portion of the footing above this depth should be used for lateral resistance). If passive lateral earth pressure is to be used to resist lateral forces, it is essential that the earth that is relied upon to provide the passive pressure resistance cannot be excavated or altered in the future, including the soil above the top of footing level. An allowable coefficient of friction between the base of the retaining wall footings and the foundation soils of 0.20 can be used in conjunction with the minimum downward load on the free-standing retaining wall foundation (based on a factor of safety of 1.5 relative to the ultimate friction).

The footings for the site retaining walls can be designed using the criteria described in Section 4.4.2 for lightly loaded, non-settlement sensitive spread footings in non-basement areas.

4.9 Pavement Design Recommendations

The test borings that were drilled for this project revealed uncontrolled fill materials that extend to depths ranging from about 3 ft to 11 ft below the existing ground surface. Due to the multiple generations of past development at these sites that included various types of structures most of which likely had basements; it is likely that other types of uncontrolled miscellaneous fill materials (e.g., rubble, debris, remnants from previous construction, such as basement floor slabs, foundations, walls, pits, wells, cisterns, utility lines, etc.) exist at various locations on-site and possibly extend to greater depths. Although the uncontrolled fill materials encountered in the test borings are not as reliable as the underlying naturally deposited soils, it does not appear to be practical and probably not economically justified to remove all of the old fill materials from under the proposed pavement areas. It is, however, recommended that any remnants of previous construction that are exposed at the pavement subgrade level (such as foundations, walls, pits, vaults, etc.) be removed to a depth of at least 2 ft below the base or bottom of the proposed pavement section and replaced with well-compacted engineered fill to provide uniform support directly beneath the pavement sections. Furthermore, any collapsible objects, pockets of “nested” debris or rubble, any soft or otherwise unsuitable materials that are identified beneath the pavement subgrade level should also be removed and replaced with well-compacted engineered fill material. Report Section 5.2 contains additional recommendations regarding site preparation and Section 5.3 describes recommended fill compaction requirements.

Final details regarding proposed grading in the new pavement areas is not available at this time; however, depending upon grading requirements and seasonal conditions, it is likely that the pavement subgrade in most pavement areas will be wet, soft or yielding at the time of construction. Furthermore, our experience indicates that most subgrade soils beneath existing pavements will be soft or yielding once the existing pavement section is removed, regardless of the presence of the existing pavements and firm soils in the test borings drilled through the pavement.

If soft or yielding subgrade soils are encountered, it may be possible to stabilize the subgrade soils by discing, aerating and recompacting; however, if it is not possible to improve the subgrade soils in this manner because of weather conditions, scheduling or other conditions (which is most often the case), it is recommended that the subgrade soils be improved or modified using either chemical stabilization (i.e., a suitable lime by-product such as lime-kiln-dust, or cement) if allowed by the owner, mechanical stabilization (i.e., a geogrid with additional crushed limestone placed over the subgrade), or removal of the unsuitable soils and replacement with crushed limestone. It is likely that there will be large areas that are not compatible with chemical stabilization and in such cases mechanical stabilization (e.g., geogrid with additional crushed limestone) or removal and replacement will likely be required. The best method for stabilizing the pavement subgrade should be determined in the field at the time of construction based upon the actual field conditions in conjunction with the specific soil type encountered at the locations requiring stabilization, the size of the areas requiring stabilization and the construction schedule.

The pavement subgrade materials at this site are likely to become unstable under construction traffic, particularly if the construction will be done during seasons when heavy precipitation and cooler temperatures typically occur (such as late fall, winter and spring). The extent to which yielding subgrades may be a problem is difficult to predict beforehand since it is dependent upon several factors including seasonal conditions, precipitation, cut depths, sequencing and schedule of earthwork, surface and subsurface drainage measures, the weight and traffic patterns of construction equipment, etc. Based on our experience, it appears likely that modification or stabilization of pavement subgrade soils will be required in most, if not all, areas at this site. In order to cope with constructability problems and to avoid schedule delays associated with these types of conditions, it would be prudent to develop a contingency plan for pavement subgrade stabilization so that it can be implemented where deemed necessary at the time of construction based on the specific field conditions encountered. It is important that the geotechnical consultant provide continuous inspection during the earthwork operations to identify areas where special stabilization will be required while also limiting the stabilization to only those areas where it is necessary.

The pavement subgrade surface should be uniformly sloped to facilitate drainage through the granular base and to avoid accumulation of water beneath the pavement. The storm water catch basins in pavement areas should be designed to allow water to drain from the aggregate base into the catch basins. At a minimum, subsurface trench drains should be included that extend out at least 25 ft in at least four directions from the catchbasins.

Based our experience with similar soils and engineering judgement, a resilient modulus value of approximately 4,000 lbs/sq.in. has been estimated for use in pavement design for the anticipated pavement subgrade soils encountered at this site. The pavement subgrade soils should be prepared and evaluated as described in Sections 5.2 and 5.3 of this report.

The following report sections outline recommendations for asphalt and concrete pavements for automobile parking areas and truck zones. It is important to note that the recommendations for the automobile parking areas are based on the assumption that these areas will not be subject to any heavy truck traffic. Therefore, in areas where truck traffic cannot be controlled (e.g., driveways, etc.), it is suggested that the thicker pavement section for heavy-duty pavement areas be utilized.

4.9.1 Asphalt Pavement

Based on a resilient modulus value of 4,000 lbs/sq.in., a pavement design period of 20 years, an average of 10 trucks per day in heavy-duty pavement areas and the conditions encountered at the site, the following asphalt pavement sections are recommended.

The table below summarizes recommended minimum asphalt pavement section thicknesses for automobile parking areas and driveways/truck zones based upon the design traffic criteria. The pavement section thicknesses were determined using the “AASHTO Guide for Design of Pavement Structures - 1993”. Based on this method; a reliability value of 85 percent, an overall standard deviation value of 0.45, an initial serviceability value of 4.2, a final serviceability value of 2.0 and pavement structural layer coefficients of 0.42 for asphalt surface, 0.40 for asphalt base and 0.12 for crushed limestone aggregate base material have been assumed for use in the pavement section calculations.

Table No. 3 – Recommended Asphalt Pavement Sections

Pavement Type	Automobile Parking Areas	Driveways and Truck Areas
Design Period	20 years	20 years
Trucks/Day	N/A	10
Subgrade Type	Natural Compacted Soil Subgrade	Natural Compacted Soil Subgrade
Recommended Minimum Asphalt Surface Course Thickness, inches	1.5	1.5
Recommended Minimum Asphalt Base Course Thickness, inches	2	4
Recommended Minimum Aggregate Base Course Thickness, inches	6	10

The aggregate base material should consist of well-compacted crushed limestone that meets the requirements for coarse aggregate size No. 53 in accordance with Indiana Department of Transportation (INDOT) Standard Specifications. Locally available materials referred to as “commercial grade” No. 53 crushed stone should not be used as pavement base material. The hot mix asphalt (HMA) pavement should be constructed in accordance with the 2022 INDOT Standard Specifications Section 400 – Asphalt Pavements. The HMA mix design should be in accordance with INDOT Standard Specifications Section 402-Hot Mix Asphalt, HMA, Pavement.

It should be expected that normal maintenance compatible with asphalt pavement and the design period selected will be required during the life of the pavement. Furthermore, overlaying the pavement surface may be desirable at an intermediate time period to extend the life of the pavement and improve serviceability.

4.9.2 Concrete Pavement

Concrete pavement thicknesses were determined from methods developed by the American Association of State Highway and Transportation Officials (AASHTO). These methods assume that the pavement subgrade is firm, well-compacted and non-pumping and that all joints are properly designed, located and sealed to minimize moisture seepage into the subgrade. It is also important to insure that proper concrete curing practices will be employed and that traffic will not be allowed until the concrete has had sufficient time to cure.

For design calculation purposes, the compressive strength of the concrete was assumed to be a minimum of 4,000 lbs/sq.in. (modulus of rupture of at least 600 lbs/sq.in.). The modulus of subgrade reaction of the soil (k_{30}) was estimated to be 100 lbs/cu.in.

Based on the traffic criteria described above and presented in the following table, recommended concrete pavement sections were determined. The pavement section thickness calculations are based on the AASHTO “Guide for Design of Pavement Structures - 1993”. Based on this method; a reliability value of 85 percent, an overall standard deviation value of 0.35, an initial serviceability value of 4.2 and a final serviceability value of 2.0 have been assumed for use in the pavement section calculations.

Table No. 4 – Recommended Concrete Pavement Sections

Pavement Type	Driveways and Truck Areas
Design Period	20 years
Initial Serviceability	4.2
Terminal Serviceability	2.0
Reliability	85%
Standard Deviation	0.35
Minimum Modulus of Rupture of Concrete, lbs/sq.in.	600
Load Transfer Factor (J)	3.2
Estimated Modulus of Subgrade Reaction, lbs/cu.in.	100
Drainage Coefficient	1.10
Recommended Minimum Concrete Thickness, inches	8
Recommended Minimum Aggregate Base Thickness, inches	6

The performance of the concrete paving section is highly dependent on controlling the pumping of the subgrade soils. It is important that surface drainage be controlled to prevent water from ponding in pavement areas.

4.10 Site Grading and Surface Drainage

Proper surface drainage should be provided at the site to minimize increase in moisture content of the subsurface soils and to limit water that can infiltrate into the backfill around the basement and tunnel walls. The exterior grade should be sloped away from the structure to prevent ponding of water or flow of surface water toward the structure. Any roof drains or down spouts should be channeled or piped away from the structure.

5 GENERAL CONSTRUCTION PROCEDURES AND RECOMMENDATIONS

Since this investigation identified actual subsurface conditions only at the test boring locations, it was necessary for our geotechnical engineers to extrapolate these conditions in order to characterize the entire project site. Even under the best of circumstances, the conditions encountered during construction can be expected to vary somewhat from the test boring results and may, in the extreme case, differ to the extent that modifications to the design recommendations become necessary. Therefore, Atlas should be retained as geotechnical consultant throughout the earth-related phases of this project to correlate actual soil conditions with test boring data, identify variations, conduct additional tests that may be needed and recommend solutions to earth-related problems that may develop.

5.1 Basement, Water Tank and Tunnel Excavation

It will be necessary to make excavations to depths of as much as about 20 ft to 32 ft, or more, below the existing ground surface. A temporary earth retention system may be required to retain the surrounding soil and to protect the adjacent structures, sidewalks, streets and underground utilities. While the design of a temporary earth retention system is beyond the scope of this study and should be performed by an experienced specialty contractor who designs and installs the system, our experience in downtown Indianapolis indicates that such earth retention systems typically consist of soldier piles and wood lagging with soil tie-back anchors. When the proposed earth retention system is designed, consideration should be given to the fact that there may be cobbles and boulders in the glacial outwash materials that could impact the installation of the soldier piles and/or anchors.

It is important to recognize that any earth retention system will permit some movement (both horizontal and vertical) of the earth behind the retention system. The earth retention system described above may permit an undesirable amount of movement if placed immediately adjacent to an existing structure. The amount of movement of the system will depend upon the geometry of the system, stiffness of the members, the locations and capacities of the tie-back anchors, etc., as well as the care and expertise of the installer. A less flexible system, such as a tied-back, steel-reinforced, auger-cast concrete tangent pile wall, may be required in instances where less deflection is required. While this type of wall, which (except for the tie-backs) can be installed prior to any excavation, will not eliminate all movement behind the wall, our experience indicates that such a wall can be designed to limit lateral movement to about ½ inch or less. It is recommended that the construction documents require that the temporary retention system be designed by a registered engineer in the State of Indiana and constructed by a qualified specialty contractor who is well-experienced in this type of work, with only certain performance items specified, such as allowable displacement restrictions (vertical and horizontal deflection), corrosion protection and tie-back testing.

In areas where an open-cut excavation may be possible, and thus an earth retention system is unnecessary, it is recommended that the temporary excavation sideslopes considered for planning purposes be made no steeper than 2 (horizontal) to 1 (vertical), provided that there are no structures located immediately adjacent to the crest of the slope. Unless detailed analyses are made based upon specific excavation geometry, building loads, bearing elevations, etc., the crest of the excavation slope should be at least 30 ft away from any existing buildings based upon excavation slopes of 2 (horizontal) to 1 (vertical), or flatter. The recommendations for temporary excavation slopes assume that the ground surface at the crest of the excavation slope is flat and that no significant, or permanent, surcharge loading is applied. If there is any surcharge loading on the slope or at the crest of the slope, specific analyses will be required based upon the specific loading conditions, overall extent of the loading, loading intensity, etc. The recommendations for the open-cut excavations are only for planning purposes and the actual slope configurations must be determined by the contractor responsible for the temporary excavation, construction means and methods and site safety and should take into account loading from adjacent facilities as well as locations and loading from other site facilities. Some sloughing of loose material should be expected with such slopes and the slopes should be maintained as necessary (including flattening the slope if necessary) and continuously monitored for detection of instabilities that may require remediation. All federal, state and local safety regulations should be followed in regard to open-cut excavations.

It is recommended that a baseline condition and crack survey be made of any nearby structures that could be impacted by the construction before construction is initiated. This should include establishing benchmarks and initial elevations on sidewalks and streets adjacent to the proposed excavation. It is also recommended that a thorough investigation of the existing nearby structures precede any construction to document any existing defects (such as cracks, uneven floors, misaligned windows and doors, etc.) in the existing structures. Periodic monitoring of horizontal and vertical movement of the walls of nearby structures should be incorporated into the retention system program to monitor any movement of the nearby structures.

5.2 Site Preparation

All areas that will support slab-on-grade floors and pavements should be properly prepared. After rough grade has been established, the exposed subgrade should be carefully inspected by the geotechnical engineer, or a qualified geotechnical technician working under the guidance of the geotechnical engineer, by probing and testing as needed. Any organic, frozen, wet, soft or loose soil and other unsuitable materials, such as concentrations of rubble and debris, remnants from previous construction, soft cohesive soils, etc., should be removed. The exposed subgrade should furthermore be inspected by proofrolling with suitable equipment to check for pockets of soft or loose material beneath a thin crust of better soil. Any unsuitable materials thus exposed should be removed and replaced with well-compacted, engineered fill as outlined in Section 5.3; or, if determined to be appropriate by the geotechnical engineer, stabilized using chemical or mechanical stabilization techniques as described in Section 4.9. Based on our experience on other projects near this site with similar subsurface conditions, it appears likely that modification or stabilization of subgrade materials will be required in most pavement areas at this site. It is suggested that the project include contingency plans for stabilization or modification (such as removal of unsuitable materials and replacement with compacted fill, mechanical stabilization, etc.) to be used as determined appropriate by the geotechnical engineer.

Our experience with soils of the type underlying most of this site indicates that the near surface subgrade soils at this site may tend to yield and become unstable under construction traffic, particularly if the construction will be done during a period of heavy precipitation. The extent to which yielding subgrade may be a problem is difficult to predict beforehand since it is dependent upon several factors including seasonal conditions, precipitation, cut depths, sequencing and scheduling of the earthwork, surface and subsurface drainage measures, the weight and traffic patterns of construction equipment, etc. In general, yielding subgrade problems are more prominent in cut areas or where little or no fill is placed.

Care should be exercised during the grading operations at the site. Due to the nature of the near surface soils, the traffic of construction equipment may create pumping and general deterioration of the shallower soils, especially if excess surface water is present. The grading, therefore, should be done during a dry season, if possible.

5.3 Fill Compaction

Any fill that placed beneath spread footings should consist of INDOT No. 53 crushed limestone compacted to a minimum dry density of at least 100 percent of the standard Proctor maximum dry density (ASTM D698), or lean concrete can be used as fill. Flowable fill shall not be used as fill beneath spread footings or mat foundations.

All engineered fill that is placed adjacent to and above spread footings or mat foundations, beneath slab-on-grade floors, and behind retaining walls should be compacted to a dry density of at least 95 percent of the standard Proctor maximum dry density (ASTM D698). All engineered fill beneath pavements should be compacted to a dry density of at least 98 percent of the standard Proctor maximum dry density (ASTM D 698).

The compaction of fill should be accomplished by placing the fill in about 8 in. (or less) loose lifts and mechanically compacting each lift to at least the specified minimum dry density. Field density tests should be performed on each lift as necessary to verify that adequate moisture conditioning and compaction is being achieved.

Only relatively clean, well-graded, granular soils such as the glacial outwash sand and gravel are considered suitable as structural fill material (excluding beneath foundations, where only INDOT No. 53 crushed limestone or lean concrete should be used as fill). Crushed limestone (such as INDOT No. 53 gradation coarse aggregate) is also acceptable as structural fill material. The fill should contain less than 12 percent (by weight) material passing the No. 200 sieve, should be generally well-graded, and no particles larger than 3 in. The material should be free of deleterious materials such as organic matter and debris.

