Addendum #01

<table>
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<tr>
<th>Client</th>
<th>Ball State University</th>
<th>Date</th>
<th>July 26, 2018</th>
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<td>Project</td>
<td>New York Avenue Parking Structure</td>
<td>Champlin Project #</td>
<td>692-6004</td>
</tr>
<tr>
<td>BSU Project #</td>
<td>2018-014.01 XP</td>
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This addendum provides information to clarify or adjust construction items which may affect any or all trade contractors. The original documents for the referenced project are amended as noted in this addendum and made part of said documents and shall govern the work covered by the Form of Proposal. All work to be in strict accordance with the terms, stipulations and conditions of contract documents.

SUMMARY OF ATTACHMENTS

1. Attachments:
   a. Pre-Bid Agenda
   b. Pre-Bid Scope of Work Summary
   c. Pre-Bid Sign-In Sheet
   d. Geotechnical Engineering Investigation
      i. Updated July 26, 2018

PRE-BID MEETING SUMMARY

The Pre-Bid meeting took place on July 18, 2018 at 11:00am

1. The attached Agenda was distributed and reviewed
   a. Substantial Completion of July 19, 2019 was emphasized

2. The attached Scope of Work was distributed and reviewed

3. The attached Sign-In Sheets indicated attendance at the pre-bid

4. Pre-Bid discussion
   a. Contractors and subcontractors were reminded of that any scope of work exceeding $350,000 requires pre-qualification with the state.
   b. Contractors were encouraged to submit bids in person. Bids delivered by FedEx/UPS/etc do not get delivered directly to the location required for receipt of bids and there may be a delay which, if received after the specified time of receipt, would disqualify the bid and it would not be opened.
   c. Parking in the adjacent church lot has been confirmed to be fully committed and will NOT be available for permit parking by the contractor. The University is pursuing an alternative and will address in a future Addendum. Contractors are restricted to use of the church property as indicated on Sheet C001

5. Site walk-thru occurred immediately following the Pre-Bid meeting

All additional questions should be submitted in writing no later than 2pm on August 1, 2018 to Sean Bright via mail (sean.bright@thinkchamplin.com).

ALL BIDS ARE DUE AT 11:00 AM ON Tuesday, August 14, at Purchasing Conference Room, Service & Stores Building, 3401 N. Tillotson Avenue, Muncie, IN 47306, Attn: June Sanders.

Late bids will not be received.
PART 1 – CONTRACTOR QUESTIONS and ADDITIONAL CLARIFICATIONS (in no particular order)

1. COMMENT: Will the geotechnical report be issued?
   The attached Geotechnical and Subsurface Investigation prepared by ATC, updated July 26, 2018 shall become part of the Contract Documents and utilized in the preparation of bids, particularly associated with Section 31 62 00 “Augured Aggregate Pier Foundation System” and for use in determining extent of unsuitable soil removal and dewatering.

Issued By:

Champlin Architecture
Sean M. Bright, AIA
Principal

End of Addendum
New York Avenue Parking Structure  
Ball State University  
BSU Project No. 2018-014.01 XP  
July 18, 2018

I. Project Team  
A. Owner’s Representative(s):  
Kelly Knable, Facilities Planning & Mgmt, 765-285-0585, email: kaknable@bsu.edu  
Ryan Koener, Facilities Planning & Mgmt, 765-285-2821, email: rkoener@bsu.edu  
June Sanders, Purchasing, 765-286-1548, email: jasanders3@bsu.edu

B. Consultant’s Representative(s):  
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Stephen Culbert, Loftus Engineering, 317-352-5821, email: sculbert@loftusengineering.com  
Jennifer Lasch, Cripe Engineering, 317-706-8348, email: jlasch@cripe.biz

II. Contract Documents:  
A. Availability of Contract Documents.  
B. Interpretation of Contract Documents.  
C. Addenda.  
D. Substitutions.

III. Bidding Procedures.  
A. Bidding Date: August 14, 2018 @ 11:00 A.M. EDT  
Location: Purchasing Conference Room  
Service & Stores Building  
3401 N. Tillotson Avenue  
Muncie, Indiana 47306

B. Bidding Form and Other Documents.  
1. Indiana Form 95 (Revised 2013).  
a. Fill out Part II, Section I. Experience Questionnaire  
b. Fill out Part II, Section II. Plan and Equipment Questionnaire.  
c. Attach Part II, Section III. Contractor’s Financial Statement.  
d. Fill out Part II, Section IV. Contractors Non – Collusion Affidavit  
e. Fill out Part II, Section V. Oath and Affirmation

2. Bid Form Supplements, Document 00 43 00  
Appendix A.  
(1) Acknowledgment of Receipt of Addenda.  
(2) Project Completion/Liquidated Damages $5,000  
Appendix B. Alternatives, 4 Alternates  
Appendix C. Unit Prices, 4 Unit Prices  
Appendix D. Principal Subcontractors  
Appendix E. Supplementary General Construction Information  
Appendix F. Supplementary Mechanical Information  
Appendix G. Supplementary Electrical Information  
Appendix H. Supplementary Telecommunications Information

3. Representations and Certifications, Document 00 45 00  
Appendix 1. Nondiscrimination Compliance Statement  
Appendix 2. Contractors Certification of Self Performance  
Appendix 3. Contractors Certification of Authorized Employment  
Appendix 4. Drug Testing Plan  
Appendix 5. Contractors Certification of Training Program Compliance  
Appendix 6. Contractors Certification of Pre-Qualification Compliance  
Appendix 7. Bidder’s Check List

Pre-Bid Conference – Agenda  
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IV. Scope of Project.
   A. Summary of Work.
   B. Project Schedule.
   C. Access to Project Area.
   D. Coordination with Other Projects.
   E. Coordination with Owner Occupancy.

