



GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED BUILDING
RICHARD L. ROUDEBUSH VA MEDICAL CENTER
INDIANAPOLIS, INDIANA

CARDNO ATC PROJECT NO. 86.00481.0321

OCTOBER 24, 2014

PREPARED FOR:

AMERICAN STRUCTUREPOINT, INC.
7260 SHADELAND STATION
INDIANAPOLIS, INDIANA 46256-3957

ATTENTION: MR. SHAUN COFER, P.E., LEED AP

October 24, 2014

Mr. Shaun Cofer, P.E., LEED AP
American Structurepoint, Inc.
7260 Shadeland Station
Indianapolis, Indiana 46256-3957

Re: **Geotechnical Engineering Investigation**
Proposed Building
Richard L. Roudebush VA Medical Center
Indianapolis, Indiana
Cardno ATC Project No. 86.00481.0321

Cardno ATC

7988 Centerpoint Dr.
Suite 100
Indianapolis, IN 46256

Phone +1 317 849 4990
Fax +1 317 849 4278
www.cardno.com

www.cardnoatc.com

Dear Mr. Cofer:

Submitted herewith is the report of our geotechnical engineering investigation for the referenced project. This study was authorized in accordance with Task Order No. 12 for 201000629 and the Subconsultant Agreement for Professional Services dated April 23, 2013.

This report contains the results of our 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 30 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,



David McIlwaine, P.E.
Project Engineer
for Cardno ATC



Thomas J. Struewing, P.E.
Senior Project Engineer
for Cardno ATC

Table of Contents

1	PURPOSE AND SCOPE	1
2	PROJECT CHARACTERISTICS	1
3	GENERAL SUBSURFACE CONDITIONS	2
3.1	Regional and Site Geology	2
3.2	Subsurface Soil Conditions	3
3.3	Ground Water	3
4	DESIGN RECOMMENDATIONS	4
4.1	General Foundation Concepts	4
4.2	Auger-Cast Concrete Piles.....	4
4.3	Floor Slabs	6
5	GENERAL CONSTRUCTION PROCEDURES AND RECOMMENDATIONS	7
5.1	Auger-Cast Concrete Pile Installation Observations.....	7
5.2	Site Preparation	7
5.3	Fill Compaction	8
5.4	Construction Dewatering.....	8
6	FIELD INVESTIGATION	9
7	LABORATORY INVESTIGATION	9
8	LIMITATIONS OF STUDY	9

Appendix

1 PURPOSE AND SCOPE

The purpose of this study was to determine the general subsurface conditions at the project site by drilling five soil test borings and to evaluate this data with respect to foundation concept and design for the proposed building. Also included is an evaluation of the site with respect to potential construction problems and recommendations dealing with earthwork and quality control during construction.

2 PROJECT CHARACTERISTICS

American Structurepoint, Inc. is preparing plans for the construction of a new building on the west side of the Richard L. Roudebush VA Medical Center in Indianapolis, Indiana. The general location of the project site is shown on the Vicinity Map (Figure 1 in the Appendix), which is taken from a map made prior to the current level of development in the surrounding area. The ground surface within the proposed building area is relatively flat, ranging from about El 703 on the west side to El 705 on the east side. The majority of the ground surface in the proposed building area is paved with asphalt or concrete, except for the east area, which is grass covered. Based on the urban location of the site and available aerial photos, much of the project site appears to have been occupied by structures that have been previously demolished. The new building will be located immediately adjacent to existing Building No. 5, which has a basement level.

It is our understanding that the proposed building will initially be a two-story structure with the option to increase the height to four-stories at a later time. The proposed building will have a slab-on-grade ground floor, with the finish floor at El 704.8 and approximate plan dimensions of 130 ft by 75 ft. Thus, less than about 2 ft of grade raise fill will be required to establish the finish ground floor elevation. The proposed location of the building on the site is shown on the Boring Plan (Figure 2 in the Appendix).

Details regarding structural loads are not available at this time; however, for the purpose of this study it has been assumed that the maximum column, wall and floor loads for the proposed building will not exceed about 500 kips/column, 5 kips/lin.ft and 100 lbs/sq.ft, respectively. No unusual loading conditions or settlement restrictions have been specified.

3 GENERAL SUBSURFACE CONDITIONS

The general subsurface conditions were investigated by drilling five test borings to depths ranging from 40 to 60 ft at the approximate locations shown on the Boring Plan (Figure 2 in the Appendix). The subsurface conditions disclosed by the field investigation are summarized in the following paragraphs. 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. 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. Natural Indianapolis surface features result from the most recent glaciation (i.e., Wisconsin 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, major valleys (e.g., those of White River and Fall Creek) were formed by meltwater streams during glacial recession. Outwash deposits within these meltwater valleys, which generally coincide with current stream channels but are much wider, are predominately granular soils consisting of sands and gravels, sometimes overlying cobbles and boulders.

The project site is located on a terrace above the White River flood plain. The site is about 500 ft east of White River. The majority of the natural unconsolidated deposits in the immediate vicinity of the site consist of outwash sand and gravel that was deposited by glacial meltwaters. The outwash is often covered by a thin layer of alluvium and interrupted by layers of glacial till that vary in thickness and appear to be somewhat random in occurrence. Geologic mapping indicates that the upper bedrock in this area is limestone that was deposited on the order of 400,000 years ago during the Devonian Age. Published geologic mapping indicates that the bedrock surface underlying downtown Indianapolis varies from about El 590 to El 640. 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 Indianapolis ground shaking 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. No significant earthquake activity is expected from any of the other faults located in Indiana. There is virtually no potential for "liquefaction" (a phenomenon whereby ground shaking causes a severe loss of soil strength) for the soil conditions at the project site under any reasonably anticipated ground shaking.