5.4 Foundation Excavations

The soil below the base of each spread footing or mat foundation excavation must be carefully observed, tested and evaluated by the geotechnical engineer-of-record, or a qualified engineering technician working for Atlas under the direction of the geotechnical engineer-of-record, to verify that all uncontrolled fill, remnants from previous construction, cohesive soils, loose natural granular soils or otherwise unsuitable materials, are removed from beneath the spread footings or mat foundations and that the spread footings and mat foundations will bear on the natural, medium dense to dense, glacial outwash sand and gravel as described in Sections 4.2, 4.3 and 4.4 of this report. It is critical that any unsuitable bearing materials must be identified, removed and replaced below spread footing and mat foundation bearing elevations. At the time of such inspection, it will be necessary to make hand auger borings, use a hand penetration device or perform a small hand excavated test pit in the base of the foundation excavation to evaluate the soils below the base of the foundation excavation to verify that the soils below the base are satisfactory for foundation support and compatible with the design assumptions described in Sections 4.2, 4.3 and 4.4 of this report.

Where undercutting is required to remove unsuitable materials, the proposed footing or mat foundation bearing elevation can be re-established by backfilling the undercut excavation with INDOT No. 53 crushed limestone compacted as prescribed in Section 5.3, or lean concrete fill with a minimum compressive strength of 2,500 lbs/sq.in. (flowable fill shall not be used in this case) after all unsuitable materials have been removed. The undercut excavation beneath each spread footing, or where necessary beneath a base mat foundation, should extend to suitable bearing soils and the entire excavation should then be refilled with well-compacted engineered fill as described in Section 5.3 or with lean concrete. Special care should be exercised to remove any sloughed or loose materials near the base of the excavation slopes.

After suitable bearing soils are exposed at the base of a spread footing excavation or mat foundation excavation, the natural granular soil at the bases of the foundation excavations should be compacted with a vibratory compactor. Soils exposed in the bases of all satisfactory foundation excavations should be protected against any detrimental change in condition such as from disturbance, rain, freezing, construction traffic including foot traffic within the excavation, etc. Surface run-off water should be drained away from the excavations and not allowed to pond. If possible, all footing concrete should be placed the same day the excavation is made. If this is not practical, the footing excavations should be adequately protected. It is suggested that concrete “mud mats” be placed at the bases of the spread footing excavations or mat foundation excavations to protect the foundation soils from disturbance and to aid in the proper placement of reinforcing steel.

All existing facilities (e.g., the existing surrounding buildings, utilities, tunnels, pavements, etc.) should be suitably protected from undermining due to excavation for the new structures. Based on the relative depths and locations of the new excavations, bracing, shoring or underpinning will likely be needed to protect the existing facilities. Recommendations regarding temporary excavations are discussed in Section 5.1. All federal, state and local safety regulations should be followed in this regard.

5.5 Construction Dewatering

The normal ground water level is generally below the basement level and the water tank level for the proposed CUP building, and at or near the lowest tunnel level at the north end of the tunnel (reference the discussion regarding ground water levels in Sections 3.3 and 4.7 of this report). It is possible that higher ground water levels could be encountered at the time of construction.

It may be necessary to depress the ground water level in order to construct the northern portion of the tunnel. It is recommended that the ground water level be depressed and maintained at least 3 ft below the deepest excavation level and that no excavations should be made until it is confirmed/demonstrated that the ground water level has been suitably and reliably depressed at least 3 ft below the deepest expected excavation level. The suitable, proper and reliable dewatering and depressing of the ground water level is critical for this project, and insufficient or inadequate dewatering could result in heaving of soils in the bases of excavations that could further result in excessive foundation settlement.

Depending on the seasonal conditions, some seepage of water into excavations at higher elevations should be expected, particularly since “perched” or “trapped” water is often encountered within miscellaneous fill materials above the normal ground water level. It is anticipated that such seepage will either infiltrate downward into the granular subsurface soils or can be handled by conventional dewatering methods such as by pumping from sumps located outside the zone of influence of the footings. This method, however, will not be effective for any excavation that extends below the actual ground water level.

6 FIELD INVESTIGATION

Eleven test borings were drilled for the proposed CUP building project at the approximate locations shown on the Boring Plan (Figure 2 in the Appendix). The test borings were extended to depths of 25.0 ft to 50.0 ft below the existing ground surface. Split-barrel samples were obtained in the test borings by the Standard Penetration Test procedures (ASTM D 1586) at 2.5 ft and 5.0 ft intervals. In addition to the eleven soil test borings that were drilled specifically for this project, this study also includes five soil test borings that were drilled immediately north and east of the proposed CUP building location for the IU Health AHC project.

Logs of the test borings, which show visual descriptions of all soil strata encountered using the Unified Soil Classification System (ASTM D 2488), have been included in the Appendix. Ground water observations, sampling information and other pertinent field data and observations are also included on the test boring logs. In addition, a "Field Classification System for Soil Exploration" document defining the terms and symbols used on the test boring logs and explaining the Standard Penetration Test procedure is provided immediately following the test boring logs.

7 LABORATORY INVESTIGATION

The soil samples were inspected and classified by a geotechnical engineer in general accordance with the Unified Soil Classification System (ASTM D 2488) and the test boring logs were edited as necessary. To aid in classifying the soils and to determine general soil engineering characteristics, the following laboratory tests were performed on selected soil samples:

- Natural moisture content tests (ASTM D 2216)
- Particle size distribution tests (ASTM D 6913)
- Atterberg limits tests (ASTM D 4318)
- Calibrated hand penetrometer (“pocket penetrometer”) tests

The results of these tests are included on the Test Boring Logs and/or summary sheets in the Appendix.

Corrosivity indicator tests are also being performed on selected soil samples from the test borings. The corrosivity indicator battery of tests include water-soluble sulfate content tests, water-soluble chloride content tests, pH tests and laboratory electrical resistivity tests. The laboratory test results for the corrosivity indicator tests will be submitted as an addendum under separate cover.

8 LIMITATIONS OF STUDY

An inherent limitation of any geotechnical engineering study is that conclusions must be drawn on the basis of data collected at a limited number of discrete locations. The recommendations provided in this report were developed from the information obtained from the test borings that depict subsurface conditions only at these specific locations and at the particular time designated on the logs. Soil conditions at other locations may differ from conditions occurring at these boring locations. The nature and extent of variations between the borings may not become evident until the course of construction. If variations then appear evident, it will be necessary to re-evaluate the recommendations of this report after performing on-site observations during the excavation period and noting the characteristics of any variation.

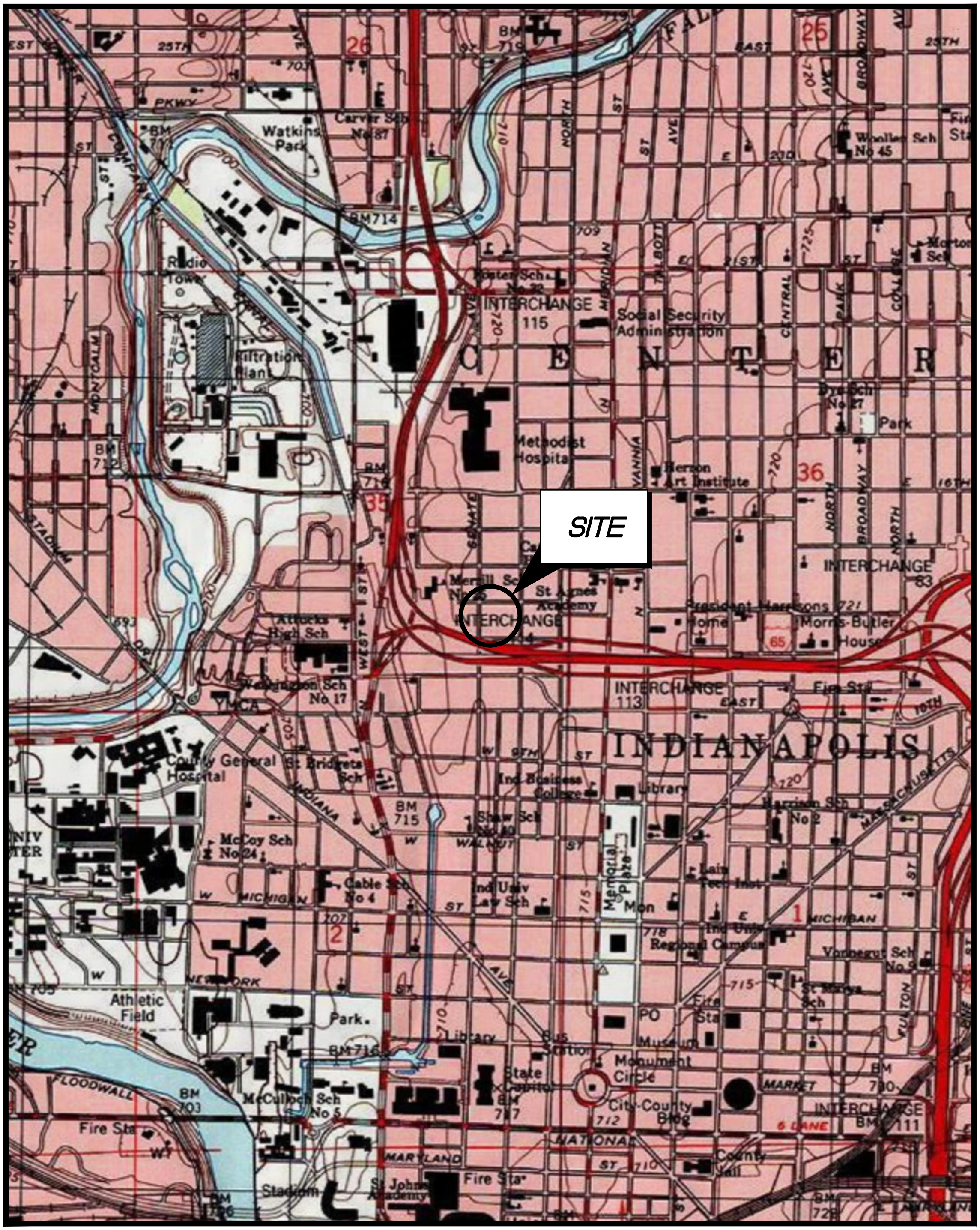
Any comments or recommendations made herein regarding construction related issues (such as temporary excavations) are solely for the purpose and use in the planning the design of the proposed facility. The scope of this investigation is not sufficient to identify all potential construction related issues, variations, anomalies, etc. or all factors that may affect construction means, methods and costs.

Our professional services have been performed, our findings obtained and our recommendations prepared in accordance with customary principles and practices in the field of geotechnical engineering at the time when the services were performed and at the location where the services were performed. This warranty is in lieu of all other warranties either express or implied. This company is not responsible for the independent conclusions, opinions or recommendations made by others based on the field exploration and laboratory test data presented in this report.

The scope of our services does not include any environmental assessment or investigation for the presence or absence of hazardous or toxic materials in the soil, ground water or surface water within or beyond the site studied.

Atlas assumes no responsibility for any construction procedures, temporary excavations (including utility trenches), temporary dewatering or site safety during or after construction. Any recommendations provided regarding temporary conditions during construction are solely for use in planning the design of the project. The contractor shall be solely responsible for all construction procedures, construction means and methods, construction sequencing and for safety measures during construction as well as the protection of all existing facilities. All applicable federal, state and local laws and regulations regarding construction safety must be followed, including current Occupational Safety and Health Administration (OSHA) Regulations including OSHA 29 CFR Part 1926 “Safety and Health Regulations for Construction”, Subpart P “Excavations”, and/or successor regulations. The Contractor shall be solely responsible for designing and constructing stable, temporary excavations and should brace, shore, slope, or bench the sides of the excavations as necessary to maintain stability of the excavation sides and bottom and to protect the integrity of all existing facilities (i.e., existing foundations, floor slabs, equipment, utilities, etc.).

Appendix



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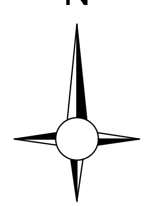

VICINITY MAP

PROPOSED IU HEALTH CENTRAL UTILITY PLANT
IU HEALTH ACADEMIC HEALTH CENTER OF THE FUTURE
13TH STREET AND NORTH SENATE AVENUE
INDIANAPOLIS, INDIANA

Project Number: 170GC01425		Drn. By: RH
Date: 09/06/2022	Scale: 1":2,000'	Ckd. By: DM
ATLAS		1

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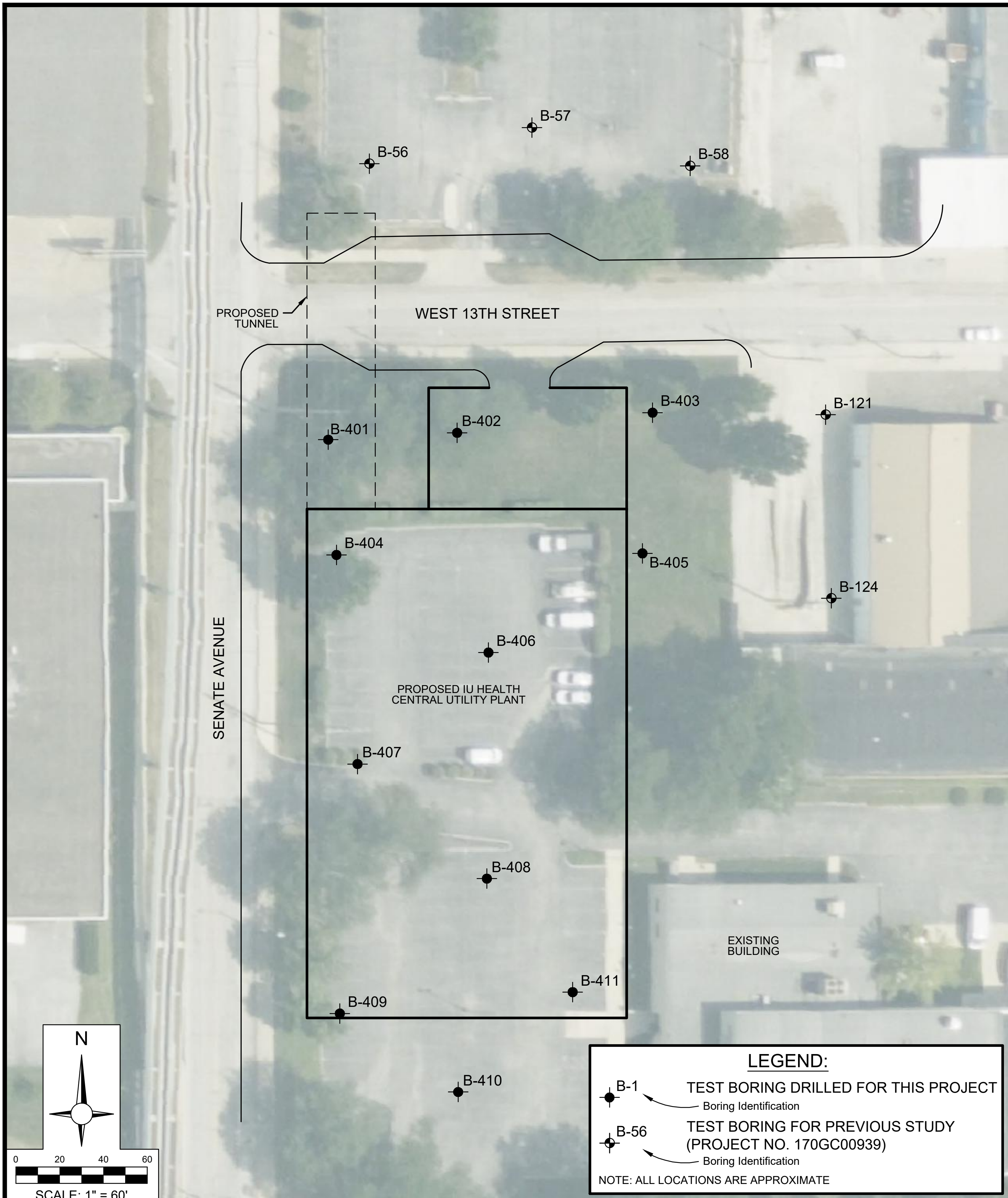