V. Questions.

VI. Tour of Project Site.

End of Agenda
Ball State University  
New York Avenue Parking Structure  
2018-014.01XP

Pre-Bid Meeting  
July 18, 2018; 11:00 am

A. Summary of Work: Construction of a new 4-tier, cast-in-place, 600 space parking structure, including all civil, landscape, structural, architectural, signage, fire protection, plumbing, mechanical, electrical, security and communications.

B. Earliest Possible Award: August 27, 2018

C. Substantial Completion: July 19, 2019

D. Site Work: Tree removal and protection, dewatering, unsuitable soil removal, major utility work and relocations, widening of Studebaker Drive, bus stop on New York, surface detention basis, basement detention, site lighting, bike racks, sidewalks

E. Structural Work: Post-Tensioned Cast-in-Place primary structure, stair towers, miscellaneous steel

F. Architectural: Precast panels with thin brick, masonry, curtainwall/glazing, elevator, aluminum panels and custom trim, doors/frames/hardware, greenwall, transformer enclosure, louvers, metal canopies, soffits, roofing

G. Mechanical: Ventilation of stair towers, mechanical rooms, basement detention basin, conditioning of elevator components, conditioning of communications room

H. Plumbing: Storm and sanitary drainage, green wall water supply, coordination with site drainage, pumping package

I. Fire Protection: Dry standpipes, fire department connections

J. Electrical: EM Generator, lighting, back-up power, EM phones, fire alarm, communications, car charging stations, routing of conduit

K. Access to Project Area:  
Construction traffic flow  
Construction boundary  
Dumpster locations  
Use of Church Property
<table>
<thead>
<tr>
<th>Name</th>
<th>Representing</th>
<th>Phone Number</th>
<th>Email Address</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ryan Koester</td>
<td>BSU - FPM</td>
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GEOTECHNICAL ENGINEERING INVESTIGATION

PROPOSED EAST PARKING GARAGE
BALL STATE UNIVERSITY
NORTH NEW YORK AVENUE AND STUDEBAKER DRIVE
MUNCIE, INDIANA

ATC PROJECT NO. 170GC00601

JULY 26, 2018

PREPARED FOR:

MR. SEAN M. BRIGHT, AIA, LEED AP
PRINCIPAL
CHAMPLIN ARCHITECTURE
212 WEST 10TH STREET, SUITE A-435
INDIANAPOLIS, INDIANA 46202
July 26, 2018

Mr. Sean M. Bright, AIA, LEED AP
Principal
Champlin Architecture
212 West 10th Street, Suite A-435
Indianapolis, Indiana 46202

Re: Geotechnical Engineering Investigation
Proposed East Parking Garage
Ball State University
North New York Avenue and Studebaker Drive
Muncie, Indiana
ATC Project No. 170GC00601

Dear Mr. Bright:

Submitted herewith is the report of the geotechnical engineering investigation performed by ATC Group Services LLC (ATC) for the referenced project. This study was authorized in accordance with ATC Proposal- Agreement No. PE-18-1020 dated February 1, 2018.

This report contains the results of our field and laboratory testing program and an engineering interpretation of this data with respect to the available project characteristics. 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,

Stephen Rushfeldt, P.E.
Senior Project Engineer

Thomas J. Struewing, P.E.
Principal Engineer
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Appendix
1 PURPOSE AND SCOPE

The purpose of this study was to characterize the general subsurface conditions at the project site by drilling eight test borings, in addition to one test boring performed by ATC in April, 2017 for a preliminary subsurface investigation; and to evaluate this data with respect to the earth-supported elements associated with the proposed East Parking Garage on the campus of Ball State University in Muncie, Indiana. 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

Ball State University is planning the construction of a new parking garage on the east side of the Ball State University campus in Muncie, Indiana. The proposed East Parking Garage will be located southwest of the intersection of North New York Avenue and Studebaker Drive. The First Presbyterian Church borders the project site to the south and Studebaker Hall borders the project site to the north. Park Hall borders the project site to the west and student apartments and residences border the project site to the east. The general location of the proposed parking garage on the campus is shown on the Vicinity Map (Figure 1 in the Appendix).