3.2 Subsurface Soil Conditions

Borings B-1 through B-4 revealed 0.4 ft of asphalt pavement at the surface and Boring B-5 revealed 0.3 ft of topsoil. Underlying the surface materials, the test borings encountered sandy silty clay, silty clay and clayey sand fill containing various amounts of gravel, roots and cinders to depths of 8 to 11 ft below the existing ground surface. Then, either very soft to medium stiff silty clay (CL) or sandy silty clay (CL) and very loose to loose sand (SP, SP-SM) or silty sand (SM) was revealed to depths ranging from 21 to 23 ft. Underlying these materials, generally medium dense to dense sand (SP, SP-SM, SP-SC) was encountered to the termination depths of 40 to 60 ft. Borings B-1, B-2 and B-3 revealed medium stiff to hard silty clay (CL) or sandy silty clay (CL) at varying depths interbedded within the granular soils. The consistencies of the cohesive soils and densities of the granular soils as described above and on the boring logs were estimated based on the results of the standard penetration test (ASTM D-1586).

It is our experience that cobbles and boulders are often present within the glacial outwash soils such as those that underlie this site. Therefore, it is important to understand that although no boulders or cobbles were encountered in the test borings, cobbles and boulders may be encountered in excavations and during pile installation at this site. Furthermore, large obstructions are often encountered within the miscellaneous uncontrolled fill materials such as those encountered in the upper 8 to 11 ft of the test borings. There may also be buried obstructions such as basement floors, footings, walls, cisterns, wells, etc., at the site as is often the case in urban settings such as this and has been observed at nearby project sites.

3.3 Ground Water

Ground water observations were made during the drilling operations by noting the depth of water on the drilling tools, at completion of the test borings and 24-hours after drilling in Boring B-4. Free ground water was noted at depths ranging from 27.2 to 32.5 ft below the existing ground surface. Borings B-1 and B-3 revealed apparent perched ground water at depths of 9.5 ft and 13.6 ft, respectively. Short term ground water level readings made in granular glacial outwash soils are generally a 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 the nearby White River and other factors. Although a higher ground water level due to rare events or a combination of rare events (e.g., an extended period of heavy rainfall, cessation of pumping from a number of nearby wells and at normal flooding of White River) cannot be ruled out with complete certainty, it appears that a ground water level higher than El 690 (approximately 10 ft below the existing ground surface) is unlikely during the life of the structure.

4 DESIGN RECOMMENDATIONS

The following design recommendations have been developed on the basis of the previously described project characteristics (Section 2.0) and subsurface conditions (Section 3.0). If there is any change in these project criteria, including project location on the site, finish floor elevation, loading conditions, etc., a review should be made by this office.

4.1 General Foundation Concepts

The test borings drilled for this project revealed miscellaneous uncontrolled fill materials to depths of about 8 ft to 11 ft below the existing ground surface. The miscellaneous fill materials are underlain by strata of very soft to soft alluvial cohesive soils and very loose to loose alluvial sand and silty sand to depths ranging from approximately 21 ft to 23 ft below the existing ground surface. The miscellaneous uncontrolled fill materials do not have strength and compressibility characteristics that are reliable enough to support conventional spread footings at nominal depths without the risk of unacceptable settlement and the softer and looser alluvial soils are also not suitable for support of the heavier loads that will be imparted by the building without greater than normal settlement.

While the medium dense to dense natural glacial outwash sand and gravel encountered below depths of about 21 to 23 ft below the existing ground surface are suitable for support of spread footings, it would be necessary to remove substantial amounts of existing miscellaneous fill materials and natural alluvial soils to expose the medium dense glacial outwash soils that are suitable for bearing, and then to replace the unsuitable materials with well-compacted engineered fill in order to use spread footings. Therefore, it is recommended that auger-cast concrete pile foundations that extend well into the medium dense to dense glacial outwash soils be used to support the proposed building. Recommendations for auger-cast concrete piles are provided in Section 4.2.

Based on geologic mapping, the results of the test borings drilled for this project and the results of shear wave velocity testing performed on nearby sites, 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.

4.2 Auger-Cast Concrete Piles

Auger-cast concrete piles appear to represent the most economical and reliable foundation system for support of the proposed building. The following table summarizes allowable pile load capacities for 16 in. diameter auger-cast concrete piles installed to various pile tip elevations. Pile lengths that are greater than 30 times the pile diameter will require approval by the building official in accordance with the 2014 Indiana Building Code. It is recommended that at least one compression pile load test be performed (in accordance with ASTM D-1143-07 "Standard Test Methods for Deep Foundations Under Static Axial Compressive Load") prior to production pile installation for each pile tip elevation that will be used.

ALLOWABLE AUGER-CAST PILE CAPACITIES, tons/pile

Pile Tip Elevation	16 in. Diameter Auger-Cast Pile		
	Vertical Downward Capacity*	Vertical Uplift Capacity*	Lateral Capacity**
660	50	15	6
655	60	20	6
650	70	25	6

*Based upon bottom-of-pile cap at El 695 or higher.

**Analysis based upon the installation of a reinforcing steel cage consisting of at least 6 No. 6 bars that extend to a depth of 20 ft below the base of the pile cap, the top-of-pile extending a distance of at least 6 in. into the pile cap and the reinforcing bars extending into the pile cap a minimum of 30 in. above the top of the pile. The actual reinforcing steel must meet the minimum requirements of the 2014 Indiana Building Code (2012 International Building Code).