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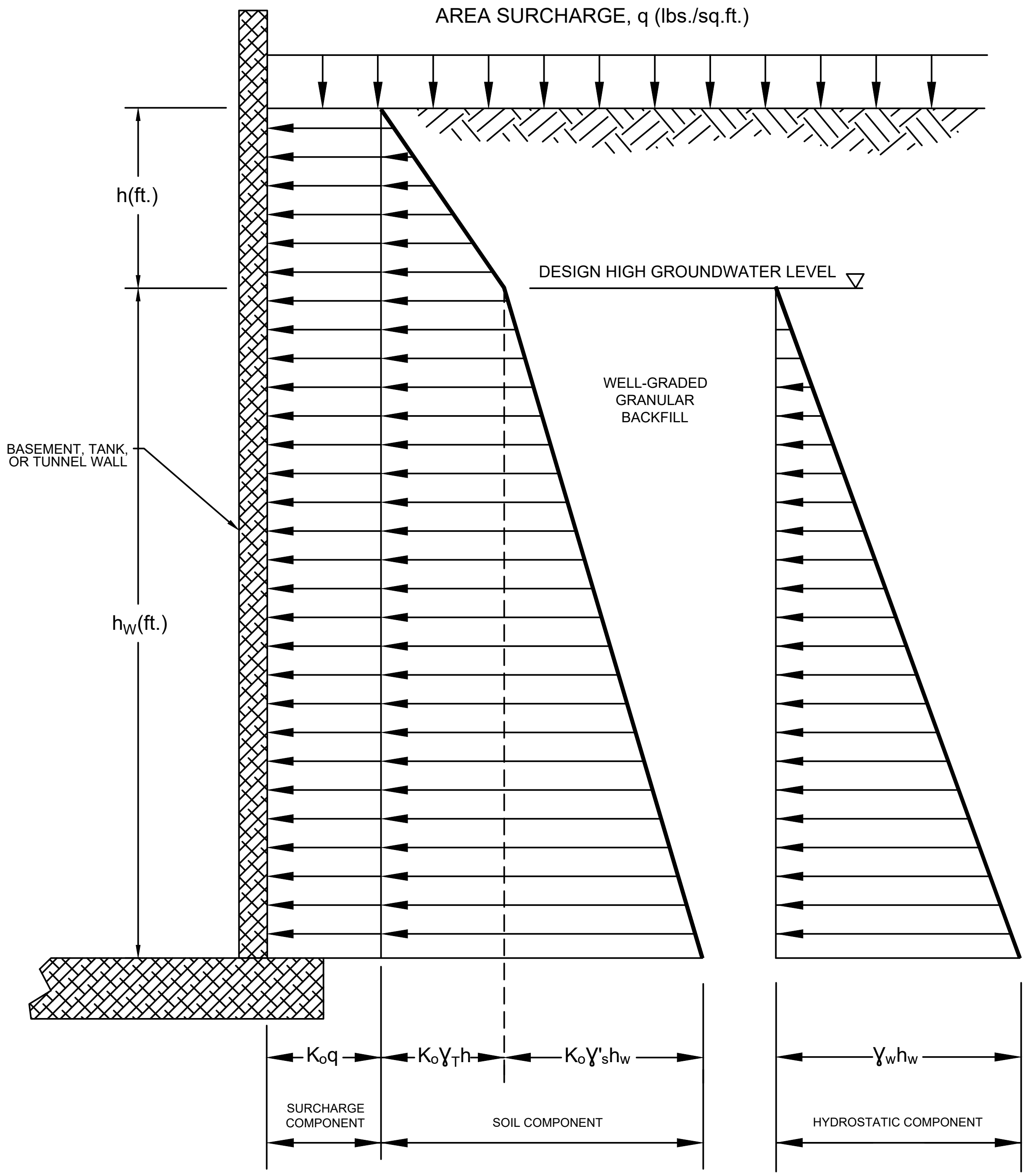
BORING PLAN

PROPOSED IU HEALTH CENTRAL UTILITY PLANT
IU HEALTH ACADEMIC HEALTH CENTER OF THE FUTURE
13TH STREET AND NORTH SENATE AVENUE
INDIANAPOLIS, INDIANA

Project Number: 170GC01425		Drn. By: RH
Date: 09/12/2022	Scale: AS SHOWN	Ckd. By: DM
ATLAS		1



γ'_s = SUBMERGED SOIL UNIT WEIGHT (lbs./cu.ft.)

γ_T = TOTAL SOIL UNIT WEIGHT (lbs./cu.ft.)

q = AREA SURCHARGE, q (lbs./sq.ft.)

γ_W = UNIT WEIGHT OF WATER (lbs./cu.ft.)

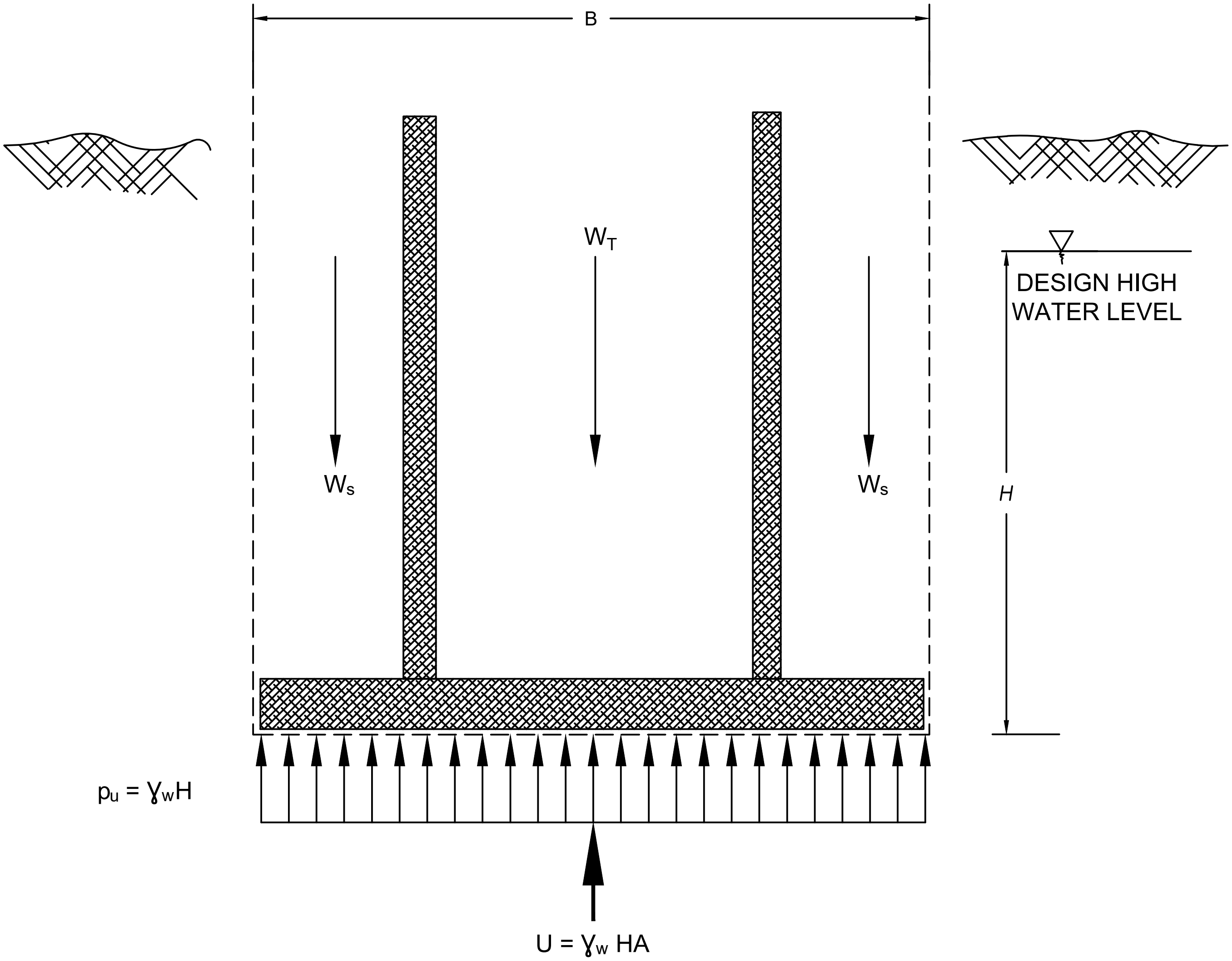
K_o = COEFFICIENT OF LATERAL EARTH PRESSURE AT-REST

h_W = DEPTH FROM DESIGN HIGH GROUND WATER LEVEL TO BASE OF WALL (ft.)

LATERAL EARTH PRESSURE AGAINST BELOW-GRADE WALL ASSUMING UNDRAINED BACKFILL W/ HYDROSTATIC PRESSURE

PROPOSED IU HEALTH CENTRAL UTILITY PLANT
IU HEALTH ACADEMIC HEALTH CENTER OF THE FUTURE
13TH STREET AND NORTH SENATE AVENUE
INDIANAPOLIS, INDIANA

Project Number: 170GC01425		Drn. By: RH
Date: 09/06/2022	Scale: NOT TO SCALE	Ckd. By: DM
ATLAS		3



NOTE: FOR THIS DESIGN APPROACH, TOTAL (NOT BUOYANT) WEIGHTS OF SOIL AND STRUCTURE MATERIALS WITHIN THE DASHED LINES SHOULD BE USED

H = DEPTH FROM DESIGN HIGH GROUND WATER LEVEL TO BOTTOM OF STRUCTURE (ft.)

γ_w = UNIT WEIGHT OF WATER (lbs./cu. ft.)

p_u = UPLIFT PRESSURE AT BASE OF FOUNDATION OR SLAB (lbs/sq.ft.)

U = TOTAL UPLIFT FORCE (lbs.)

W_T = WEIGHT OF STRUCTURE (lbs.)

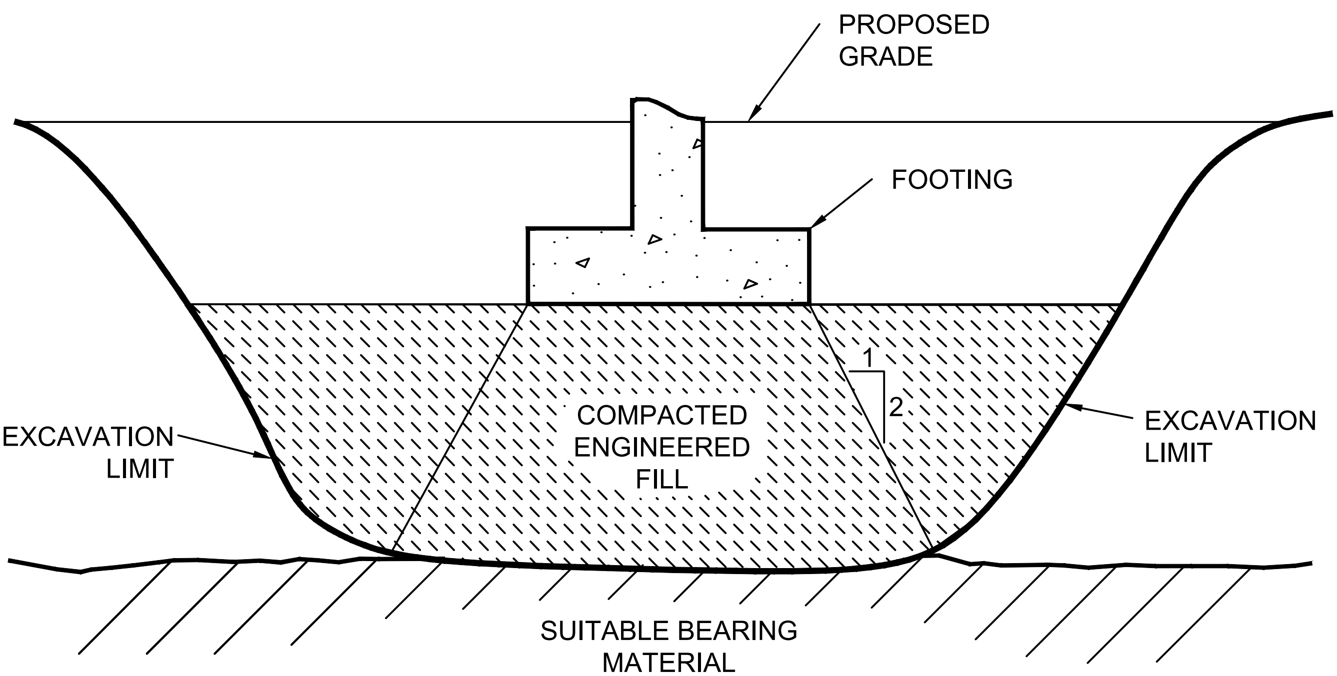
W_s = WEIGHT OF SOIL OVER FOUNDATION SLAB (lbs.)

A = AREA OF STRUCTURE BASE (sq.ft.)

DESIGN ILLUSTRATION-UPLIFT CONSIDERATION OF SUBMERGED BELOW-GRADE STRUCTURE

PROPOSED IU HEALTH CENTRAL UTILITY PLANT
IU HEALTH ACADEMIC HEALTH CENTER OF THE FUTURE
13TH STREET AND NORTH SENATE AVENUE
INDIANAPOLIS, INDIANA

Project Number: 170GC01425		Drn. By: RH
Date: 09/06/2022	Scale: NOT TO SCALE	Ckd. By: DM
ATLAS		4



DESIGN ILLUSTRATION
FOOTINGS WITH UNDERCUTS

PROPOSED IU HEALTH CENTRAL UTILITY PLANT
IU HEALTH ACADEMIC HEALTH CENTER OF THE FUTURE
13TH STREET AND NORTH SENATE AVENUE
INDIANAPOLIS, INDIANA

Project Number: 170GC01425		Drn. By: RH
Date: 09/06/2022	Scale: NOT TO SCALE	Ckd. By: DM
ATLAS		5

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TEST BORING LOG

CLIENT **Indiana University Health**

BORING # **B-401**

PROJECT NAME **Proposed IU Health Central Utility Plant**

JOB # **170GC01425**

PROJECT LOCATION **13th Street and North Senate Avenue**
Indianapolis, Indiana

DRILLING and SAMPLING INFORMATION

Date Started	**8/17/22**	Hammer Wt.	**140** lbs.
Date Completed	**8/17/22**	Hammer Drop	**30** in.
Drill Foreman	**G. Lauber**	Spoon Sampler OD	**2.0** in.
Inspector	**D. McIlwaine**	Rock Core Dia.	**--** in.
Boring Method	**HSA**	Shelby Tube OD	**--** in.

TEST DATA

SOIL CLASSIFICATION SURFACE ELEVATION 718	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
4 in. Topsoil	717.7	0.3									
Brown, slightly moist, silty clay with some sand, trace gravel, and trace brick fragments (FILL)				1	SS			7-9-9	6.3		Ground surface elevation estimated based on topographic mapping provided by IU Health.
				2	SS			6-5-5	7.2		
			5	3	SS			9-8-8	8.1		
				4	SS			10-8-50/0.3			
	707.0	11.0	10								Apparent concrete slab at approximately 9.5 ft.
Brown, slightly moist, very loose to dense, SAND (SW-SM) with some gravel and trace silt				5	SS			6-4-10			
			15	6	SS			2-1-3			
				7	SS			8-10-14			
			20	8	SS			4-9-14			
				9	SS			13-22-14			
			25	10	SS			5-13-24			
				11	SS			9-12-16			
			30	12	SS			16-17-16			
				13	SS			7-10-14			
	684.0	34.0									
Gray, wet, medium dense to dense, SAND (SP-SM) with some gravel and trace silt			35	14	SS		●	17-20-21			
				15	SS			10-17-22			
	678.0	40.0		16	SS			16-15-13			

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **34.0** ft.
At Completion **None** ft.
After **--** hours **--** ft.
Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT **Indiana University Health**
PROJECT NAME **Proposed IU Health Central Utility Plant**
PROJECT LOCATION **13th Street and North Senate Avenue**
Indianapolis, Indiana

BORING # **B-401**
JOB # **170GC01425**

DRILLING and SAMPLING INFORMATION

Date Started	**8/17/22**	Hammer Wt.	**140** lbs.
Date Completed	**8/17/22**	Hammer Drop	**30** in.
Drill Foreman	**G. Lauber**	Spoon Sampler OD	**2.0** in.
Inspector	**D. McIlwaine**	Rock Core Dia.	**--** in.
Boring Method	**HSA**	Shelby Tube OD	**--** in.

TEST DATA

SOIL CLASSIFICATION (continued)	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics / Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Gray, wet, medium dense to dense, SAND (SP-SM) with some gravel and trace silt											
				17	SS			18-20-22			
			45								
				18	SS			15-15-12			
	668.0	50.0	50								
Bottom of Test Boring at 50.0 ft.											

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
Noted on Drilling Tools **34.0** ft.
At Completion **None** ft.
After **--** hours **--** ft.
Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT: **Indiana University Health**
PROJECT NAME: **Proposed IU Health Central Utility Plant**
PROJECT LOCATION: **13th Street and North Senate Avenue**
Indianapolis, Indiana

BORING #: **B-402**
JOB #: **170GC01425**

DRILLING and SAMPLING INFORMATION

Date Started	**8/17/22**	Hammer Wt.	**140** lbs.
Date Completed	**8/17/22**	Hammer Drop	**30** in.
Drill Foreman	**G. Lauber**	Spoon Sampler OD	**2.0** in.
Inspector	**D. McIlwaine**	Rock Core Dia.	**--** in.
Boring Method	**HSA**	Shelby Tube OD	**--** in.

TEST DATA

SOIL CLASSIFICATION SURFACE ELEVATION 719	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
4 in. Topsoil	718.7	0.3								Ground surface elevation estimated based on topographic mapping provided by IU Health.
Dark gray, slightly moist, sandy silty clay with trace gravel (FILL)	716.0	3.0		1	SS		6-4-5			
Brown, slightly moist, silty clay with trace sand (FILL)	713.0	6.0	5	2	SS		6-5-6	6.9		
Brown, moist, stiff, SILT (ML) with some sand	710.5	8.5		3	SS		5-7-7	10.4		Atterberg Limits: Non-Plastic
Brown, slightly moist, loose to medium dense, SAND (SW-SM) with little gravel and trace silt			10	4	SS		3-2-6			
				5	SS		10-9-10			
			15	6	SS		9-6-8			
				7	SS	Cave Depth (15.8)	7-8-7			
			20	8	SS		5-6-10			
	694.0	25.0	25	9	SS		11-12-15			
Bottom of Test Boring at 25.0 ft.										