The footprint area of the proposed parking garage is mostly a mowed grass area with sparse, mature trees. An existing 36 in. diameter reinforced concrete storm sewer runs beneath the site from near the northwest corner of the proposed parking garage to near the south-central portion of the proposed garage. The existing ground surface is relatively flat and varies from about El 927 at the northwest portion of the property to about El 931 at the northeast portion of the property. In general, the project area is slightly depressed relative to the surrounding grades. It is reported that surface water frequently collects within the project area during heavy precipitation events. It is also believed that York Prairie Creek ("Cardinal Creek") may have flowed through this general area in the past.

It is our understanding that the proposed parking garage will have four elevated levels and will be a cast-in-place, post-tensioned concrete structure with plan dimensions of about 290 ft by 180 ft. Additionally, the central portion of the parking garage will include a basement level that will serve as a storm-water detention basin for a peak capacity of about 150,000 cubic feet of water. The proposed basement floor level will vary from about El 915.7 to El 919.5, which will be about 15 to 17 ft below the main parking level (ground level) of the garage. The basement floor level will be about 11 to 15 ft below the existing ground surface. It is our understanding that the main parking level will vary from about El 932.8 to El 934.5, which is about 4 ft to 6 ft above the existing site grade. The parking garage will be accessed via new entrance and exit drives along the south side of Studebaker Drive.

In addition to the basement level storm-water detention basin, the project will include the construction of two shallow exterior detention ponds east and southwest of the proposed parking garage. We understand that the detention ponds will have a base elevation of about El 927. The location of the proposed parking garage and other site features, along with existing facilities at the project site, are shown on the Boring Plan (Figure 2 in the Appendix).
It is our understanding that the new parking garage structure will have maximum interior and exterior column loads of approximately 750 and 450 kips/column, respectively. It is assumed that the maximum wall loads will not exceed about 8 kips/lin.ft. No unusual loading conditions or settlement restrictions have been specified.

3 GENERAL SUBSURFACE CONDITIONS

The general subsurface conditions were investigated by drilling eight test borings to depths of 20.0 ft to 60.0 ft below the existing ground surface 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. In addition to the test borings drilled for this investigation, a soil boring log for one test boring drilled for a preliminary geotechnical engineering investigation at the project site in April, 2017 is included in the Appendix.

The test borings were drilled in a low-lying grassy lawn area with relatively sparsely spaced, large, mature trees. Topsoil was encountered at the surface in each of the test borings varying from about 9 to 11.5 inches thick.

Underlying the topsoil, the test borings generally encountered layers of dark brown, dark gray and/or black, soft to medium stiff, silty clay (CL) and sandy silty clay (CL) with marl, organic material, wood fragments and root material, along with occasional traces of gravel. Borings B-101 and B-104 encountered layers of dark brown or black, very loose, clayey sand (SC) with varying amounts of gravel and organic material. The softer cohesive soil and looser clayey sand layers with organic material extend to depths of about 3 ft to 7 ft below the existing ground surface. Standard Penetration Test N-values within the softer cohesive and looser clayey sand soils varied from about 3 to 7 blows/ft. The organic content of these soils varies from about 1 to 21 percent. The percentage of marl (Ca/MgCO₃) in these layers varied from about 4 to 47 percent.

Below the upper layers of soils with organic material, the test borings generally encountered brown or gray, wet, medium dense to very dense layers of sand and gravel (SP, SP-SM, SW-SM, SC, SM, GP-GM and GW-GM) with varying amounts of silt to depths of 20.0 ft to 50.0 ft below the existing ground surface. Standard Penetration Test N-values within the granular soils typically were in the range of about 13 to 56 blows/ft. Occasional looser sand layers with Standard Penetration Test N-values of about 8 to 10 blows/ft were encountered within about the upper 15 ft below the existing ground surface.

Beneath and/or interbedded within the sand and gravel in six of the test borings, are layers of gray, slightly moist to very moist, stiff to hard sandy and/or silty clay (CL) with traces of gravel. Moisture contents within the cohesive sandy and/or silty clay (CL) varied from about 9 to 15 percent. Standard Penetration Test N-values typically varied from about 12 to 62 blows/ft.
Cobbles and boulders were encountered at varying depths as noted on the test boring logs in five of the test borings. Our experience also indicates that cobbles and boulders are often present within glacial soils such as those that underlie this site. Therefore, it is important to understand that cobbles and boulders will be encountered at various locations and depths at this site.

The consistencies of the cohesive soils as described above and on the boring logs were estimated based on the results of the standard penetration test (ASTM D-1586) and based on the definitions as described on the Field Classification System for Soil Exploration contained in the Appendix of this report.

Ground water level observations were made during the drilling operations by noting the depth of water on the drilling tools and in the open boreholes following withdrawal of the drilling augers. Free ground water was observed in each of the test borings at depths varying from about 1 ft to 7 ft below the existing ground surface, although ground water was typically encountered at depths within a range of about 3 ft to 6 ft below the existing ground surface. Thus, the ground water level at the site appeared to generally be in the range of about El 923 to 927 at the time of this investigation. Short term ground water level observations made in granular soils (such as the predominant soils beneath this site) are typically 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 and other factors not evident at the time when the test borings were drilled. It is expected that higher ground water levels could occur at other times during the life of this structure. It is also possible that “perched” ground water may be encountered at various depths and locations across the site above the hydrostatic ground water level. Water is often trapped within old miscellaneous fill materials, abandoned utilities, utility trenches, etc. and although the amount of such water is usually not significant, it is important to recognize that such ground water may be encountered at various depths and locations across the site due to the presence of existing utilities underlying and adjacent to the project site.