It is important to note that obstructions may be encountered when drilling through miscellaneous fill materials and remnants of previous construction, and that difficult drilling may be experienced when drilling through the stiff to hard glacial till layers noted at depths of about 48 ft and 27.5 ft in Borings B-1 and B-2, respectively. However, it is essential that the piles extend to the design pile tip elevations since the piles must bear in the dense glacial outwash sand and gravel soils to achieve the allowable capacities prescribed above and because a portion of the pile capacities are derived from shaft friction for a predetermined pile length. Properly installed piles as described above should experience vertical deflection less than about 1 in.

It is recommended that each pile be equipped with a center bar extending the full length of the pile, with the bar size dictated by the vertical uplift loads, or as required by the building code. In order to properly align the reinforcing steel within the auger-cast concrete piles and to aid the installation of the center bar and cage by preventing the center bar and cage from “stabbing” the sides of the pile excavation, it is recommended that centralizer devices be used. The outside diameter of the reinforcing cage should be at least 6 in. less than the pile diameter. All piles should be located at least 3 diameters (48 inches) apart center-to-center. The grout ratio (i.e., the ratio of grout pumped per pile divided by the theoretical volume of the pile) should be at least 1.25 for each 5 ft segment of the pile. The grout should have a compressive strength of 4,000 lbs/sq.in. and the reinforcing steel should have a yield strength of 60,000 lbs/sq.in.

Because of the presence of old miscellaneous fill noted in the upper stratigraphy at this site, it is possible that the installation of some of the auger-cast concrete piles may be obstructed. If this happens, it may be possible to remove the obstruction with a backhoe where the obstruction is relatively shallow. In other cases, it may be necessary to offset the pile and adjust the pile cap to accommodate the offset location. Furthermore, because of the miscellaneous nature of the fill at some locations, installation problems with “grout-take” may be experienced. This may include high grout ratios and settlement of the grout at the top of the piles after withdrawal of the augers. If the grout at the top of a pile settles after withdrawal of the augers, the pile should be redrilled and regouted, if possible, or replaced. In accordance with the 2014 Indiana Building Code (i.e., 2012 International Building Code), auger-cast concrete piles within 6 pile diameters of each other, as measured from center-to-center, should not be installed within less than 12 hours of each other. In the event that grout drops occur in adjacent piles during installation of a pile,

the piles that experience the grout drop must be replaced and adjustment to the delay criterion may be required.

During the advancement of the augers in areas near existing buildings, retaining walls, etc., it is critical that the contractor not "over-spin" the augers, or rotate the augers without corresponding advancement in the depth of the augers. Excessive rotation of the augers can result in a volume of granular soils being removed from around the auger-cast pile excavation that is significantly greater than the theoretical volume of the auger-cast pile, which can cause overmining of the adjacent soil that may impact the stability of the existing building. Therefore, the auger-cast pile contractor must be required to install the piles in a manner that does not adversely impact the existing adjacent building and the existing basement walls. It is recommended that only one pile per pile cap per day be installed at the pile cap locations immediately adjacent to the existing building. Displacement piles should also not be used in this case. We also recommended that the existing building be monitored regularly during the installation of the auger-cast concrete piles (by surveying pre-established points for both vertical and horizontal movement) to determine whether the structure is adversely affected by the installation of the piles.

All existing utilities, pavements, sidewalks, etc., should be suitably protected from undermining due to excavation for the new pile caps. Depending on the locations of the existing footings and the new pile caps, bracing or underpinning may be needed to protect the existing elements. All federal, state and local safety regulations should be followed in this regard.

4.3 Floor Slabs

It appears that it is possible to support the slab-on-grade floor on the existing soils 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 engineered fill. Based on the type of fill materials encountered in the test borings, in conjunction with the anticipated light floor slab loading, the cost of complete removal and replacement of the existing uncontrolled fill materials beneath the floor slab areas, or the support of the floor slab as a structural slab supported on auger-cast piles may not be justified in order to completely eliminate the relatively small risk of greater-than-normal floor slab settlement that could occur at some locations if the existing fill is 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. If this risk is unacceptable it is recommended that the floor slab be supported on auger-cast piles in a fashion similar to the building columns and walls.

It is recommended that all floor slabs be "floating", that is, fully ground supported and not structurally connected to walls or foundations. This is to minimize the possibility of cracking and displacement of the floor slabs because of differential movements between the slab and the foundation. Although the movements are estimated to be within the tolerable limits for structural safety, such movements could be detrimental to the slabs if they were rigidly connected to the foundations.

It is furthermore 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 stone. This is to help distribute concentrated loads and equalize moisture conditions beneath the slab. Provided that a minimum of 6 in. of granular material is placed below the slab, a modulus of subgrade reaction (k_{30}) of 150 lbs/cu.in. can be used for design of the floor slabs.

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 foundation recommendations become necessary. Therefore, we recommend that Cardno ATC be retained as geotechnical consultant through 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 Auger-Cast Concrete Pile Installation Observations

In order to document that the auger-cast concrete piles are properly installed, it is recommended that a representative of the geotechnical engineer who is independent of the contractor perform continuous observations during pile installation. An accurate record should be kept of the date, time, depth of penetration and other pertinent data for each pile. It should also be verified that proper grout pressure is maintained, that the ratio of grout pumped to theoretical volume of the pile is at least 1.25 (for each 5 ft segment of the pile) and that the piles are plumb. Because of the miscellaneous fill within the upper portions of the stratigraphy at this site, it is particularly important that the piles be observed after withdrawal of the augers to verify that there is no grout loss after installation (as described in Section 4.2).

5.2 Site Preparation

All areas that will support floor slabs should be properly prepared. After rough grade has been established in cut areas and prior to placement of fill in all fill areas, the exposed subgrade should be carefully observed by the geotechnical engineer, or a qualified soils technician working under the direction of the geotechnical engineer, by probing and testing as needed. Any topsoil or other organic material still in place, frozen, wet, soft or loose soil and other undesirable materials such as concentrations of rubble and debris, compressible or collapsible objects, degradable materials, etc. should be removed. The exposed subgrade should furthermore be evaluated by proofrolling with suitable equipment to check for pockets of soft material hidden 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.