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
Noted on Drilling Tools **None** ft.
At Completion **None** ft.
After **--** hours **--** ft.
Cave Depth **15.8** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT: **Indiana University Health**
BORING #: **B-403**
PROJECT NAME: **Proposed IU Health Central Utility Plant**
JOB #: **170GC01425**
PROJECT LOCATION: **13th Street and North Senate Avenue**
Indianapolis, Indiana

DRILLING and SAMPLING INFORMATION

Date Started	**8/17/22**	Hammer Wt.	**140** lbs.
Date Completed	**8/17/22**	Hammer Drop	**30** in.
Drill Foreman	**G. Lauber**	Spoon Sampler OD	**2.0** in.
Inspector	**D. McIlwaine**	Rock Core Dia.	**--** in.
Boring Method	**HSA**	Shelby Tube OD	**--** in.

TEST DATA

SURFACE ELEVATION 719

SOIL CLASSIFICATION	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
4 in. Topsoil	718.7	0.3								Ground surface elevation estimated based on topographic mapping provided by IU Health.
Brown, slightly moist, silty clay with little to some sand and trace gravel (FILL)				1	SS		7-8-8	9.2		
	713.5	5.5		2	SS		7-3-4	10.2		
Gray and brown, slightly moist, gravel with trace sand (FILL)			5	3	SS		7-10-12			
				4	SS		8-9-12			
	708.0	11.0	10							
Brown, slightly moist, medium dense, SAND and GRAVEL (SW-SM) with trace silt	706.0	13.0		5	SS		8-11-12			
Brown, slightly moist, loose, SAND (SP) with trace silt	703.5	15.5	15	6	SS	Cave	4-3-5			
Brown, slightly moist, medium dense to dense, SAND (SW-SM) with some gravel and trace silt				7	SS		5-6-8			
				8	SS		5-7-8			
			20							
	694.0	25.0	25	9	SS		14-16-20			
Bottom of Test Boring at 25.0 ft.										

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
Noted on Drilling Tools **None** ft.
At Completion **None** ft.
After **--** hours **--** ft.
Cave Depth **14.6** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT **Indiana University Health**
PROJECT NAME **Proposed IU Health Central Utility Plant**
PROJECT LOCATION **13th Street and North Senate Avenue**
Indianapolis, Indiana

BORING # **B-404**
JOB # **170GC01425**

DRILLING and SAMPLING INFORMATION

Date Started	8/18/22	Hammer Wt.	140 lbs.
Date Completed	8/18/22	Hammer Drop	30 in.
Drill Foreman	G. Lauber	Spoon Sampler OD	2.0 in.
Inspector	D. McIlwaine	Rock Core Dia.	-- in.
Boring Method	HSA	Shelby Tube OD	-- in.

TEST DATA

SOIL CLASSIFICATION — SURFACE ELEVATION 718

Soil Classification	Stratum Elevation	Stratum Depth, ft	Sample No.	Sample Type	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
5 in. Asphalt	717.6	0.4							Ground surface elevation estimated based on topographic mapping provided by IU Health.
Bricks with trace sand and gravel (FILL)	714.0	4.0	1	SS		3-5-5			
Brown, slightly moist, silty clay with little sand and trace brick fragments (FILL)	712.0	6.0	2	SS		6-7-5	7.3		
Brown, moist, medium stiff, SANDY SILTY CLAY (CL)	710.0	8.0	3	SS		4-4-6	15.0	2.0	
Dark brown, moist, very loose, CLAYEY SAND (SC) with trace gravel	707.0	11.0	4	SS		2-2-2			
Brown, slightly moist, medium dense to dense, SAND (SW-SM) with some gravel and trace silt			5	SS		8-9-10			
			6	SS		9-9-9			
			7	SS		9-11-12			
			8	SS		7-10-9			
			9	SS		7-12-13			
			10	SS		7-8-12			
			11	SS		7-10-10			
			12	SS		8-18-21			
			13	SS		9-16-17			
- moist below 33 ft.	683.0	35.0	14	SS	●	12-12-13			
Gray, wet, medium dense, SAND and GRAVEL (SW-SM) with trace silt			15	SS		21-13-12			
	678.0	40.0	16	SS		10-12-12			

Depth Scale, ft: 5, 10, 15, 20, 25, 30, 35

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools 35.0 ft.
At Completion None ft.
After -- hours -- ft.
Cave Depth -- ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT **Indiana University Health**
PROJECT NAME **Proposed IU Health Central Utility Plant**
PROJECT LOCATION **13th Street and North Senate Avenue**
Indianapolis, Indiana

BORING # **B-404**
JOB # **170GC01425**

DRILLING and SAMPLING INFORMATION

Date Started	**8/18/22**	Hammer Wt.	**140** lbs.
Date Completed	**8/18/22**	Hammer Drop	**30** in.
Drill Foreman	**G. Lauber**	Spoon Sampler OD	**2.0** in.
Inspector	**D. McIlwaine**	Rock Core Dia.	**--** in.
Boring Method	**HSA**	Shelby Tube OD	**--** in.

TEST DATA

SOIL CLASSIFICATION (continued)	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics / Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Gray, wet, medium dense, SAND and GRAVEL (SW-SM) with trace silt	675.0	43.0									
Brown, wet, dense to medium dense, SAND (SP-SM) with some gravel and trace silt			45	17	SS			20-19-18			
	668.0	50.0	50	18	SS			12-10-11			
Bottom of Test Boring at 50.0 ft.											

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
Noted on Drilling Tools **35.0** ft.
At Completion **None** ft.
After **--** hours **--** ft.
Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT **Indiana University Health**
PROJECT NAME **Proposed IU Health Central Utility Plant**
PROJECT LOCATION **13th Street and North Senate Avenue**
Indianapolis, Indiana

BORING # **B-405**
JOB # **170GC01425**

DRILLING and SAMPLING INFORMATION

Date Started	**8/17/22**	Hammer Wt.	**140** lbs.
Date Completed	**8/17/22**	Hammer Drop	**30** in.
Drill Foreman	**G. Lauber**	Spoon Sampler OD	**2.0** in.
Inspector	**D. McIlwaine**	Rock Core Dia.	**--** in.
Boring Method	**HSA**	Shelby Tube OD	**--** in.

TEST DATA

SOIL CLASSIFICATION SURFACE ELEVATION 720	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
3 in. Topsoil	719.7	0.3								Ground surface elevation estimated based on topographic mapping provided by IU Health.
Gray, slightly moist, silty clay with sand, trace gravel, and trace brick fragments (FILL)				1	SS		3-3-3			
				2	SS		2-2-1	14.2		
	714.0	6.0	5							
Brown, moist, medium stiff, CLAY (CH) with little sand and gravel				3	SS		2-3-5	19.3		Atterberg Limits: LL=54 PL=18 PI=36
	711.5	8.5								
Brown, slightly moist, medium dense, SAND (SP-SM) with little gravel and trace silt				4	SS		4-5-6			
			10	5	SS		5-6-9			
				6	SS	Cave	5-6-8			
	704.5	15.5	15							
Brown, slightly moist, medium dense to dense, SAND (SW-SM) with little to some gravel and trace silt				7	SS		5-7-7			
				8	SS		8-8-8			
			20							
				9	SS		14-16-17			
	695.0	25.0	25							
Bottom of Test Boring at 25.0 ft.										

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
Noted on Drilling Tools **None** ft.
At Completion **None** ft.
After **--** hours **--** ft.
Cave Depth **14.6** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT: **Indiana University Health**
PROJECT NAME: **Proposed IU Health Central Utility Plant**
PROJECT LOCATION: **13th Street and North Senate Avenue**
Indianapolis, Indiana

BORING #: **B-406**
JOB #: **170GC01425**

DRILLING and SAMPLING INFORMATION

Date Started	**8/16/22**	Hammer Wt.	**140** lbs.
Date Completed	**8/16/22**	Hammer Drop	**30** in.
Drill Foreman	**G. Lauber**	Spoon Sampler OD	**2.0** in.
Inspector	**D. McIlwaine**	Rock Core Dia.	**--** in.
Boring Method	**HSA**	Shelby Tube OD	**--** in.

TEST DATA

SOIL CLASSIFICATION
SURFACE ELEVATION 719

Soil Classification	Stratum Elevation	Stratum Depth, ft	Sample No.	Sample Type	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
4 in. Asphalt	718.7	0.3						Ground surface elevation estimated based on topographic mapping provided by IU Health.
Gray and black, moist, silty clay with little sand, trace gravel, and trace brick fragments (FILL)	715.5	3.5	1	SS	2-3-4			
Brown, moist, medium stiff, SANDY SILTY CLAY (CL) with trace gravel	713.0	6.0	2	SS	4-4-3	14.9	2.75	Atterberg Limits: LL=42 PL=16 PI=26
Brown, slightly moist, medium dense, SAND (SP-SC) with little gravel and trace clay	711.0	8.0	3	SS	5-6-7			
Brown, slightly moist, medium dense, SAND (SP-SM) with little gravel and trace silt	701.0	18.0	4	SS	6-7-7			
			5	SS	8-11-10			
			6	SS	8-9-9			
			7	SS	11-12-15			
Gray, slightly moisst, medium dense to dense, SAND (SW-SM) with some gravel and trace silt	694.0	25.0	8	SS	6-9-7			
			9	SS	20-19-26			
Bottom of Test Boring at 25.0 ft.								

Depth Scale, ft: 5, 10, 15, 20, 25

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
Noted on Drilling Tools **None** ft.
At Completion **None** ft.
After **--** hours **--** ft.
Cave Depth **12.2** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT: **Indiana University Health**
PROJECT NAME: **Proposed IU Health Central Utility Plant**
PROJECT LOCATION: **13th Street and North Senate Avenue**
Indianapolis, Indiana

BORING #: **B-407**
JOB #: **170GC01425**

DRILLING and SAMPLING INFORMATION

Date Started	**8/19/22**	Hammer Wt.	**140** lbs.
Date Completed	**8/19/22**	Hammer Drop	**30** in.
Drill Foreman	**G. Lauber**	Spoon Sampler OD	**2.0** in.
Inspector	**D. McIlwaine**	Rock Core Dia.	**--** in.
Boring Method	**HSA**	Shelby Tube OD	**--** in.

TEST DATA

SURFACE ELEVATION 719

SOIL CLASSIFICATION	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
5 in. Asphalt	718.6	0.4								Ground surface elevation estimated based on topographic mapping provided by IU Health.
Gray and black, slightly moist, sand with little gravel and trace asphalt fragments (FILL)	715.5	3.5		1	SS		8-5-5			
Dark brown, moist, soft, SILTY CLAY (CL) with little sand	713.0	6.0	5	2	SS		3-2-2	17.2		Atterberg Limits: LL=29 PL=16 PI=13
Dark brown, moist, medium dense, CLAYEY SAND (SC) with trace gravel	711.0	8.0		3	SS		5-5-6			
Dark brown, moist, medium stiff, SILTY CLAY (CL) with little sand and trace gravel	709.5	9.5	10	4	SS		3-3-4	18.9		
Brown, slightly moist, soft, SANDY SILTY CLAY (CL) with trace gravel	705.5	13.5		5	SS		3-2-2	11.6		
Brown, slightly moist, loose to dense, SAND (SM) with little gravel and silt			15	6	SS	Cave	3-4-3			
				7	SS		7-7-6			
			20	8	SS		7-7-8			
			25	9	SS		8-12-10			
	689.0	30.0	30	10	SS		6-17-22			
Bottom of Test Boring at 30.0 ft.										

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
Noted on Drilling Tools **None** ft.
At Completion **None** ft.
After **--** hours **--** ft.
Cave Depth **15.2** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT **Indiana University Health**
PROJECT NAME **Proposed IU Health Central Utility Plant**
PROJECT LOCATION **13th Street and North Senate Avenue**
Indianapolis, Indiana

BORING # **B-408**
JOB # **170GC01425**

DRILLING and SAMPLING INFORMATION

Date Started	**8/16/22**	Hammer Wt.	**140** lbs.
Date Completed	**8/16/22**	Hammer Drop	**30** in.
Drill Foreman	**G. Lauber**	Spoon Sampler OD	**2.0** in.
Inspector	**D. McIlwaine**	Rock Core Dia.	**--** in.
Boring Method	**HSA**	Shelby Tube OD	**--** in.

TEST DATA

SURFACE ELEVATION 719

SOIL CLASSIFICATION	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
3 in. Asphalt over 10 in. Aggregate Base	717.9	1.1								Ground surface elevation estimated based on topographic mapping provided by IU Health.
Brown, slightly moist, sand with some gravel and little silt (FILL)	715.5	3.5		1	SS		5-3-3			
Dark brown, moist, very soft, SANDY SILTY CLAY (CL) with trace gravel	713.0	6.0	5	2	SS		3-2-1	20.1	1.25	Atterberg Limits: LL=34 PL=17 PI=17
Brown, slightly moist, loose to medium dense, SAND (SP-SM) with little gravel and trace silt				3	SS		6-5-7			
			10	4	SS		8-9-8			
				5	SS		8-9-10			
			15	6	SS		4-4-5			
				7	SS		11-9-13			
			20	8	SS		6-8-13			
			25	9	SS		8-11-15			
	685.0	34.0	30	10	SS		14-16-14			
Gray, wet, medium dense to very dense, SAND (SW-SM) with little to some gravel and trace silt			35	11	SS	●	8-15-17			
	679.0	40.0		12	SS		13-13-13			

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **34.0** ft.
At Completion **None** ft.
After **--** hours **--** ft.
Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT: **Indiana University Health**
PROJECT NAME: **Proposed IU Health Central Utility Plant**
PROJECT LOCATION: **13th Street and North Senate Avenue**
Indianapolis, Indiana

BORING #: **B-408**
JOB #: **170GC01425**

DRILLING and SAMPLING INFORMATION

Date Started	**8/16/22**	Hammer Wt.	**140** lbs.
Date Completed	**8/16/22**	Hammer Drop	**30** in.
Drill Foreman	**G. Lauber**	Spoon Sampler OD	**2.0** in.
Inspector	**D. McIlwaine**	Rock Core Dia.	**--** in.
Boring Method	**HSA**	Shelby Tube OD	**--** in.

TEST DATA

SOIL CLASSIFICATION (continued)	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Gray, wet, medium dense to very dense, SAND (SW-SM) with little to some gravel and trace silt											
			45	13	SS			19-24-29			
	669.0	50.0	50	14	SS			10-19-23			
Bottom of Test Boring at 50.0 ft.											

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
Noted on Drilling Tools **34.0** ft.
At Completion **None** ft.
After **--** hours **--** ft.
Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT: **Indiana University Health**
PROJECT NAME: **Proposed IU Health Central Utility Plant**
PROJECT LOCATION: **13th Street and North Senate Avenue**
Indianapolis, Indiana

BORING #: **B-409**
JOB #: **170GC01425**

DRILLING and SAMPLING INFORMATION

Date Started	8/17/22	Hammer Wt.	140 lbs.
Date Completed	8/17/22	Hammer Drop	30 in.
Drill Foreman	G. Lauber	Spoon Sampler OD	2.0 in.
Inspector	D. McIlwaine	Rock Core Dia.	-- in.
Boring Method	HSA	Shelby Tube OD	-- in.

TEST DATA

SOIL CLASSIFICATION

SURFACE ELEVATION 719

Soil Classification	Stratum Elevation	Stratum Depth, ft
3 in. Topsoil	718.7	0.3
Brown, slightly moist, silty sand with little gravel (FILL)	715.5	3.5
Brown, slightly moist, stiff, SILTY CLAY (CL) with some sand and trace gravel	713.0	6.0
Dark brown and brown, slightly moist, medium dense, SAND (SP-SC) with little gravel and trace clay	711.0	8.0
Brown, slightly moist, loose to dense, SAND (SP-SM) with little gravel and trace silt	689.0	30.0
Bottom of Test Boring at 30.0 ft.		

Sample No.	Sample Type	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
1	SS		7-11-13			Ground surface elevation estimated based on topographic mapping provided by IU Health.
2	SS		5-6-9	15.7	4.5+	
3	SS		6-4-7			
4	SS		4-4-5			
5	SS	Cave Depth (13.3 ft)	4-5-7			
6	SS		7-8-7			
7	SS		7-7-8			
8	SS		7-9-12			
9	SS		10-9-13			
10	SS		11-19-22			

Depth Scale, ft: 5, 10, 15, 20, 25, 30

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
Noted on Drilling Tools **None** ft.
At Completion **None** ft.
After **--** hours **--** ft.
Cave Depth **13.3** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT: **Indiana University Health**
PROJECT NAME: **Proposed IU Health Central Utility Plant**
PROJECT LOCATION: **13th Street and North Senate Avenue, Indianapolis, Indiana**
BORING #: **B-410**
JOB #: **170GC01425**

DRILLING and SAMPLING INFORMATION

Date Started	**8/17/22**	Hammer Wt.	**140** lbs.
Date Completed	**8/17/22**	Hammer Drop	**30** in.
Drill Foreman	**G. Lauber**	Spoon Sampler OD	**2.0** in.
Inspector	**D. McIlwaine**	Rock Core Dia.	**--** in.
Boring Method	**HSA**	Shelby Tube OD	**--** in.