4 FINDINGS, CONCLUSIONS AND DESIGN RECOMMENDATIONS

The following findings, conclusions and design recommendations are based on the previously described project characteristics (Section 2) in conjunction with the subsurface conditions (Section 3) that were investigated for this project. If there is any change in the project criteria, including proposed location and finish floor elevations of the project elements, loading conditions, structure types, etc., a review should be made by this office. The design recommendations presented herein are based upon the assumption that careful field observations, testing and evaluations of all of the soil related aspects of the project will be performed during construction as described herein. These field observations are considered a critical part of the design of the project.

4.1 General Foundation and Slab-On-Grade Floor Concepts

The test borings drilled for this project revealed an upper stratum of softer and looser soils with a relatively high percentage of organic and marl material to depths of about 3 ft to 7 ft below the existing ground surface. These soft, loose and organic soils are quite compressible and not suitable for the reliable support of the proposed parking structure, including the slabs-on-grade. The soils encountered in the test borings at this project site are similar to the soils encountered in the test borings drilled prior
to the construction of Worthen Arena in the 1980s and also in 2016 for the Shondell Practice Center, which are in a similar geologic setting, confirming the presence of soft, loose, organic and marly soils of varying strength, compressibility and consistency within the mapped geological soil unit (Urban Land – Millgrove Complex). Additionally, we understand that Cardinal Creek (York Prairie Creek) formerly ran through or immediately adjacent to the west side of the property and that the adjoining geologic map units are characterized by depressed areas and outflow deposits.

Underlying the upper stratum of soft, compressible organic soils are layers of medium dense to very dense sand and gravel and stiff to very stiff cohesive sandy clay glacial till soils that are considered suitable for support of the proposed structure (i.e., spread footings and slab-on-grade floors). It is important to note that occasional layers of looser sand and gravel may be encountered within the upper approximately 15 ft below the existing ground surface. Although the underlying denser sand and gravel layers and stiffer sandy clay cohesive glacial till soils are considered suitable for support of the proposed structure, cobbles and boulders were encountered at varying depths in the test borings, which will require special consideration when excavating for foundation elements. Shallow saturated sand and gravel layers with ground water under apparent pressure were encountered and thus significant dewatering efforts will be required during construction. Furthermore, significant permanent dewatering measures and/or watertight design elements will be required for the below-grade project elements, such as the basement level storm water detention area and the elevator pits.

The following table summarizes the depths and estimated elevations at which soils that are judged to be suitable for reliable support of conventional spread footings and slab-on-grade floors (i.e., medium dense sand and gravel or stiff sandy clay) were encountered in the test borings. It is important to note that the depths to suitable bearing soils will vary across the site and suitable bearing soils may be encountered deeper at other locations. Additionally, if the existing 36-inch diameter storm sewer underneath the western portion of the proposed parking garage footprint is removed, and other subsurface utilities or below-grade structures are demolished, deeper undercutting and replacement with engineered fill will likely be required.
Estimated Depths to Suitable Bearing Soils for Conventional Spread Footings and Slab-On-Grade Floors

<table>
<thead>
<tr>
<th>Boring No.</th>
<th>Estimated Ground Surface Elevation, ft*</th>
<th>Estimated Depth to Suitable Bearing Soil, ft</th>
<th>Estimated Elevation of Suitable Bearing Soil, ft*</th>
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<tr>
<td>B-1</td>
<td>930</td>
<td>3.5</td>
<td>926</td>
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<td>B-101</td>
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<td>B-108</td>
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*Ground surface elevations estimated from handheld GPS and Delaware County GIS mapping.

In order to support the proposed parking structure on conventional spread footings and to use a slab-on-grade ground floor, it will be necessary to first remove all existing unsuitable soils including the softer, looser and organic soils at the footing locations and beneath the slab-on-grade floors in order to reach the natural, medium dense to very dense sand and gravel or the natural, stiffer cohesive sandy clay (glacial till) that is judged to be suitable for reliable support of conventional spread footings and slab-on-grade floors. This will require significant temporary dewatering measures in order to remove the unsuitable materials and replacement of the unsuitable materials with well-compacted granular fill materials as recommended in Section 5.3 of this report. It is recommended that only well-graded granular soils such as pit-run sand and gravel or INDOT No. 53 crushed limestone should be used to fill the required undercut excavations. Because of the highly compressible materials that currently exist over the entire footprint area of the parking garage structure, it will be necessary to completely remove the unsuitable materials, including beneath the floor slab areas. In order to ensure that the unsuitable materials are completely removed from beneath the entire zone of influence of the structure, it is recommended that the base of the undercut excavation extend at least 15 ft beyond the outside edge of the structure.