Care must be exercised during the grading operations at the site. Due to the silty and clayey 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. It is important that positive surface drainage be established at the beginning of the earthwork operations and be maintained throughout the project. Surface water must not be allowed to pond. Furthermore, compaction and sealing of the subgrade surface is important when precipitation is expected. The site storm drainage elements (i.e., catch basins, pipes, manholes, etc.) should be installed as early as possible, which will aid in control of surface and ground water.

5.3 Fill Compaction

All engineered fill should be compacted to a dry density of at least 95 percent of the standard Proctor maximum dry density (ASTM D-698). The compaction 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 document moisture conditions and the actual compaction that is being achieved.

Compaction of any fill by flooding is not considered acceptable. This method will generally not achieve the desired compaction and the large quantities of water will tend to soften the foundation soils.

5.4 Construction Dewatering

No serious dewatering problems are anticipated unless it becomes necessary to excavate below the ground water level, which appeared to be at about El 675 to El 678 at the time the test borings were completed. However, depending on the seasonal conditions, some seepage into shallow excavations may be experienced due to "perched" water that may be encountered in sand seams or that may be encountered within old miscellaneous fill materials, abandoned utilities, utility trenches, etc. It is anticipated that any such seepage into shallow excavation will either percolate downward through the granular subsurface soils or can be handled by conventional dewatering methods such as by pumping from sumps. This method, however, will not be effective for any excavation that extends below the actual ground water level. In this case, it would be necessary to pump from wells or well points in order to depress the ground water level.

6 FIELD INVESTIGATION

Five test borings were drilled at the approximate locations shown on the Boring Plan (Figure 2 in the Appendix). The borings were extended to depths of 40 to 60 ft below the existing grade. Split-barrel samples were obtained by the Standard Penetration Test procedures (ASTM D-1586) at 2.5 to 5 ft intervals.

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

7 LABORATORY INVESTIGATION

The disturbed samples were inspected and classified in accordance with the Unified Soil Classification System and the boring logs were edited as necessary. To aid in classifying the soils and to determine general soil characteristics, natural moisture content tests and calibrated hand penetrometer ("pocket penetrometer") tests were performed on selected samples. The results of these tests are included on the Test Boring Logs in the Appendix.

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.

Our professional services have been performed, our findings obtained and our recommendations prepared in accordance with generally accepted geotechnical engineering principles and practices. 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.

Cardno ATC assumes no responsibility for any construction procedures, temporary excavations (including utility trenches), temporary dewatering or site safety during or after construction. The contractor will be solely responsible for all construction procedures, construction means and methods, construction sequencing and for safety measures during construction. 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 is 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.

Appendix

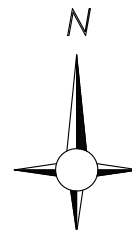
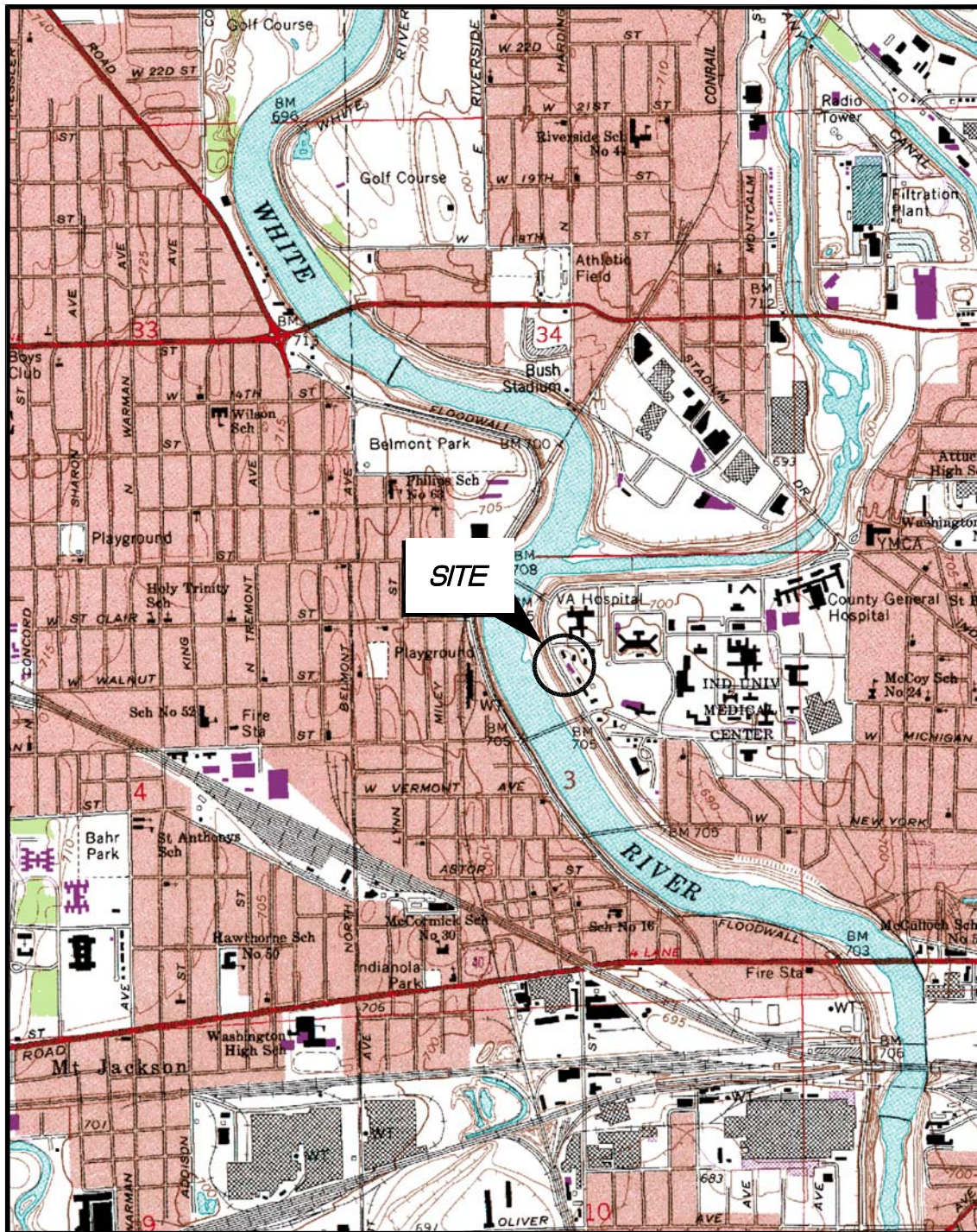
Figure 1: Vicinity Map