TEST DATA

SOIL CLASSIFICATION SURFACE ELEVATION 718	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
3 in. Asphalt	717.7	0.3								Ground surface elevation estimated based on topographic mapping provided by IU Health.
Gray, dark gray, and black, moist, silty clay with some sand and little gravel (FILL)	714.5	3.5		1	SS		5-3-6	16.9		
Brown, slightly moist, medium dense to dense, SAND (SP-SM) with little gravel and trace silt				2	SS		9-8-8			
			5	3	SS		9-9-11			
				4	SS		7-6-11			
			10	5	SS		10-11-8			
				6	SS		3-8-11			
			15	7	SS		7-11-10			
						Cave (18.2)				
			20	8	SS		8-11-14			
			25	9	SS		10-15-16			
	688.0	30.0	30	10	SS		13-12-17			
Bottom of Test Boring at 30.0 ft.										

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
Noted on Drilling Tools **None** ft.
At Completion **None** ft.
After **--** hours **--** ft.
Cave Depth **18.2** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT **Indiana University Health**
PROJECT NAME **Proposed IU Health Central Utility Plant**
PROJECT LOCATION **13th Street and North Senate Avenue**
Indianapolis, Indiana

BORING # **B-411**
JOB # **170GC01425**

DRILLING and SAMPLING INFORMATION

Date Started	8/17/22	Hammer Wt.	140 lbs.
Date Completed	8/17/22	Hammer Drop	30 in.
Drill Foreman	G. Lauber	Spoon Sampler OD	2.0 in.
Inspector	D. McIlwaine	Rock Core Dia.	-- in.
Boring Method	HSA	Shelby Tube OD	-- in.

TEST DATA

SOIL CLASSIFICATION (SURFACE ELEVATION 719)	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
4 in. Asphalt	718.7	0.3								Ground surface elevation estimated based on topographic mapping provided by IU Health. Atterberg Limits: LL=62 PL=20 PI=42
Dark gray and black, moist, silty clay with some sand and little gravel (FILL)	715.5	3.5		1	SS		6-4-4	13.5		
Brown, slightly moist, medium dense to dense, SAND (SW-SM) with little gravel and trace silt				2	SS		8-9-9			
			5	3	SS		10-13-16			
				4	SS		10-12-13			
			10	5	SS		10-11-10			
				6	SS		4-6-8			
			15	7	SS	Cave	9-10-13			
				8	SS		7-8-13			
			20							
			25	9	SS		13-9-12			
	689.0	30.0	30	10	SS		15-18-21			
Bottom of Test Boring at 30.0 ft.										

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
Noted on Drilling Tools **None** ft.
At Completion **None** ft.
After -- hours -- ft.
Cave Depth **16.8** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

ATLAS
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TEST BORING LOG

CLIENT **Indiana University Health**
PROJECT NAME **Proposed IU Health Academic Health Center**
PROJECT LOCATION **West 16th Street and North Senate Avenue**
Indianapolis, Indiana

BORING # **B-56**
JOB # **170GC00939**

DRILLING and SAMPLING INFORMATION

Date Started	**12/23/19**	Hammer Wt.	**140** lbs.
Date Completed	**12/23/19**	Hammer Drop	**30** in.
Drill Foreman	**G. Lauber**	Spoon Sampler OD	**2.0** in.
Inspector	**D. McIlwaine**	Rock Core Dia.	**--** in.
Boring Method	**HSA**	Shelby Tube OD	**--** in.

SOIL CLASSIFICATION

SURFACE ELEVATION 719

Soil Classification	Stratum Elevation	Stratum Depth, ft
3 in. Asphalt over 4 in. Gravel	718.4	0.6
Gray and brown, moist, sandy silty clay with little gravel and brick fragments (FILL)	715.5	3.5
Brown, moist, soft to medium stiff, SILTY CLAY (CL) with little to some sand and trace gravel	711.0	8.0
Brown, slightly moist, loose to very loose, SAND (SP) with little gravel	705.5	13.5
Brown, slightly moist, medium dense to very dense, SAND (SP) with little to some gravel - cobbles between 21.0 and 38.5 ft	682.0	37.0
Gray, wet, very dense to dense, SAND (SP-SM) with little gravel and trace silt		

TEST DATA

Sample No.	Sample Type	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf
1	SS	3-3-6	13.9	
2	SS	2-2-3	14.2	1.0
3	SS	3-3-3	18.5	1.5
4	SS	4-2-4		
5	SS	5-2-2		
6	SS	3-5-7		
7	SS	11-11-10		
8	SS	7-8-6		
9	SS	32-50/0.2		
10	SS	50/0.4		
11	SS	32-50/0.4		
12	SS	50/0.3		
13	SS	35-42-47		
14	SS	26-29-30		

Depth Scale, ft: 5, 10, 15, 20, 25, 30, 35

Groundwater: noted on drilling tools at 37.0 ft.

Remarks

Ground surface elevation estimated from topographic map provided by client (survey performed by the Schneider Corporation, dated August 14, 2019).

Sample No. 2:
Atterberg Limits:
LL=17 PL=12 PI=5

Boring backfilled with bentonite grout by tremie as augers were withdrawn from the boring.

Boring patched with concrete at the surface.

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
Noted on Drilling Tools **37.0** ft.
At Completion **--** ft.
After **--** hours **--** ft.
Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT	Indiana University Health	BORING #	B-56
PROJECT NAME	Proposed IU Health Academic Health Center	JOB #	170GC00939
PROJECT LOCATION	West 16th Street and North Senate Avenue Indianapolis, Indiana		

DRILLING and SAMPLING INFORMATION

Date Started	12/23/19	Hammer Wt.	140 lbs.
Date Completed	12/23/19	Hammer Drop	30 in.
Drill Foreman	G. Lauber	Spoon Sampler OD	2.0 in.
Inspector	D. McIlwaine	Rock Core Dia.	-- in.
Boring Method	HSA	Shelby Tube OD	-- in.

TEST DATA

SOIL CLASSIFICATION (continued)	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Gray, wet, very dense to dense, SAND (SP-SM) with little gravel and trace silt											
			45	15	SS			18-27-30			
			50	16	SS			15-23-26			
			55	17	SS			18-18-22			
	659.0	60.0	60	18	SS			15-15-20			
Bottom of Test Boring at 60.0 ft											

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
Noted on Drilling Tools 37.0 ft.
At Completion -- ft.
After -- hours -- ft.
Cave Depth -- ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT: Indiana University Health
PROJECT NAME: Proposed IU Health Academic Health Center
PROJECT LOCATION: West 16th Street and North Senate Avenue, Indianapolis, Indiana
BORING #: B-57
JOB #: 170GC00939

DRILLING and SAMPLING INFORMATION

Date Started	12/23/19	Hammer Wt.	140 lbs.
Date Completed	12/23/19	Hammer Drop	30 in.
Drill Foreman	G. Lauber	Spoon Sampler OD	2.0 in.
Inspector	D. McIlwaine	Rock Core Dia.	-- in.
Boring Method	HSA	Shelby Tube OD	-- in.

TEST DATA

SOIL CLASSIFICATION — SURFACE ELEVATION 718

Soil Classification	Stratum Elevation	Stratum Depth, ft	Sample No.	Sample Type	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
3 in. Asphalt	717.7	0.3							Ground surface elevation estimated from topographic map provided by client (survey performed by the Schneider Corporation, dated August 14, 2019).
Gray and brown, moist, sandy silty clay with little gravel (FILL)	714.5	3.5	1	SS		3-3-2	12.5		
Brown, moist, very soft to medium stiff, SILTY CLAY (CL) with some sand and trace gravel	710.0	8.0	2	SS		2-1-2	16.5		
			3	SS		4-4-6	19.1	2.0	
Brown and dark brown, slightly moist, loose, SAND (SP-SC) with little gravel and trace clay	707.0	11.0	4	SS		2-1-7			
Brown, slightly moist, medium dense, SAND (SP) with little to some gravel	697.0	21.0	5	SS		9-8-7			
			6	SS		8-8-8			
			7	SS		8-11-15			
			8	SS		6-5-8			
Brown, slightly moist, dense to medium dense, SAND (SP) with little to some gravel	690.0	28.0	9	SS		18-16-18			Boring backfilled with bentonite grout by tremie as augers were withdrawn from the boring.
			10	SS		8-16-18			
			11	SS		11-13-13			Boring patched with concrete at the surface.
Brown, moist, medium dense, SAND (SP-SM) with some gravel and trace silt	684.5	33.5	12	SS		10-11-12			
Brown, wet, medium dense to dense, SAND (SP-SM) with some gravel and little silt			13	SS	● (33.5 ft)	10-12-13			
Bottom of Test Boring at 40.0 ft	678.0	40.0	14	SS		10-18-20			

Depth Scale, ft: 5, 10, 15, 20, 25, 30, 35

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools 33.5 ft.
▽ At Completion -- ft.
▼ After -- hours -- ft.
Cave Depth -- ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT **Indiana University Health**
BORING # **B-58**
PROJECT NAME **Proposed IU Health Academic Health Center**
JOB # **170GC00939**
PROJECT LOCATION **West 16th Street and North Senate Avenue**
Indianapolis, Indiana

DRILLING and SAMPLING INFORMATION

Date Started	**12/23/19**	Hammer Wt.	**140** lbs.
Date Completed	**12/23/19**	Hammer Drop	**30** in.
Drill Foreman	**G. Lauber**	Spoon Sampler OD	**2.0** in.
Inspector	**D. McIlwaine**	Rock Core Dia.	**--** in.
Boring Method	**HSA**	Shelby Tube OD	**--** in.

TEST DATA

SOIL CLASSIFICATION SURFACE ELEVATION 719	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
5 in. Asphalt over 4 in. Gravel	718.2	0.8								Ground surface elevation estimated from topographic map provided by client (survey performed by the Schneider Corporation, dated August 14, 2019).
Dark brown, moist, silty clay with some sand, trace gravel, and trace brick fragments (FILL)				1	SS		3-3-3	18.8		
	715.5	3.5								
Brown, moist, medium stiff to very soft, SILTY CLAY (CL) with some sand and trace gravel				2	SS		3-3-5	21.3	1.5	
			5							
				3	SS		1-1-2	15.5		
	710.5	8.5								
Brown, slightly moist, loose to medium dense, SAND (SP) with little to some gravel				4	SS		6-6-7			
			10							
				5	SS		6-4-5			
				6	SS		6-6-7			
			15							
				7	SS		8-12-12			
				8	SS		4-4-4			
			20							
	698.0	21.0								Boring backfilled with bentonite grout by tremie as augers were withdrawn from the boring.
Brown, slightly moist, medium dense, SAND (SP) with little to some gravel				9	SS		7-12-15			
				10	SS		12-15-14			
			25							Boring patched with concrete at the surface.
				11	SS		8-7-12			
				12	SS		9-11-13			
			30							
	685.5	33.5				●				
Brown, wet, medium dense to dense, SAND (SP-SM) with some gravel and little silt				13	SS		6-11-12			
			35							
				14	SS		16-17-19			

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **33.5** ft.
At Completion **--** ft.
After **--** hours **--** ft.
Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT: **Indiana University Health**
PROJECT NAME: **Proposed IU Health Academic Health Center**
PROJECT LOCATION: **West 16th Street and North Senate Avenue**
Indianapolis, Indiana

BORING #: **B-58**
JOB #: **170GC00939**

DRILLING and SAMPLING INFORMATION

Date Started	12/23/19	Hammer Wt.	140 lbs.
Date Completed	12/23/19	Hammer Drop	30 in.
Drill Foreman	G. Lauber	Spoon Sampler OD	2.0 in.
Inspector	D. McIlwaine	Rock Core Dia.	-- in.
Boring Method	HSA	Shelby Tube OD	-- in.

TEST DATA

SOIL CLASSIFICATION (continued)	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Brown, wet, medium dense to dense, SAND (SP-SM) with some gravel and little silt			45	15	SS			17-18-19			
Gray, wet, medium dense, SAND (SP-SM) with little gravel and trace silt	671.0	48.0									
Bottom of Test Boring at 50.0 ft	669.0	50.0	50	16	SS			11-11-13			

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
Noted on Drilling Tools 33.5 ft.
At Completion -- ft.
After -- hours -- ft.
Cave Depth -- ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT **Indiana University Health**
BORING # **B-121**
PROJECT NAME **Proposed IU Health Academic Health Center**
JOB # **170GC00939**
PROJECT LOCATION **West 16th Street and North Senate Avenue**
Indianapolis, Indiana

DRILLING and SAMPLING INFORMATION

Date Started	**1/12/20**	Hammer Wt.	**140** lbs.
Date Completed	**1/12/20**	Hammer Drop	**30** in.
Drill Foreman	**C. Carroll**	Spoon Sampler OD	**2.0** in.
Inspector	**D. McIlwaine**	Rock Core Dia.	**--** in.
Boring Method	**HSA**	Shelby Tube OD	**--** in.

TEST DATA

SOIL CLASSIFICATION SURFACE ELEVATION 720	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
3 in. Concrete over 8 in. Aggregate Base	719.1	0.9								Ground surface elevation estimated from topographic map provided by client (survey performed by the Schneider Corporation, dated August 14, 2019).
Gray, moist, silty sand with little gravel (FILL)	717.0	3.0		1	SS		6-2-3			
Brown and gray, moist, silty clay with little sand and gravel (FILL)	711.5	8.5	5	2	SS		4-2-3	23.7		
				3	SS		1-2-4	27.6		
Brown, slightly moist, medium dense, SAND (SW) with little gravel	704.5	15.5	10	4	SS		11-14-15			
				5	SS		14-15-12			
			15	6	SS		10-14-16			
Brown, slightly moist, medium dense, SAND (SW-SM) with trace gravel and silt	699.0	21.0		7	SS		10-11-15			
			20	8	SS		6-6-7			
Brown, slightly moist, medium dense to dense, SAND (SW) with little to some gravel	683.5	36.5		9	SS		10-9-11			Boring backfilled with bentonite grout by tremie as augers were withdrawn from the boring.
			25	10	SS		16-14-14			
				11	SS		10-11-14			Boring patched with concrete at the surface.
			30	12	SS		12-15-19			
			35	13	SS		22-22-26			
Brown, wet, medium dense, SAND (SW-SM) with some gravel and little silt						● (36.5)				
				14	SS		7-9-17			

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **36.5** ft.
▽ At Completion **--** ft.
▼ After **--** hours **--** ft.
Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT **Indiana University Health**

BORING # **B-121**

PROJECT NAME **Proposed IU Health Academic Health Center**

JOB # **170GC00939**

PROJECT LOCATION **West 16th Street and North Senate Avenue**
Indianapolis, Indiana

DRILLING and SAMPLING INFORMATION

Date Started	**1/12/20**	Hammer Wt.	**140** lbs.
Date Completed	**1/12/20**	Hammer Drop	**30** in.
Drill Foreman	**C. Carroll**	Spoon Sampler OD	**2.0** in.
Inspector	**D. McIlwaine**	Rock Core Dia.	**--** in.
Boring Method	**HSA**	Shelby Tube OD	**--** in.

TEST DATA

SOIL CLASSIFICATION (continued)	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Brown, wet, medium dense, SAND (SW-SM) with some gravel and little silt			45	15	SS			14-14-15			
Gray, wet, medium dense, SILTY SAND (SM) with little gravel	672.0	48.0		16	SS			18-14-15			
Bottom of Test Boring at 50.0 ft	670.0	50.0	50								

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
Noted on Drilling Tools **36.5** ft.
At Completion **--** ft.
After **--** hours **--** ft.
Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT: **Indiana University Health**
PROJECT NAME: **Proposed IU Health Academic Health Center**
PROJECT LOCATION: **West 16th Street and North Senate Avenue**
Indianapolis, Indiana

BORING #: **B-124**
JOB #: **170GC00939**

DRILLING and SAMPLING INFORMATION

Date Started	**1/12/20**	Hammer Wt.	**140** lbs.
Date Completed	**1/12/20**	Hammer Drop	**30** in.
Drill Foreman	**C. Carroll**	Spoon Sampler OD	**2.0** in.
Inspector	**D. McIlwaine**	Rock Core Dia.	**--** in.
Boring Method	**HSA**	Shelby Tube OD	**--** in.