As an alternative to completely removing and replacing the soft, loose, compressible organic soil and other unsuitable materials in the non-basement areas of the proposed parking garage (it is apparent that the basement footings and floor slab will extend below the unsuitable soils), it may be possible for the existing natural soils to be modified and improved in-place with a ground improvement technique such as aggregate columns. Due to the variability in the soil layers encountered at this site (e.g., shallow soft/loose/organic soils, denser sand and glacial till, etc.) that extend to varying depths below the existing ground surface, the ground improvement system selected must be able to suitably improve the existing subsurface materials within the depth zone required for proper bearing of spread footings and...
support of the slab-on-grade floors. Since aggregate column systems are proprietary specialty geotechnical systems that result in modified subgrade materials, the ground improvement plan and final spread footing and slab-on-grade design criteria shall be developed and prepared by an engineer registered in the State of Indiana from the specialty geotechnical contractor who shall be entirely responsible for the design, installation, performance and warranty of the system. It will be necessary to consult directly with the design/build specialty geotechnical contractor and to convey to them the project characteristics and requirements such as settlement restrictions, loading conditions, finish floor elevation, existing grade, etc. The specialty design/build contractor should be made aware of the amount of new grade-raise fill that is required to be placed (it is estimated that about 4 ft to 6 ft of new grade-raise fill is required over the entire structure area), the presence of the basement level under the central portion of the parking garage along with any associated dewatering measures that will affect the settlement behavior of the existing soils. The ground improvement would be required for the structure foundations as well as for the floor slab. The ground improvement measures will also need to be installed in subsurface soil conditions with high ground water. Furthermore, if aggregate columns are to be used, consideration must be given to potential issues regarding ground vibrations during installation of the aggregate columns and potential impact on the existing surrounding facilities, as well as potential obstructions such as large cobbles or boulders. Additional information regarding ground improvement measures are provided in Section 4.2.1.

Other foundation systems such as auger-cast piles may also be used to support the structure, including the floor slab that would need to be designed as a structural slab supported on piles. Additional recommendations for auger-cast piles can be provided upon request.

The shallow unsuitable organic soils encountered at this site are generally cohesive in nature and the deeper layers of sand and gravel are generally relatively dense with relatively large particle sizes and correspondingly open graded. It is therefore concluded that liquefaction (or any significant loss of strength during ground shaking) of the soils underlying the project site during earthquake ground motions is extremely unlikely. To our knowledge, there are no recorded cases of liquefaction of subsurface materials similar to those at this project site within the general project vicinity. Therefore, no special design measures relative to soil liquefaction appear to be warranted.

Because of the high ground water level that exists at this site, it may be more cost-effective to design the basement portion of the parking structure as a watertight structure and utilize a mat foundation for support of the structure. This would require the installation of a continuous watertight mat foundation capable of resisting the gravity loads as well as buoyant pressures due to being submerged below the ground water level. The basement walls would also need to be designed for full hydrostatic pressure and made watertight (see Section 4.4).

Based on geologic mapping and the results of the test borings drilled for this project, it is our opinion that the subsurface conditions at this site meet the criteria for Site Class “D” 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”). Based on the USGS hazard data available for latitude 40.202405° N and longitude 85.401979° W, $S_{DS} = 0.143g$ and $S_{D1} = 0.116g$ should be used as the seismic design parameters for the proposed structure.
4.2  Conventional Spread Footings

Our findings show that the proposed parking garage can be constructed using conventional spread footings bearing on the firm natural medium dense sand and gravel or the stiff sandy clay (glacial till) soils that were typically encountered in the test borings at or below about El 923 to El 918, or on well-compacted structural fill that is placed over such materials after first removing all unsuitable materials as described in Section 4.1. It is extremely important that the soil at the bases of all foundation excavations (e.g., spread footings) be carefully observed, tested and evaluated as described in Section 5.4 in order to identify unsuitable materials that must be removed and replaced, such as old utilities, utility trench backfill, old foundations, construction debris, old fill, organic soils, soft natural soils, etc. and to verify that the footings will bear on the firm natural medium dense sand and gravel or stiff sandy clay. It is anticipated that undercutting of unsuitable materials will be required at all non-basement foundation locations to remove existing softer/looser natural soils, organic deposits, remnants from previous construction such as existing utilities, and any other unsuitable materials as described in Section 4.1.

Spread footings that bear on suitable firm natural soil as described above, or on well-compacted engineered fill (or on lean concrete) that is placed over firm natural soil after first removing all unsuitable materials as described above, can be designed for a maximum net allowable soil pressure 4,000 lbs/sq.ft for column footings and 3,000 lbs/sq.ft. for wall footings, provided that all unsuitable materials (e.g., softer/looser soil, organic material, subsurface utilities, etc.) are removed and replaced with well-compacted engineered fill or lean concrete. The allowable bearing capacities can be increased by a factor of 1.33 for transient loading conditions such as wind gusts and earthquake loads. A vertical modulus of subgrade reaction value \(k_{30}\) of 25 lbs/cu.in. can be used for the design of the spread footings or a mat foundation. It is important that the soil at the base of each spread footing excavation be carefully observed and evaluated as described in Section 5.4 to determine whether the actual bearing materials are consistent with those upon which the recommendations are based. All remnants from previous construction, such as old utilities, etc. should be removed from beneath the spread footings. It is recommended that the contract documents include provisions for the removal and replacement of unsuitable materials as determined to be necessary based on field observations at the time of construction.