Figure 2: Boring Plan

Test Boring Logs (5)

“Field Classification System for Soil Exploration”

“Important Information About Your Geotechnical Engineering Report”



VICINITY MAP

PROPOSED BUILDING
RICHARD L. ROUDEBUSH VA MEDICAL CENTER
INDIANAPOLIS, INDIANA

Project Number:
86.00481.0321

Drawing File:
00481~321A

Date:
10/14

Scale:
1" = 2000'



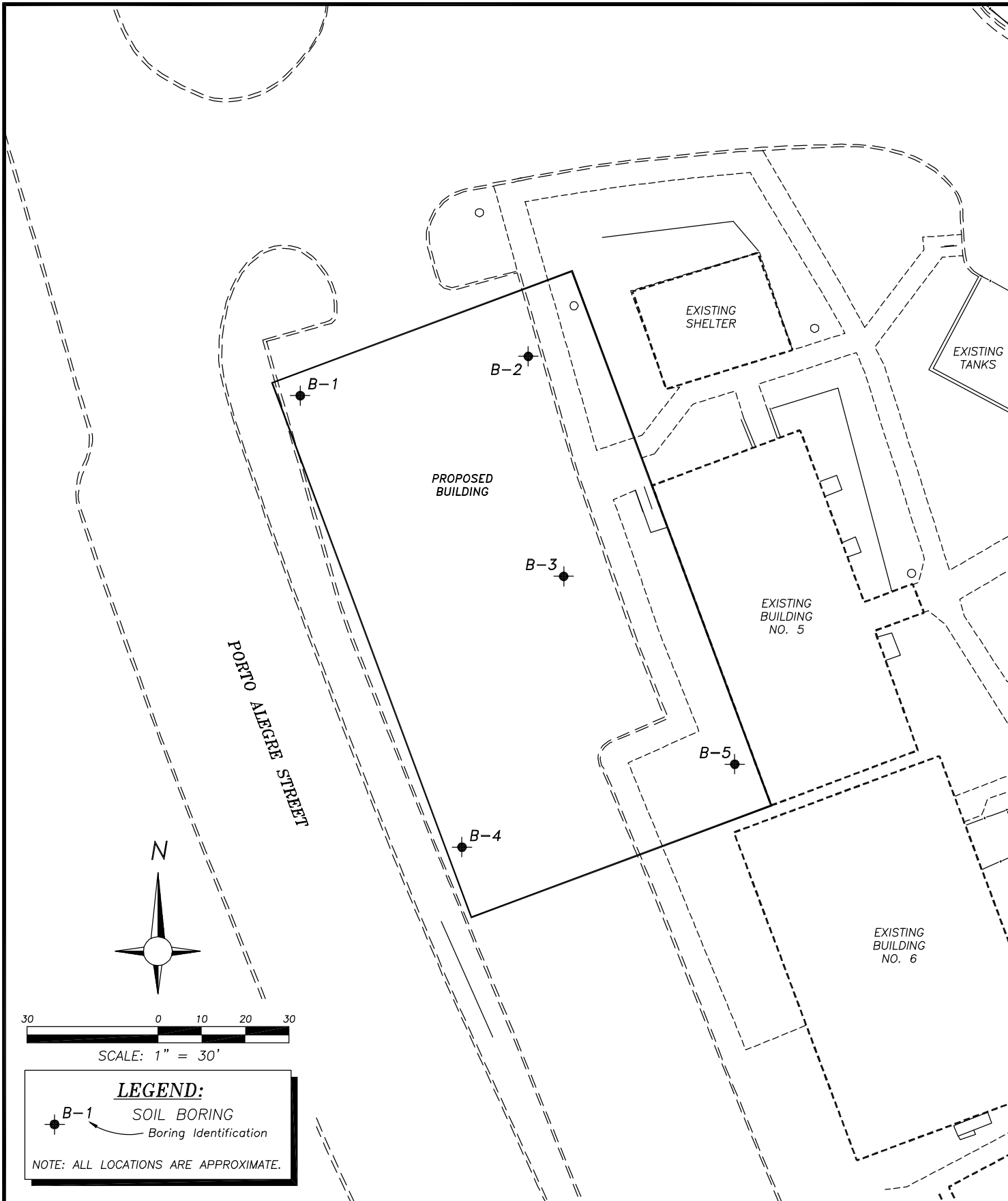
Drn. By:
SP

Ckd. By:
DM

App'd By:

Figure:

1



BORING PLAN

PROPOSED BUILDING
RICHARD L. ROUDEBUSH VA MEDICAL CENTER
INDIANAPOLIS, INDIANA

Project Number:
86.00481.0321

Drawing File:
00481~321A

Date:
10/14

Scale:
AS SHOWN



Drn. By:
SP

Ckd. By:
DM

App'd By:

Figure:

2

CLIENT American Structurepoint, Inc.
PROJECT NAME Proposed Building
PROJECT LOCATION Richard L. Roudebush VA Medical Center
Indianapolis, Indiana

BORING # B-1
JOB # 86.00481.0321

DRILLING and SAMPLING INFORMATION

TEST DATA

Date Started 10/13/14 Hammer Wt. 140 lbs.
Date Completed 10/13/14 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.