TEST DATA

SURFACE ELEVATION 722

SOIL CLASSIFICATION	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
4.5 in. Concrete over 8 in. Aggregate Base	721.0	1.0								Ground surface elevation estimated from topographic map provided by client (survey performed by the Schneider Corporation, dated August 14, 2019).
Dark brown and black, moist, silty clay with little sand, trace gravel, and cinders (FILL)	718.5	3.5		1	SS		2-1-2	25.2		
Dark brown, moist, medium stiff, SILTY CLAY (CL) with some sand and trace gravel	715.0	7.0	5	2	SS		6-4-4	22.0	2.25	
				3	SS		1-2-5			
Brown, moist, loose, SAND (SW-SM) with little gravel and silt	713.5	8.5								
Brown, slightly moist, medium dense to very dense, SAND (SW) with little to some gravel	689.0	33.0	10	4	SS		11-12-16			
				5	SS		13-35-28			
			15	6	SS		8-7-9			
				7	SS		15-17-18			
			20	8	SS		10-12-17			
				9	SS		35-21-19			Boring backfilled with bentonite grout by tremie as augers were withdrawn from the boring.
			25	10	SS		21-30-25			
				11	SS		12-13-18			Boring patched with concrete at the surface.
			30	12	SS		12-12-10			
Gray, slightly moist, dense, SAND (SW-SM) with little gravel and silt	685.0	37.0	35	13	SS		10-16-26			
Brown, wet, medium dense, SAND (SW-SM) with some gravel and little silt				14	SS	●	16-11-14			

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
● Noted on Drilling Tools **37.0** ft.
▽ At Completion **--** ft.
▼ After **--** hours **--** ft.
Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

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TEST BORING LOG

CLIENT	**Indiana University Health**	BORING #	**B-124**
PROJECT NAME	**Proposed IU Health Academic Health Center**	JOB #	**170GC00939**
PROJECT LOCATION	**West 16th Street and North Senate Avenue**		
	Indianapolis, Indiana		

DRILLING and SAMPLING INFORMATION

Date Started	**1/12/20**	Hammer Wt.	**140** lbs.
Date Completed	**1/12/20**	Hammer Drop	**30** in.
Drill Foreman	**C. Carroll**	Spoon Sampler OD	**2.0** in.
Inspector	**D. McIlwaine**	Rock Core Dia.	**--** in.
Boring Method	**HSA**	Shelby Tube OD	**--** in.

TEST DATA

SOIL CLASSIFICATION (continued)	Stratum Elevation	Stratum Depth, ft	Depth Scale, ft	Sample No.	Sample Type	Sampler Graphics Recovery Graphics	Groundwater	Standard Penetration Test, Blows per 6 in. Increments	Moisture Content, %	Pocket Penetrometer PP-tsf	Remarks
Brown, wet, medium dense, SAND (SW-SM) with some gravel and little silt				15	SS			15-14-16			
			45								
	672.0	50.0		16	SS			13-13-15			
			50								
Bottom of Test Boring at 50.0 ft											

Sample Type
SS - Driven Split Spoon
ST - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core
CU - Cuttings
CT - Continuous Tube

Depth to Groundwater
Noted on Drilling Tools **37.0** ft.
At Completion **--** ft.
After **--** hours **--** ft.
Cave Depth **--** ft.

Boring Method
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
CA - Casing Advancer
MD - Mud Drilling
HA - Hand Auger

FIELD CLASSIFICATION SYSTEM FOR SOIL EXPLORATION

NON-COHESIVE SOILS

(Silt, Sand, Gravel and Combinations)

Density		SPT*
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 Proportions

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

Particle Size Identification

Boulders	-	8 inch or greater	
Cobbles	-	3 to 8 inch	
Gravel	-	Coarse	- 1 to 3 inch
		Medium	- ½ to 1 inch
		Fine	- ¼ to ½ inch
Sand	-	Coarse	2.00mm to ¼ inch (dia. of pencil lead)
		Medium	0.42 to 2.00mm (dia. of broom straw)
		Fine	0.074 to 0.42mm (dia. of human hair)
Silt			0.074 to 0.002mm (cannot see particles)

COHESIVE SOILS

(Clay, Silt and Combinations)

Consistency		SPT*
Very Soft	-	3 blows/ft or less
Soft	-	4 to 5 blows/ft
Medium Stiff	-	6 to 10 blows/ft
Stiff	-	11 to 15 blows/ft
Very Stiff	-	16 to 30 blows/ft
Hard	-	31 blows/ft or more

Plasticity

Degree of Plasticity	Plasticity Index
None to slight	0 - 4
Slight	5 - 7
Medium	8 - 22
High to Very High	over 22

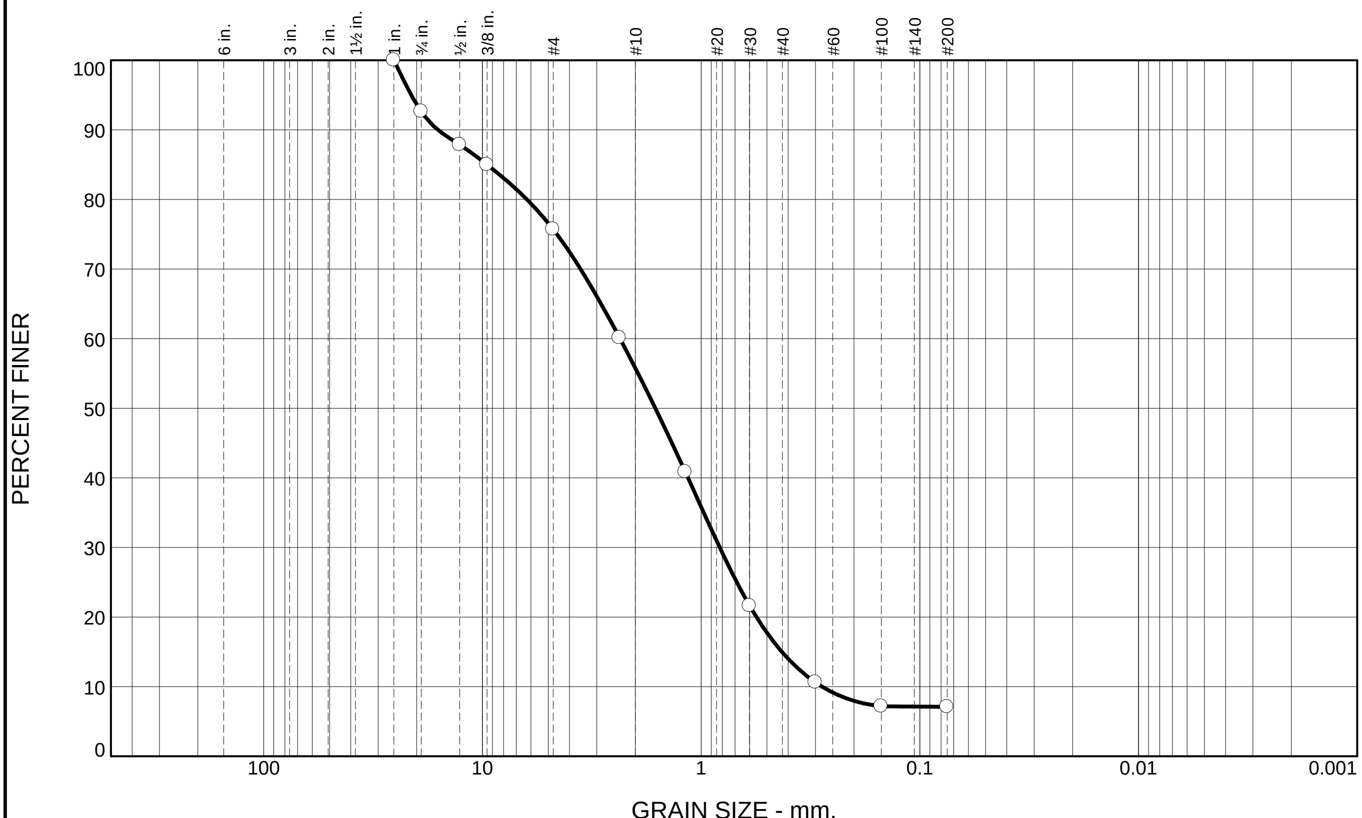
Classification on the logs are made by visual inspection of samples.
*Based upon results of Standard Penetration Test as described below.

Standard Penetration Test — Driving a 2.0" O.D. 1-3/8" I.D. sampler a distance of 12 inches into undisturbed soil with a 140 pound hammer free falling a distance of 30 inches. It is customary for ATC to drive the split-barrel sampler 6 inches to seat into undisturbed soil, then perform the test. The number of hammer blows for seating the split-barrel sampler and making the test are recorded for each 6 inches of penetration of the sampler (Example – 6-8-9). The standard penetration test result can be obtained by adding the last two figures (i.e., 8 + 9 = 17 blows/ft). The Standard Penetration Test is performed according to ASTM D-1586-18.

Strata Changes — In the column "Soil Classifications" on the Test Boring Logs the horizontal lines represent strata changes. A solid line (______) represents an actually observed change. A dashed line (_ _ _ _ _ _) represents an estimated change.

Ground Water observations were made at the times and conditions indicated on the Test Boring Logs. Porosity of soil strata, weather conditions, site topography, etc., may cause changes in the water levels indicated on the logs.

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	7.4	16.9	19.9	40.9	7.8	7.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1	100.0		
3/4	92.6		
1/2	87.9		
3/8	85.0		
#4	75.7		
#8	60.1		
#16	40.8		
#30	21.6		
#50	10.6		
#100	7.2		
#200	7.1		

* (no specification provided)

Material Description

Sand with some Gravel and trace Silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 15.9775 D_{85}= 9.5278 D_{60}= 2.3480
D_{50}= 1.6192 D_{30}= 0.8220 D_{15}= 0.4271
D_{10}= 0.2798 C_u= 8.39 C_c= 1.03

Classification

USCS= SW-SM AASHTO=

Remarks

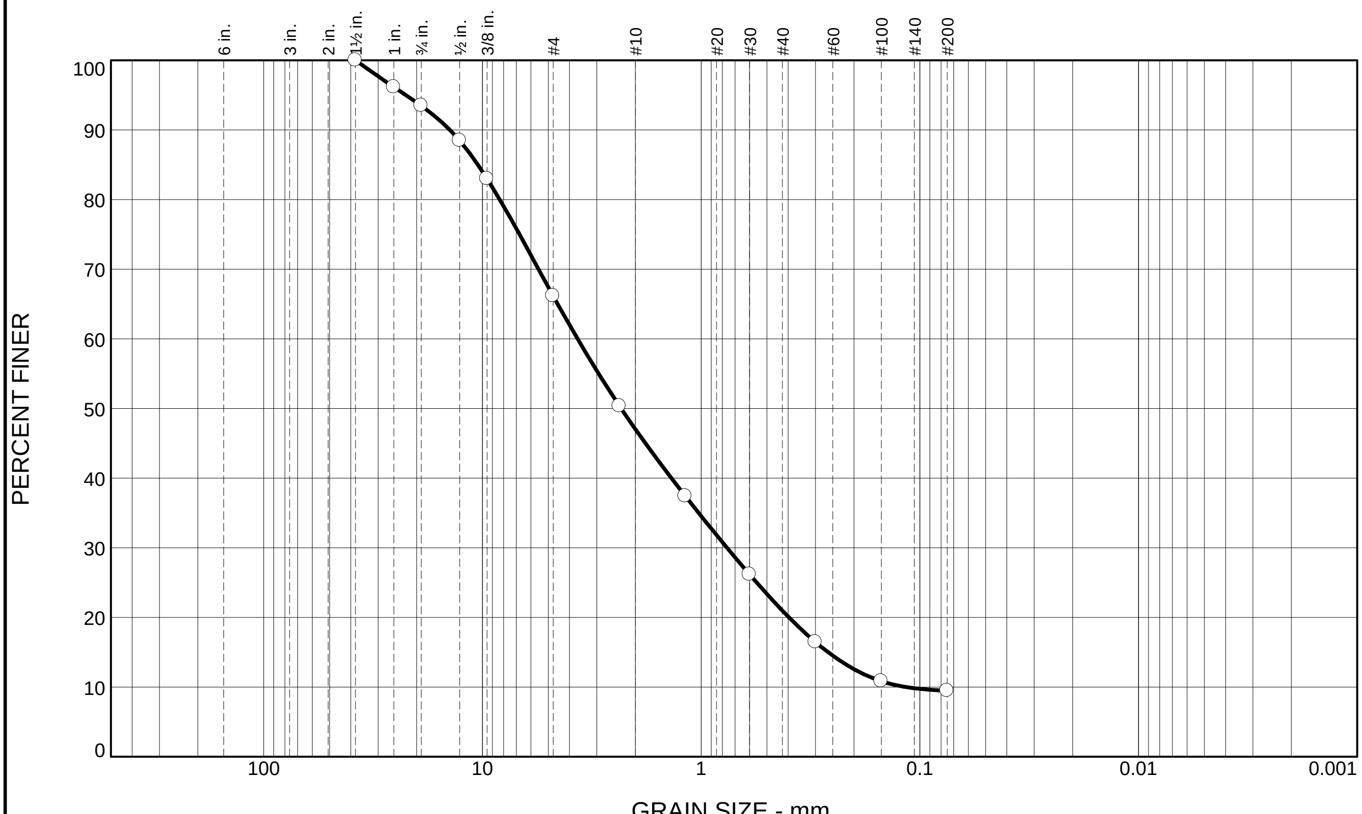
Source of Sample: 14373 **Depth:** 11.0'-15.0'
Sample Number: B-401; S-5 & 6 **Date:**

Atlas

Indianapolis, Indiana

Client: IUH
Project: IUH CUP
Project No: 170GC01425 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	6.5	27.3	19.2	26.0	11.5	9.5	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/2	100.0		
1	96.2		
3/4	93.5		
1/2	88.5		
3/8	83.0		
#4	66.2		
#8	50.3		
#16	37.4		
#30	26.2		
#50	16.5		
#100	10.8		
#200	9.5		

* (no specification provided)

Material Description

Sand with some Gravel and trace Silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 14.0876 D_{85}= 10.4945 D_{60}= 3.6700
D_{50}= 2.3204 D_{30}= 0.7610 D_{15}= 0.2623
D_{10}= 0.1155 C_u= 31.78 C_c= 1.37

Classification

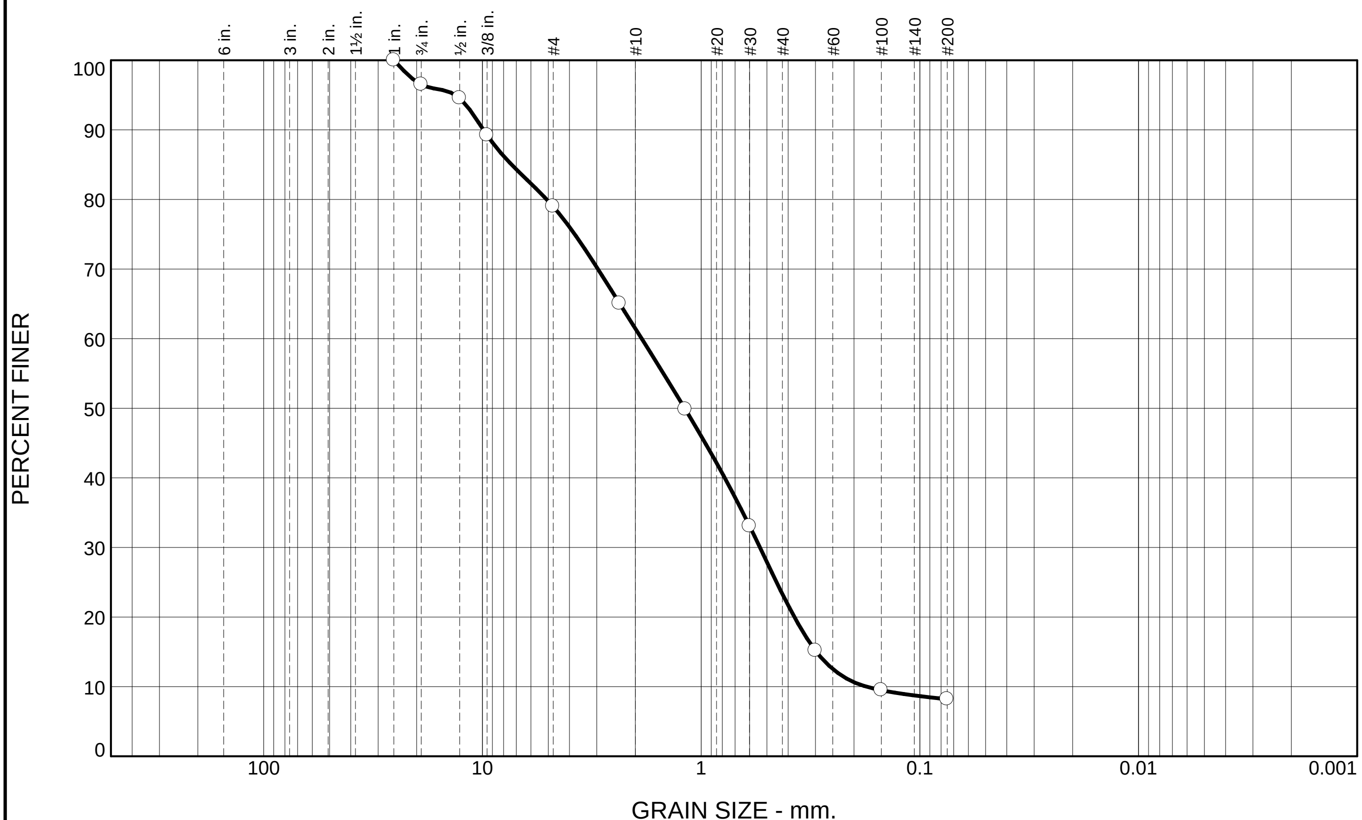
USCS= SW-SM AASHTO=

Remarks

Source of Sample: 14373 **Depth:** 21.0'-30.0'
Sample Number: B-401; S-9 thru 12 **Date:**