Provided that the spread footings are designed as prescribed herein and the soils at the bases of the foundation excavations are observed and evaluated as outlined in Section 5.4, it is estimated that the total and differential post-construction foundation settlements of similarly loaded spread footings should not exceed about 1½ in. and ¾ in., respectively. Careful field control will contribute substantially to minimizing the settlements. Wall footings should be at least 3 ft wide and column footings should be at least 6 ft wide for bearing capacity considerations. All foundations should be located at a depth of at least 3 ft below the final exterior grade for frost protection.

Care must be exercised when excavating near existing facilities (e.g., pavements, utilities, tunnels, etc.) to protect the integrity of the existing facilities and project elements. Bracing or underpinning may be required if it becomes necessary to excavate below the bottom elevation of the existing pavements, tunnels, utilities and other project elements.
Uplift forces on the spread footings can be resisted by the weight of the 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 submerged soil unit weight of 65 lbs/cu.ft can be used for the backfill material placed above the footings, provided it is compacted as recommended in Section 5.3. 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 shallow spread footing can be resisted by the passive lateral earth pressure against the side of the footing and by friction between the subgrade soil and the base of the footing. If passive lateral earth pressure is to be used to resist lateral loads imparted on the spread footings, it is essential that the soil that is relied upon to provide the passive lateral earth pressure resistance cannot be excavated or otherwise disturbed at any time in the future. If it is possible that disturbance or an excavation could be made in any portion of the passive zone (including not only soils beside the footings but soils above the tops of footings), then passive lateral earth pressure resistance should not be considered. An allowable passive lateral earth pressure of 60 lbs/sq.ft per foot of depth below the ground surface or floor slab 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 0.2 (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.

Due to the presence of soft, loose and organic soils at shallow depths, lightly loaded project elements (such as site retaining walls, lightly loaded canopies, etc.) should also be supported on footings bearing on firm natural soils in a manner similar to the parking structure foundations.

4.2.1 Aggregate Columns

It may be possible and cost effective to use a proprietary in-place soil modification or ground improvement technique such as aggregate columns to modify and improve the existing subsurface materials such that spread footings and slab-on-grade floors could be used without the need for complete removal and replacement of the unsuitable materials as described in Sections 4.1 and 4.2. In this case, consideration must be given to the aggregate columns relative to the existing underground utilities that will remain in service (e.g., storm sewers, tunnels and other subsurface utilities); along with other site elements. If aggregate columns 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 elements adjacent to existing facilities such as pavements, tunnels, utilities, etc. to ensure that existing features are not adversely affected due to the installation of the aggregate columns. Consideration must also be given to the sequencing of aggregate column installation with respect to the construction of the basement such that the installation process does not adversely affect the new basement walls or foundations, or that excavation for the basement does not compromise the integrity of previously installed aggregate columns.
It is recommended that the specialty geotechnical contractor be required to confirm the compatibility of the selected proprietary ground improvement system with the subsurface soil and ground water conditions and the project requirements (e.g., loading conditions, settlement criteria, the addition of grade-raise fill, permanent dewatering of the basement portion of the structure, etc.). Due to the variability in the type and condition of the existing subsurface materials at this site, which includes zones of softer cohesive soils, looser granular soils and organic soils 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 of spread footings and proper control of settlement due to structure loading and loading due to new grade-raise fill. Because of the variability in the depth to the stronger natural bearing soils over relatively short lateral distances, and thus uncertainty of the condition of the existing subsurface materials at any specific foundation location, the specialty geotechnical contractor must install the ground improvement measures to sufficient depths and fashion such that soils within the zone of influence of the foundation are appropriately modified in order to enhance the reliability of the ground improvement measures and limit settlement.

Aggregate columns are proprietary techniques 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. After the “in-place” improvement/modification, spread footings and slab-on-grade floors can be used without undercutting and replacement of the existing unsuitable materials. If such a system is to be used, consideration by the specialty geotechnical contractor must be given to potential issues regarding ground vibrations during installation of the aggregate columns and potential impact on adjacent structures, operations and functions; as well as potential obstructions that may exist within the natural soils such as cobbles and boulders. The specialty geotechnical contractor should be consulted regarding the type of equipment and method of aggregate compaction used to determine the magnitude of ground vibrations and potential adverse impacts on the existing surrounding facilities.