SOIL CLASSIFICATION	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
SURFACE ELEVATION 703.5												
15 in. Asphalt	703.1	0.4										Borehole cleared of underground utilities to a depth of 5 ft below the existing ground surface utilizing air-knife methods by Cardno TBE.
Blank drilled to 6.0 ft, no samples obtained												
	697.5	6.0	5									
Brown and black, moist sandy silty clay with little gravel and cinders (FILL)	695.5	8.0		1	SS				3-2-3			
Brown, moist clayey sand with little gravel (FILL)				2	SS				2-2-3			Ground surface elevation estimated from topographic map provided by client. No soil samples could be obtained in upper 5 ft.
	692.5	11.0	10									
Dark brown, moist, very soft SILTY CLAY (CL) with trace sand	690.5	13.0		3	SS				1-1-2	21.8	1.5	
Brown, moist, soft SILTY CLAY (CL) with some sand				4	SS				2-2-3	16.5	2.0	
	687.5	16.0	15									
Brown, moist, loose SILTY SAND (SM) with trace gravel				5	SS				3-3-4			
				6	SS				3-4-5			
	682.5	21.0	20									
Brown, slightly moist to moist, medium dense to dense SAND (SP)				7	SS				8-12-13			
				8	SS				9-12-21			
			25									
				9	SS				12-12-14			
			30									
	671.0	32.5										
Brown, wet, medium dense SAND (SP)				10	SS				12-6-14			
			35									
				11	SS				5-7-9			

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 32.5 ft.
▽ At Completion 9.5 ft.
▽ After -- hours -- ft.
⊗ Cave Depth 19.0 ft.









Boring Method

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

CLIENT **American Structurepoint, Inc.**
PROJECT NAME **Proposed Building**
PROJECT LOCATION **Richard L. Roudebush VA Medical Center**
Indianapolis, Indiana

BORING # **B-1**
JOB # **86.00481.0321**
DRILLING and SAMPLING INFORMATION
TEST DATA

Date Started **10/13/14** Hammer Wt. **140** lbs.
Date Completed **10/13/14** 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.

SOIL CLASSIFICATION		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-1sf	Remarks
(continued)													
Brown, wet, medium dense SAND (SP)													
				45	12	SS				5-6-11			
		655.5	48.0										
Brown, moist, stiff SILTY CLAY (CL) with little sand and trace gravel													
				50	13	SS				7-6-9			
		650.5	53.0										
Light brown, wet, medium dense to dense SAND (SP-SM) with trace silt and little gravel													
				55	14	SS				9-11-14			
		643.5	60.0	60	15	SS				15-17-21			
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 **32.5** ft.
▽ At Completion **9.5** ft.
▼ After **--** hours **--** ft.
⚠ Cave Depth **19.0** ft.

Boring Method

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

CLIENT **American Structurepoint, Inc.**
PROJECT NAME **Proposed Building**
PROJECT LOCATION **Richard L. Roudebush VA Medical Center**
Indianapolis, Indiana

BORING # **B-2**
JOB # **86.00481.0321**
DRILLING and SAMPLING INFORMATION
TEST DATA

Date Started **10/13/14** Hammer Wt. **140** lbs.
Date Completed **10/13/14** 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.

SOIL CLASSIFICATION		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-1sf	Remarks
SURFACE ELEVATION 704.0													
4.5 in. Asphalt		703.6	0.4										Borehole cleared of underground utilities to a depth of 5 ft below the existing ground surface utilizing air-knife methods by Cardno TBE. Ground surface elevation estimated from topographic map provided by client. No soil samples could be obtained in upper 5 ft.
Blank drilled to 6.0 ft, no samples obtained													
		698.0	6.0	5									
Dark brown, moist clayey sand with trace gravel (FILL)		696.0	8.0		1	SS				3-4-4			
Brown, moist, medium stiff SANDY SILTY CLAY (CL) with trace gravel		693.5	10.5	10	2	SS				3-3-4	15.1		
Brown, moist, very loose to loose SILTY SAND (SM) with trace gravel					3	SS				2-2-2			
					4	SS				3-2-3			
				15	5	SS				2-2-2			
					6	SS				2-3-4			
		683.0	21.0	20	7	SS				6-11-10			
Brown, slightly moist, medium dense SAND (SP) with trace gravel					8	SS				9-6-7			
				25									Bottom of Test Boring at 40.0 ft
		676.5	27.5										
Brown, moist, hard SILTY CLAY (CL) with little sand and trace gravel		675.0	29.0	30	9	SS				25-20-11			
Brown, wet, dense SAND (SP) with little gravel													
		671.0	33.0										
Dark brown, wet, medium dense SAND (SP-SC) with trace clay and gravel				35	10	SS				12-12-17			
		664.0	40.0		11	SS				9-11-15			

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 **27.5** ft.
▽ At Completion **None** ft.
▼ After **--** hours **--** ft.
⊠ Cave Depth **23.0** ft.