Atlas **Indianapolis, Indiana**	**Client:** IUH **Project:** IUH CUP **Project No:** 170GC01425 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	3.5	17.5	17.5	38.2	15.1	8.2	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1	100.0		
3/4	96.5		
1/2	94.5		
3/8	89.2		
#4	79.0		
#8	65.1		
#16	49.8		
#30	33.1		
#50	15.2		
#100	9.5		
#200	8.2		

* (no specification provided)

Material Description

Sand with some Gravel and trace Silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 9.8932 D_{85}= 7.3524 D_{60}= 1.8699
D_{50}= 1.1883 D_{30}= 0.5382 D_{15}= 0.2969
D_{10}= 0.1751 C_u= 10.68 C_c= 0.88

Classification

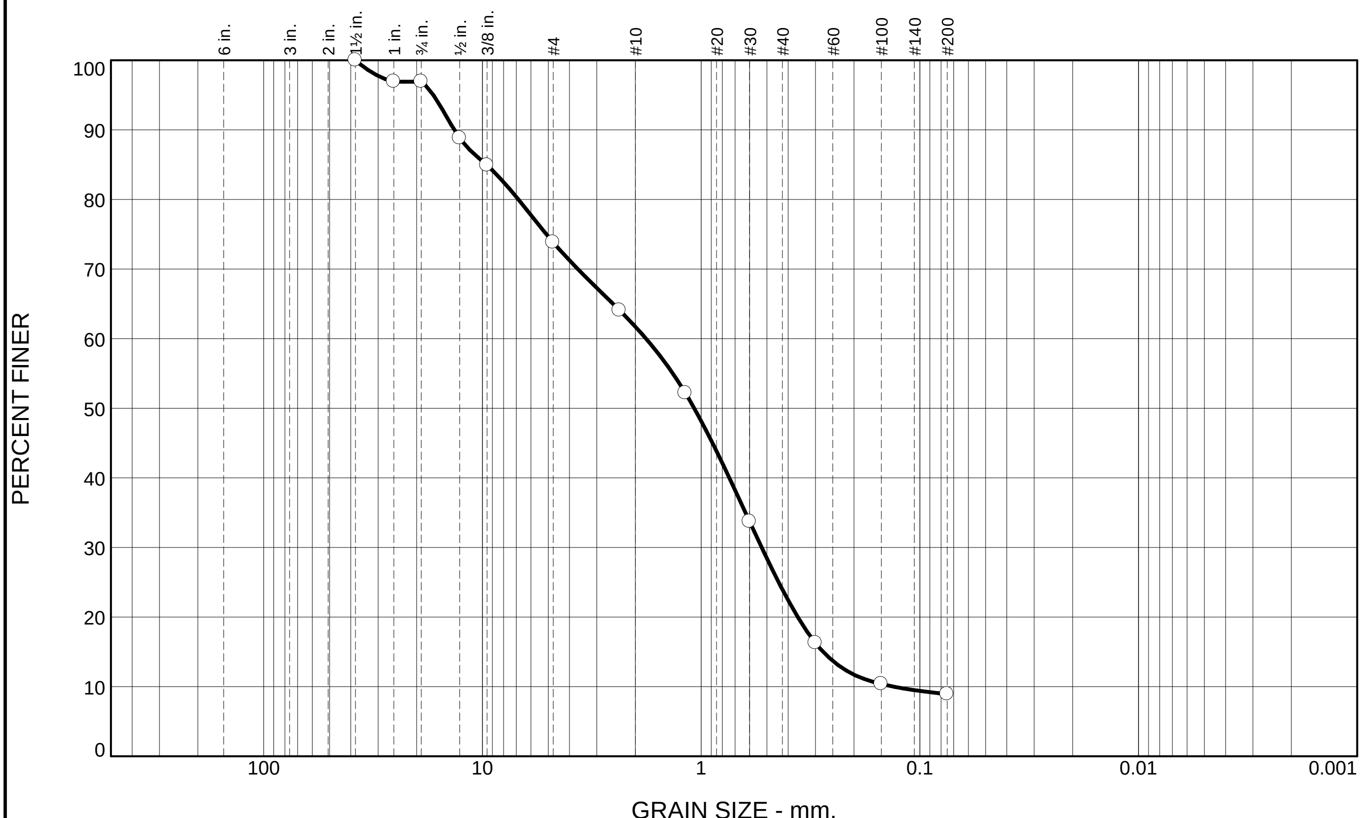
USCS= SP-SM AASHTO=

Remarks

Source of Sample: 14373 **Depth:** 38.5'-45.0'
Sample Number: B-401; S-16 & 17 **Date:**

Atlas **Indianapolis, Indiana**	**Client:** IUH **Project:** IUH CUP **Project No:** 170GC01425 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	3.1	23.0	12.2	37.7	15.1	8.9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/2	100.0		
1	96.9		
3/4	96.9		
1/2	88.8		
3/8	84.9		
#4	73.9		
#8	64.1		
#16	52.2		
#30	33.7		
#50	16.3		
#100	10.4		
#200	8.9		

* (no specification provided)

Material Description

Sand with some Gravel and trace Silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 13.4546 D_{85}= 9.5883 D_{60}= 1.7857
D_{50}= 1.0750 D_{30}= 0.5284 D_{15}= 0.2766
D_{10}= 0.1313 C_u= 13.60 C_c= 1.19

Classification

USCS= SW-SM AASHTO=

Remarks

Source of Sample: 14379 **Depth:** 21.0'-27.5'
Sample Number: B-404; S-9 thru 11 **Date:**

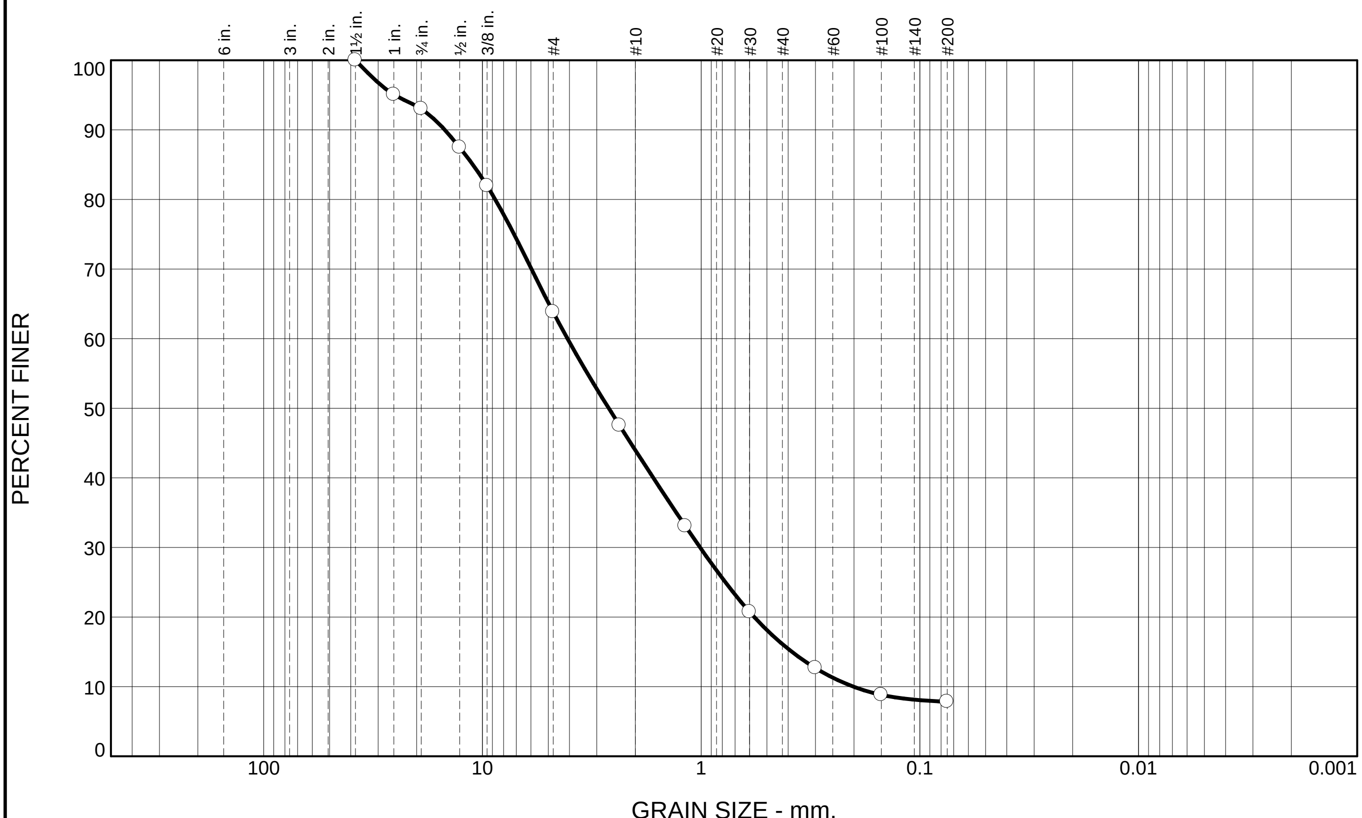
Atlas

Indianapolis, Indiana

Client: IUH
Project: IUH CUP

Project No: 170GC01425 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	7.0	29.2	19.8	27.9	8.3	7.8	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/2	100.0		
1	95.1		
3/4	93.0		
1/2	87.5		
3/8	82.0		
#4	63.8		
#8	47.5		
#16	33.1		
#30	20.7		
#50	12.7		
#100	8.8		
#200	7.8		

* (no specification provided)

Material Description

Sand and Gravel with trace Silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 14.8674 D_{85}= 11.0665 D_{60}= 4.0838
D_{50}= 2.6443 D_{30}= 1.0103 D_{15}= 0.3841
D_{10}= 0.2007 C_u= 20.35 C_c= 1.25

Classification

USCS= SW-SM AASHTO=

Remarks

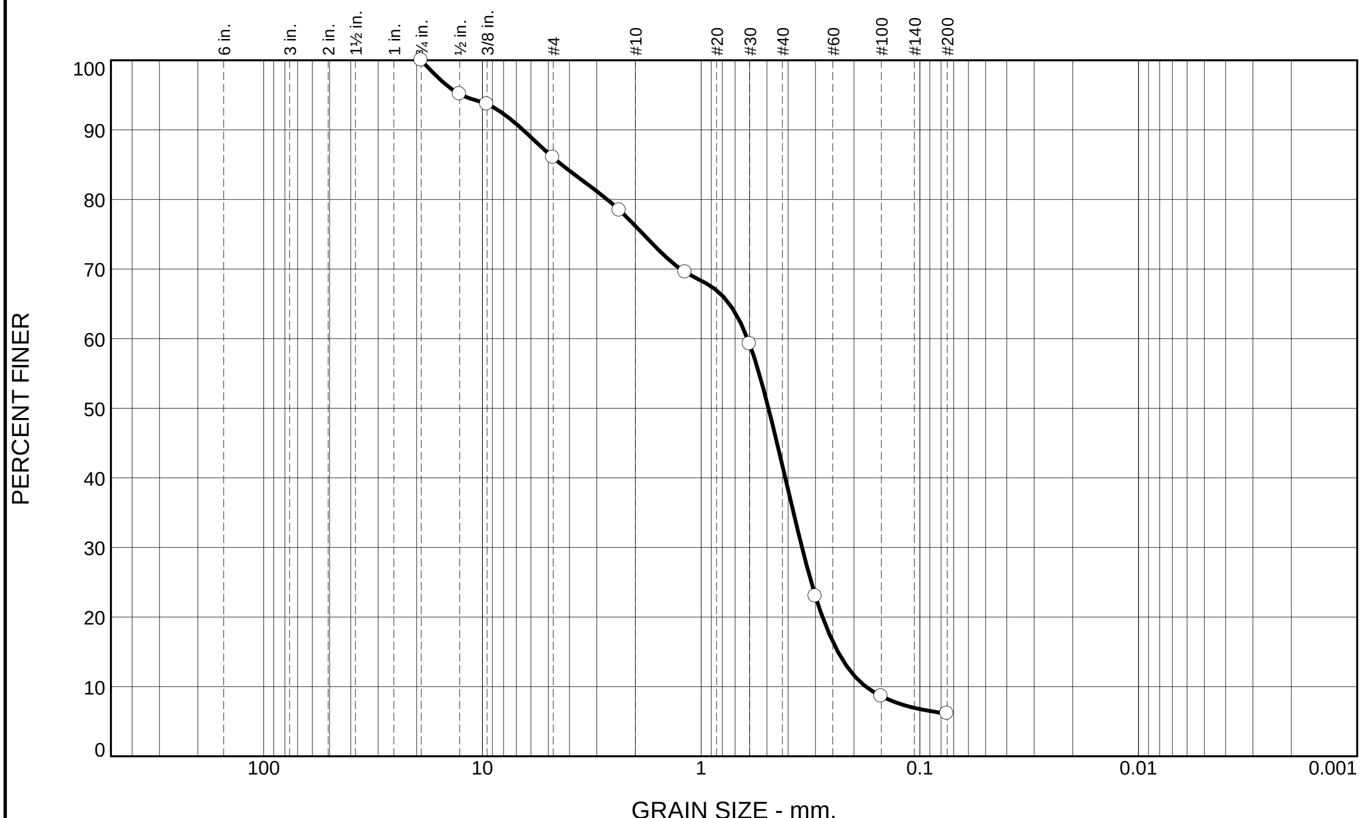
Source of Sample: 14379 **Depth:** 36.0'-40.0'
Sample Number: B-404; S-15 & 16 **Date:**

Atlas

Indianapolis, Indiana

Client: IUH
Project: IUH CUP
Project No: 170GC01425 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	14.0	9.8	34.4	35.7	6.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4	100.0		
1/2	95.1		
3/8	93.7		
#4	86.0		
#8	78.4		
#16	69.6		
#30	59.2		
#50	23.0		
#100	8.6		
#200	6.1		

* (no specification provided)

Material Description

Sand with little Gravel and trace Silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 6.5264 D_{85}= 4.3395 D_{60}= 0.6130
D_{50}= 0.4919 D_{30}= 0.3458 D_{15}= 0.2368
D_{10}= 0.1763 C_u= 3.48 C_c= 1.11

Classification

USCS= SP-SM AASHTO=

Remarks

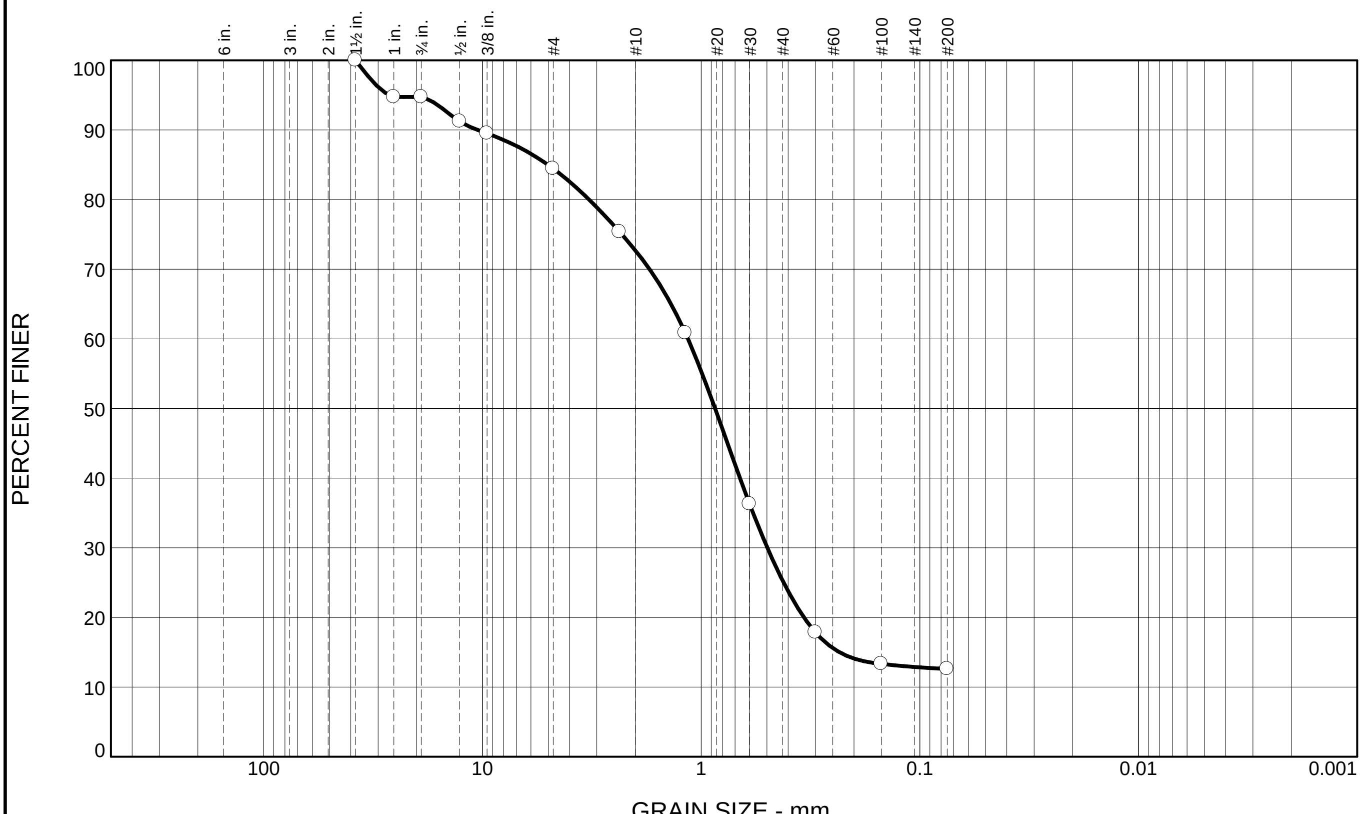
Source of Sample: 14373 **Depth:** 13.5'-15.0'
Sample Number: B-405; S-6 **Date:**