Ground improvement techniques such as aggregate column soil modification and ground improvement systems are proprietary specialty geotechnical design/build procedures that shall be designed by a registered engineer with the specialty geotechnical foundation contractor and installed by a specialty geotechnical contractor. Therefore, the aggregate column contractor shall be required to confirm compatibility of the selected ground improvement system with specific applicability to this project and the development of the specific program to meet the project requirements (i.e., bearing capacity and settlement limitations). Spread footings that bear on modified and improved subgrade materials as described above can usually be designed for an allowable bearing pressure in the range of about 4,000 to 6,000 lbs/sq.ft while limiting settlement within required project tolerances without the need for undercutting and replacing the existing soils 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. It is recommended that the ground improvement system be designed to limit settlement to the criteria prescribed by the structural engineer. Since aggregate column systems are proprietary specialty geotechnical systems that result in modified subgrade 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 from the specialty geotechnical contractor who shall be entirely responsible for the design, installation, performance and warranty of the system.
4.3 **Ground Supported Slabs**
The existing soft, loose and/or organic soils encountered within the upper approximately 3 ft to 7 ft below the existing ground surface are not suitable for the reliable support of the slab-on-grade floors for the proposed parking structure, particularly since approximately 4 ft to 6 ft of new grade-raise fill will be required to establish the ground floor grade. The compressibility characteristics of the existing shallow soils pose a risk of differential settlement to the ground supported slabs that could be detrimental to the performance of the structure. While the engineered grade-raise fill is anticipated to be suitable for the support of slab-on-grade floors, the underlying natural compressible organic soil will likely experience excessive settlement during the life of the structure due to drying and/or increase in moisture content, degradation of the organic material, and long-term compressibility characteristics of the organic cohesive soils with respect to the applied loading of the grade-raise fill and ground-supported slabs. Therefore, it is recommended that all existing soft, loose and/or organic soils be removed from beneath the ground supported slabs prior to placement of grade-raise fill as described in Section 4.1. Alternatively, ground improvement measures could be implemented for the slab-on-grade floors in a fashion similar to those described in Sections 4.1 and 4.2.1 in order to control settlement of the floor slab.

It is recommended that all slab-on-grade floors 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.

It is recommended that the non-basement slab-on-grade floors be supported on a 6 in. thick (minimum) layer of 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. Recommendations regarding underslab materials and drainage measures below the basement level floor slab are presented in Section 4.4.

The proposed main-level floor slabs are anticipated to be about 4 ft to 6 ft above the existing ground surface. Based on the ground water conditions at the time of the test borings and our knowledge of the tendency of the project site to receive standing water during significant precipitation events, a high ground water level corresponding with the existing ground surface should be anticipated. Furthermore, the ground water elevation at this site can be expected to vary within a range of several feet during the life of the structure. Therefore, if any floor finishes or floor coverings that are to be used for the lobby or stairwells of the parking garage are sensitive to moisture, a vapor barrier should be included beneath the floor slabs in those areas of the structure that will receive the moisture sensitive floor finish or floor covering. It is recommended that where vapor barriers are used that the vapor barrier should be installed in accordance with ACI Manual of Concrete Practice 302.1R, “Guide for Concrete Floor and Slab Construction”.

4.4 **Basement, Below-Grade Walls and Permanent Dewatering**
The magnitude of the lateral earth pressure against the below-grade walls is dependent on the method of backfill placement, the type of backfill materials used and drainage provisions. When a wall is held rigidly against horizontal movement (such as a below-grade wall that is braced by the floors, structural framing and the other walls), 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, the below-grade walls (including those that may be designed as cantilever walls) must be designed for higher, "at-rest" lateral earth pressures (using an at-rest 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.
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 base of the wall footing. Any granular materials that are imported for this purpose should conform to INDOT Standard Specifications Section 211.03.1(c) Structure Backfill Type 3, except that No. 30 gradation material shall not be used and only natural sand, gravel or crushed limestone shall be used. Provided that well-graded, free-draining granular soils are used for backfill behind the below-grade walls, a total soil unit weight of 130 lbs/cu.ft and a coefficient of lateral earth pressure at-rest ($K_o$) of 0.45 can be used to calculate the lateral earth pressure against the below-grade walls using Figure 3 in the Appendix. If the on-site cohesive soils are used for backfill within any portion of the backfill zone described above (or if a temporary retention system is used that results in any of the existing soils remaining in-place within this zone), a coefficient of lateral earth pressure at-rest of 0.65 should be used for calculating the lateral earth pressure. If on-site cohesive soils are used for backfill within the zone defined above (or 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. 5 or No. 8 coarse aggregate placed against the walls).

For relatively short site retaining walls that are designed as cantilever retaining walls that are free to rotate sufficiently to develop the active lateral earth pressure condition, an active lateral earth pressure coefficient ($K_a$) of 0.33 and a total soil unit weight of 130 lbs/cu.ft. can be used to calculate the lateral earth pressures on these walls and footings provided that well-graded free-draining granular backfill material is used behind these walls. An allowable coefficient of friction between the base of the retaining wall footings and the foundation soils of 0.20 can be used to resist lateral forces based on a factor of safety of 1.5 relative to lateral resistance of the retaining wall footings. Lateral forces can also be resisted by the passive lateral earth pressure against the sides of the wall footings. An allowable passive earth pressure ("equivalent fluid pressure") of 60 lbs/sq.ft per foot 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). If passive earth pressure is to be used to resist lateral forces, it is essential that the earth that is relied upon to provide the passive lateral earth pressure cannot be excavated or disturbed in the future, including all of the soils above the footing level.