Boring Method

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

CLIENT American Structurepoint, Inc.
PROJECT NAME Proposed Building
PROJECT LOCATION Richard L. Roudebush VA Medical Center
Indianapolis, Indiana

BORING # B-3
JOB # 86.00481.0321
DRILLING and SAMPLING INFORMATION
TEST DATA

Date Started	<u>10/13/14</u>	Hammer Wt.	<u>140</u> lbs.
Date Completed	<u>10/13/14</u>	Hammer Drop	<u>30</u> in.
Drill Foreman	<u>J. Cook</u>	Spoon Sampler OD	<u>2.0</u> in.
Inspector	<u>D. McIlwaine</u>	Rock Core Dia.	<u>--</u> in.
Boring Method	<u>HSA</u>	Shelby Tube OD	<u>--</u> in.

SOIL CLASSIFICATION		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
SURFACE ELEVATION 704.0													
15 in. Asphalt		703.6	0.4										Borehole cleared of underground utilities to a depth of 5 ft below the existing ground surface utilizing air-knife methods by Cardno TBE. Ground surface elevation estimated from topographic map provided by client. No soil samples could be obtained in upper 5 ft.
Blank drilled to 6.0 ft, no samples obtained													
		698.0	6.0	5	1	SS				5-4-4	13.6	2.0	
Dark brown, moist silty clay with little sand and trace gravel (FILL)					2	SS				3-3-4	14.5	1.5	
		693.0	11.0	10	3	SS				2-3-3	13.5	1.0	
Brown, moist, medium stiff SILTY CLAY (CL) with some sand and trace gravel		691.0	13.0		4	SS				2-2-3	19.4	1.5	
Dark brown, moist, soft SILTY CLAY (CL) with little sand and trace gravel		688.5	15.5	15	5	SS				4-3-4			
Brown, moist, loose SAND (SP-SM) with trace silt		686.0	18.0	20	6	SS				3-3-4	23.1		
Brown, moist, medium stiff SANDY SILTY CLAY (CL)					7	SS				3-3-3	26.9		
		681.0	23.0	25	8	SS				4-6-6			
Brown, slightly moist, medium dense SAND (SP)		676.0	28.0	30	9	SS				6-8-6			
Brown, wet, medium dense SAND (SP) with gravel					10	SS				3-4-8			
Dark brown, wet, medium dense to very dense SAND (SP)		671.0	33.0	35									Bottom of Test Boring at 40.0 ft
		664.0	40.0		11	SS				16-25-31			

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 28.2 ft.
▽ At Completion 13.6 ft.
▽ After -- hours -- ft.
⊠ Cave Depth 24.6 ft.

Boring Method

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

CLIENT **American Structurepoint, Inc.**
PROJECT NAME **Proposed Building**
PROJECT LOCATION **Richard L. Roudebush VA Medical Center**
Indianapolis, Indiana

BORING # **B-4**
JOB # **86.00481.0321**

DRILLING and SAMPLING INFORMATION

Date Started	<u>10/13/14</u>	Hammer Wt.	<u>140</u>	lbs.
Date Completed	<u>10/14/14</u>	Hammer Drop	<u>30</u>	in.
Drill Foreman	<u>J. Cook</u>	Spoon Sampler OD	<u>2.0</u>	in.
Inspector	<u>D. McIlwaine</u>	Rock Core Dia.	<u>--</u>	in.
Boring Method	<u>HSA</u>	Shelby Tube OD	<u>--</u>	in.

TEST DATA

Date Started		10/13/14	Hammer Wt.		140	lbs.												
Date Completed		10/14/14	Hammer Drop		30	in.												
Drill Foreman		J. Cook	Spoon Sampler OD		2.0	in.												
Inspector		D. McIlwaine	Rock Core Dia.		--	in.												
Boring Method		HSA	Shelby Tube OD		--	in.												

SOIL CLASSIFICATION		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-1sf	Remarks
SURFACE ELEVATION 703.0													
15 in. Asphalt		702.6	0.4										Borehole cleared of underground utilities to a depth of 5 ft below the existing ground surface utilizing air-knife methods by Cardno TBE.
Blank drilled to 6.0 ft, no samples obtained		697.0	6.0	5									
Brown, moist sandy silty clay with trace gravel (FILL)		695.0	8.0		1	SS	X	█		2-2-2			
Brown, moist, very loose to loose SAND (SP) with trace gravel				10	2	SS	X	█		2-1-1			
					3	SS	X	█		2-1-2			
				15	4	SS	X	█		2-1-2			
					5	SS	X	█		2-3-3			
					6	SS	X	█		2-3-3			
		682.0	21.0	20	7	SS	X	█		5-3-3			
Brown, slightly moist, loose SAND (SP)		680.0	23.0		8	SS	X	█		2-2-2			
Brown, moist, very loose to medium dense SAND (SP) with little gravel				25	8A	SS	X	█		5-6-14			
Brown, wet, medium dense to dense SAND (SP)		675.8	27.2		9	SS	X	█		10-8-10			
				30									
				35	10	SS	X	█		12-16-21			
		665.0	38.0										
Brown, wet, dense SAND (SP-SM) with trace silt and little gravel		663.0	40.0		11	SS	X	█		17-21-22			
Bottom of Test Boring at 40.0 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	<u>28.0</u>	ft.
📏	At Completion	<u>None</u>	ft.
⏱	After <u>24</u> hours	<u>27.2</u>	ft.
🏠	Cave Depth	<u>11.8</u>	ft.

Boring Method

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

CLIENT American Structurepoint, Inc.
PROJECT NAME Proposed Building
PROJECT LOCATION Richard L. Roudebush VA Medical Center
Indianapolis, Indiana

BORING # B-5
JOB # 86.00481.0321
DRILLING and SAMPLING INFORMATION
TEST DATA

Date Started 10/13/14 Hammer Wt. 140 lbs.
Date Completed 10/13/14 Hammer Drop 30 in.
Drill Foreman J. Cook Spoon Sampler OD 2.0 in.
Inspector D. McIlwaine Rock Core Dia. -- in.
Boring Method HSA Shelby Tube OD -- in.