Atlas

Indianapolis, Indiana

Client: IUH
Project: IUH CUP
Project No: 170GC01425 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	5.3	10.2	11.8	47.3	12.8	12.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 1/2	100.0		
1	94.7		
3/4	94.7		
1/2	91.2		
3/8	89.5		
#4	84.5		
#8	75.4		
#16	60.8		
#30	36.3		
#50	17.9		
#100	13.3		
#200	12.6		

* (no specification provided)

Material Description

Silty Sand with little Gravel

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 10.5207 D_{85}= 5.0183 D_{60}= 1.1481
D_{50}= 0.8633 D_{30}= 0.4980 D_{15}= 0.2328
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

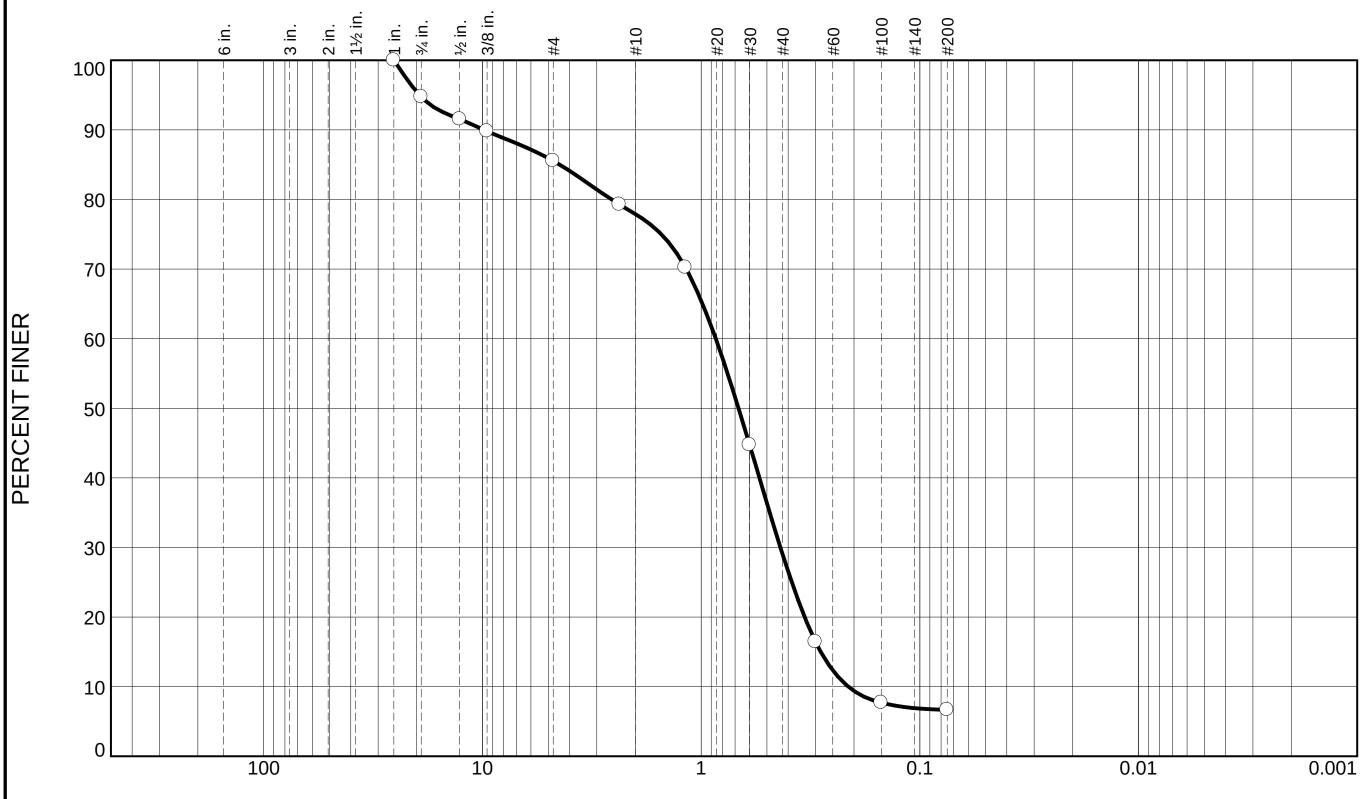
Source of Sample: 14379 **Depth:** 13.5'-17.5'
Sample Number: B-407; S-6 & 7 **Date:**

Atlas

Indianapolis, Indiana

Client: IUH
Project: IUH CUP
Project No: 170GC01425 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	5.3	9.1	7.7	48.8	22.4	6.7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1	100.0		
3/4	94.7		
1/2	91.5		
3/8	89.8		
#4	85.6		
#8	79.3		
#16	70.2		
#30	44.7		
#50	16.5		
#100	7.7		
#200	6.7		

* (no specification provided)

Material Description

Sand with little Gravel and trace Silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 9.8312 D_{85}= 4.4338 D_{60}= 0.8572
D_{50}= 0.6743 D_{30}= 0.4339 D_{15}= 0.2837
D_{10}= 0.2124 C_u= 4.04 C_c= 1.03

Classification

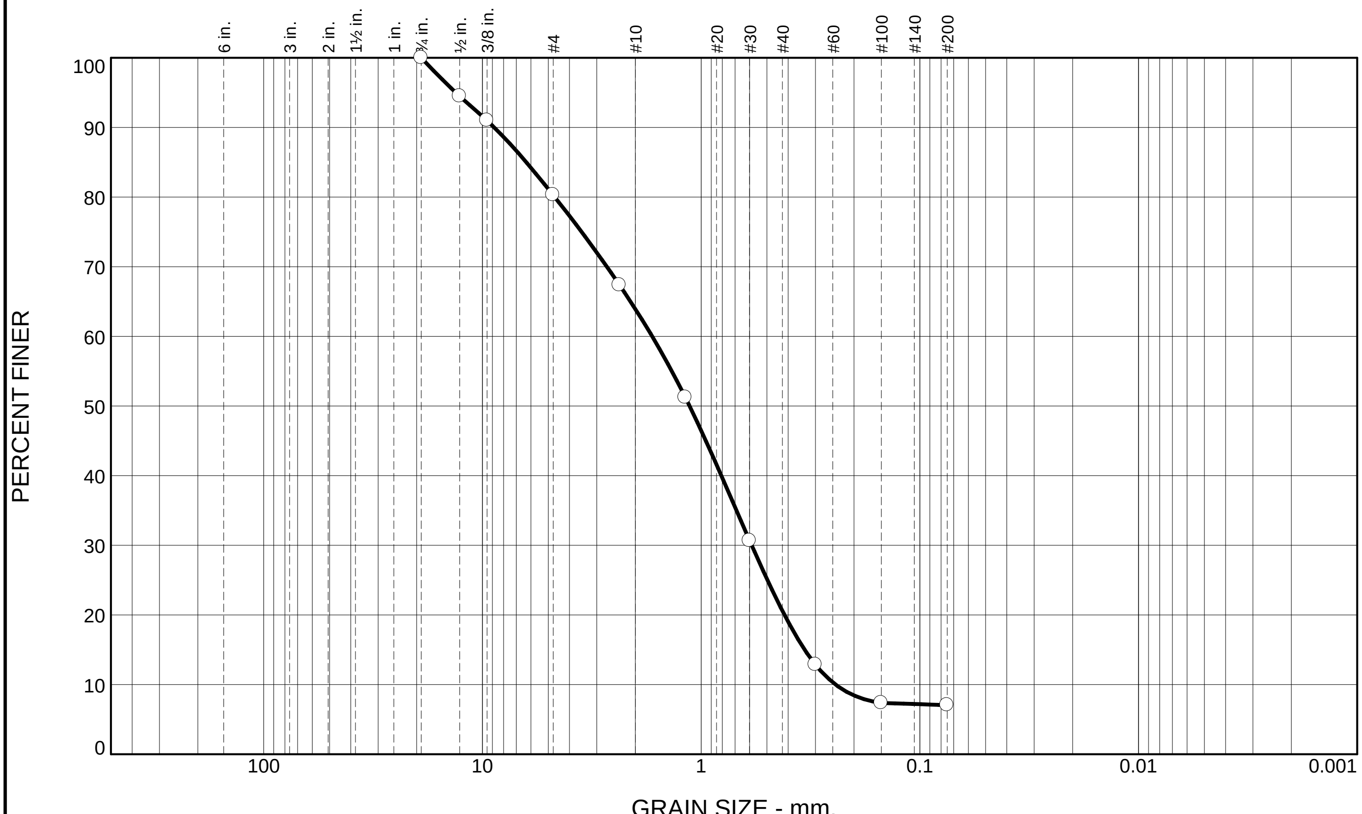
USCS= SP-SM AASHTO=

Remarks

Source of Sample: 14373 **Depth:** 6.0'-15.0'
Sample Number: B-408; S-3 thru 6 **Date:**

Atlas **Indianapolis, Indiana**	**Client:** IUH **Project:** IUH CUP **Project No:** 170GC01425 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	19.7	16.4	43.3	13.5	7.1	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4	100.0		
1/2	94.5		
3/8	91.0		
#4	80.3		
#8	67.4		
#16	51.2		
#30	30.7		
#50	12.9		
#100	7.4		
#200	7.1		

* (no specification provided)

Material Description

Sand with little Gravel and trace Silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 8.8316 D_{85}= 6.3011 D_{60}= 1.6737
D_{50}= 1.1291 D_{30}= 0.5871 D_{15}= 0.3361
D_{10}= 0.2427 C_u= 6.90 C_c= 0.85

Classification

USCS= SP-SM AASHTO=

Remarks

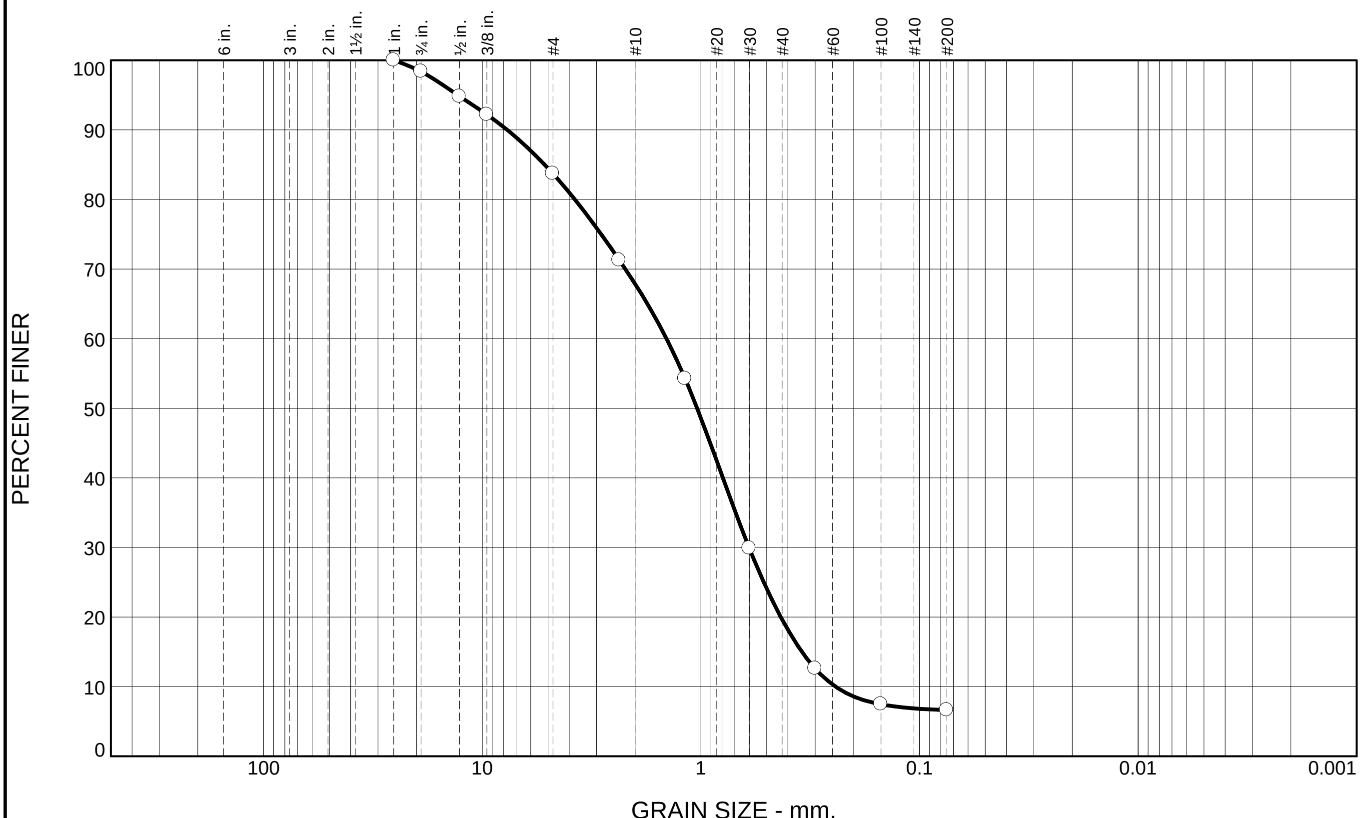
Source of Sample: 14373 **Depth:** 13.5'-17.5'
Sample Number: B-409; S-6 & 7 **Date:**

Atlas

Indianapolis, Indiana

Client: IUH
Project: IUH CUP
Project No: 170GC01425 **Figure**

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	1.6	14.7	15.8	48.2	13.1	6.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1	100.0		
3/4	98.4		
1/2	94.8		
3/8	92.2		
#4	83.7		
#8	71.3		
#16	54.2		
#30	29.9		
#50	12.6		
#100	7.5		
#200	6.6		

* (no specification provided)

Material Description

Sand with little Gravel and trace Silt

Atterberg Limits

PL= LL= PI=

Coefficients

D_{90}= 7.6928 D_{85}= 5.1850 D_{60}= 1.4360
D_{50}= 1.0404 D_{30}= 0.6021 D_{15}= 0.3448
D_{10}= 0.2419 C_u= 5.94 C_c= 1.04

Classification

USCS= SP-SM AASHTO=

Remarks

Source of Sample: 14373 **Depth:** 3.5'-10.0'
Sample Number: B-410; S-2 thru 4 **Date:**

Atlas

Indianapolis, Indiana

Client: IUH
Project: IUH CUP

Project No: 170GC01425 **Figure**

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

ATLAS Technical Consultants LLC

7988 Centerpoint Drive
Indianapolis, IN 46256

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September 29, 2022

Mr. Brent Bohan
Project Director
Design & Construction
Indiana University Health
950 North Meridian Street, Suite 1100
Indianapolis, IN 46204

Re: Addendum No. 1 to Geotechnical Engineering Investigation
Proposed IU Health Central Utility Plant
IU Health Academic Health Center of the Future Campus
West 13th Street and North Senate Avenue
Indianapolis, Indiana
Atlas Project No. 170GC01425

Dear Mr. Bohan:

Submitted herewith is Addendum No. 1 to the report for the geotechnical engineering investigation performed by Atlas Technical Consultants for the referenced project. Refer to the report dated September 15, 2022 for Atlas Project No. 170GC01425 for additional information regarding this project.

A series of corrosivity indicator tests was performed on selected soil samples from the test borings that were drilled for this project. The series of corrosivity indicator tests include water-soluble sulfate content tests, water-soluble chloride content tests, laboratory electrical resistivity tests and pH tests. The laboratory test results for the corrosivity indicator tests are included in the following table.

Summary of Corrosivity Indicator Test Results

Boring No.	Sample Depth Range, ft	Water-Soluble Sulfate Content, ppm	Water-Soluble Chloride Content, ppm	Laboratory Electrical Resistivity, ρ (ohm-cm)	pH
B-401	25.0 – 30.0	13	105	3,000	8.6
B-408	5.0 – 10.0	10	119	2,600	9.1

We appreciate the opportunity to be of continued service to you on this project. If you have any questions regarding the content of this letter, please do not hesitate to contact either of the undersigned.

Sincerely,

Thomas J. Struewing, P.E.
Principal Engineer

W 13th St
W 13th St
B-1
B-2
B-3
Brooker St
B-4
B-5
B-6
N Senate Ave
B-7
Smith Street
B-8
B-9
B-11
B-10
W 12th St
W 12th St
65
Google Earth
N
200 ft