SOIL CLASSIFICATION	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-1sf	Remarks
SURFACE ELEVATION 705.0												
4 in. Topsoil	704.7	0.3										Borehole cleared of underground utilities to a depth of 5 ft below the existing ground surface utilizing air-knife methods by Cardno TBE.
Blank drilled to 6.0 ft, no samples obtained												
	699.0	6.0	5	1	SS				10-9-7	14.7		Ground surface elevation estimated from topographic map provided by client. No soil samples could be obtained in upper 5 ft.
Dark gray, moist sandy silty clay with little gravel and trace roots (FILL)	695.5	9.5	10	2	SS				9-6-4	12.6		
Brown, moist, loose SAND (SP-SM) with trace gravel and little silt	692.0	13.0		3	SS				3-2-3			
Brown, moist, soft SILTY CLAY (CL) with some sand and trace gravel	689.5	15.5	15	4	SS				2-2-3	17.0		
Brown, slightly moist, very loose to loose SAND (SP)				5	SS				3-2-3			
				6	SS				3-3-4			
	684.0	21.0	20	7	SS				4-5-6			
Brown, slightly moist, medium dense SAND (SP) with trace gravel				8	SS				7-11-15			
				9	SS				7-12-7			
Brown, wet, medium dense SAND (SP) with little gravel	676.0	29.0	30									
	672.0	33.0	35	10	SS				7-9-14			
Brown, wet, medium dense to dense SAND (SP)												
				11	SS				14-19-22			

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 29.0 ft.
▽ At Completion None ft.
▼ After -- hours -- ft.
⊠ Cave Depth 18.7 ft.









Boring Method

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

CLIENT American Structurepoint, Inc.
PROJECT NAME Proposed Building
PROJECT LOCATION Richard L. Roudebush VA Medical Center
Indianapolis, Indiana

BORING # B-5
JOB # 86.00481.0321
DRILLING and SAMPLING INFORMATION
TEST DATA

Date Started 10/13/14 Hammer Wt. 140 lbs.
Date Completed 10/13/14 Hammer Drop 30 in.
Drill Foreman J. Cook Spoon Sampler OD 2.0 in.
Inspector D. McIlwaine Rock Core Dia. -- in.
Boring Method HSA Shelby Tube OD -- in.

SOIL CLASSIFICATION		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-1sf	Remarks
(continued)													
Brown, wet, medium dense to dense SAND (SP)		662.0	43.0										
Brown to gray, wet, dense to medium dense SAND (SP) with some gravel				45	12	SS				12-15-22			
				50	13	SS				15-19-19			
				55	14	SS				11-14-16			
		645.0	60.0	60	15	SS				30-42-31			
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 29.0 ft.
▽ At Completion None ft.
▼ After -- hours -- ft.
⚠ Cave Depth 18.7 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)

<u>Density</u>		<u>Particle Size Identification</u>	
Very Loose	- 5 blows/ft or less	Boulders	- 8 inch diameter or more
Loose	- 6 to 10 blows/ft	Cobbles	- 3 to 8 inch diameter
Medium Dense	- 11 to 30 blows/ft	Gravel	- Coarse - 1 to 3 inch
Dense	- 31 to 50 blows/ft		Medium - ½ to 1 inch
Very Dense	- 51 blows/ft or more		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)
<u>Relative Proportions</u>		Silt	0.074 to 0.002mm (cannot see particles)
Descriptive Term	Percent		
Trace	1 - 10		
Little	11 - 20		
Some	21 - 35		
And	36 - 50		

COHESIVE SOILS (Clay, Silt and Combinations)

<u>Consistency</u>		<u>Plasticity</u>	
Very Soft	- 3 blows/ft or less	Degree of Plasticity	Plasticity Index
Soft	- 4 to 5 blows/ft	None to slight	0 - 4
Medium Stiff	- 6 to 10 blows/ft	Slight	5 - 7
Stiff	- 11 to 15 blows/ft	Medium	8 - 22
Very Stiff	- 16 to 30 blows/ft	High to Very High	over 22
Hard	- 31 blows/ft or more		

Classification on the logs are made by visual inspection of samples.

Standard Penetration Test — Driving a 2.0" O.D. 1-3/8" I.D. sampler a distance of 1.0 foot into undisturbed soil with a 140 pound hammer free falling a distance of 30 inches. It is customary for ATC to drive the spoon 6 inches to seat into undisturbed soil, then perform the test. The number of hammer blows for seating the spoon and making the test are recorded for each 6 inches of penetration on the drill log (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). (ASTM D-1586-11).

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

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

Important Information About Your Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

The following information is provided to help you manage your risks.

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 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 your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you — should apply the 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.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of 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,

- elevation, configuration, location, orientation, or weight of the proposed structure,
- 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 study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the 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 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 construction recommendations included in your report. *Those 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 recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower 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. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing 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 Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors 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 contractors have sufficient time to perform additional study.* Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not 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.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental 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 geoenvironmental 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, a number of 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 ASFE-Member Geotechnical Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you ASFE-member geotechnical engineer for more information.



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

Copyright 2004 by ASFE, Inc. Duplication, reproduction, or copying of this document, in whole or in part, by any means whatsoever, is strictly prohibited, except with ASFE's specific written permission. Excerpting, quoting, or otherwise extracting wording from this document is permitted only with the express written permission of ASFE, and only for purposes of scholarly research or book review. Only members of ASFE may use this document as a complement to or as an element of a geotechnical engineering report. Any other firm, individual, or other entity that so uses this document without being an ASFE member could be committing negligent or intentional (fraudulent) misrepresentation.