The basement finish floor level (El 915.7 to El 919.5) will be established well below the ground water level that was measured during this investigation (i.e., ground water levels generally measured in the test borings in the range of about El 923 to El 927 at the time of this investigation) and it is expected that the ground water level will be higher at other times during the life of this structure. Therefore, unless the basement structure is designed to be watertight and designed to resist the buoyant and hydrostatic pressures, a significant permanent dewatering system will be required in order to reliably and continuously maintain the ground water level below the basement floor level. Such a permanent dewatering system would be required to collect and pump ground water continuously during the entire life of the structure. As such, the permanent dewatering system will require redundancy of various elements so that the system can be maintained, equipment replaced as needed and the system cleaned routinely during the life of the structure.
It is not possible to determine with complete certainty the highest ground water level that could occur during the life of the structure without an extensive ground water monitoring program over a period of many years. However, based upon our experience and engineering judgment, it is expected that the ground water level at this site will be above the levels encountered in the test borings for this investigation during the life of the structure. At a minimum, it is suggested that a high ground water level of El 931 be used as the minimum design high ground water level for this site. Because of the significant permanent dewatering measures that would be required in order to permanently depress the ground water level below the basement floor level, it may be more desirable to use a “passive system” and construct the basement to be watertight and designed to resist the buoyant and hydrostatic pressures that will be imparted upon the submerged structure. In this case, Figure 3 in the Appendix can be used for the design of the basement walls with hydrostatic pressure in conjunction with the design high ground water level suggested above. The buoyant pressures that will be exerted upon the basement floor slab (uplift pressure) due to being submerged can be determined based upon the general concept depicted in Figure 4 in the Appendix, where the buoyant pressure acting at the base of the slab is equal to the depth of the base of the slab below the design high ground water times the unit weight of water.

Unless permanent dewatering measures are included around the elevator pits (similar to that recommended above for the basement) to maintain the ground water level below the base of the pit (which is often considered undesirable or impractical for elevator pit applications), the pit walls must be made watertight and designed for hydrostatic pressures and the pits designed to counteract buoyancy. It is not possible to accurately predict the future high ground water level; however, based upon the general regional geology and our experience, we recommend using a design high ground water level no lower than El 931 (assuming that permanent dewatering measures are installed for the basement as described above). The buoyancy of the pit can be resisted by increasing the weight of the pit structure and a “lip” can be added to the pit bottom such that the soil above the lip can be utilized to resist the uplift forces acting on the base of the pit. Similarly, the basement itself can be designed as a water-tight structure designed to resist buoyancy and uplift. Hydrostatic water pressures must be included on the basement walls in this case along with lateral earth pressures. Figure 4 in the Appendix can be used in the analysis and design for resistance of buoyancy of the pit and basement.

The specific design of a permanent dewatering system is beyond the scope of this investigation. It may be possible to permanently depress the ground water level using a series of wells around the perimeter of the basement, although this will require significant continuous discharge of water in order to adequately develop a composite “cone of depression” beneath the structure area. If wells are to be used, they should be designed and installed by an experienced specialty dewatering company that routinely designs and installs such dewatering systems. A permanent dewatering system may also include drains installed around the exterior perimeter of the basement along with a subfloor drainage system. The subfloor drainage system would include a layer of free-draining aggregate placed beneath the basement floor slab along with perforated drain pipes set in closely spaced trenches below the drainage layer and filled with the same free-draining aggregate as used beneath the floor slab. Further design recommendations for a perimeter and subfloor drainage system can be developed if desired.
It is not possible to determine with complete certainty the amount of ground water that will be required to be pumped for the permanent dewatering system for soils such as those at this site because the soil stratigraphy is not uniform and includes completely saturated open graded sands and gravel with occasional interbedded layers of cohesive glacial till soils. Furthermore, the recharge mechanisms of the subsurface soils are not well defined and can only be evaluated with an extensive hydrogeological study over a prolonged period. However, based on a permanent dewatering system for the proposed basement that includes subfloor trench drains and perimeter wall drains, in conjunction with our experience, it has been estimated that the maximum pumping rate may be on the order of approximately 2,000 gal./min. It is extremely important that multiple pumps of varying capacity be installed to handle variations in flow rate with potential flow rates varying from very little to the estimated maximum flow rate.

It is recommended that the project specifications require the temporary construction dewatering contractor to carefully monitor the construction dewatering and provide details of the system used to dewater the excavation along with the pumping rates of the system. This information should be evaluated to determine if additional pumping capacity is needed for the permanent dewatering system that will be installed.

4.5 Pavement

It is assumed that the main parking level will be designed as a ground supported slab and that traditional pavements will be limited to access ramps connecting the parking garage to the surrounding streets. It is further assumed that the pavement section of any access ramps will match the surrounding street pavement sections. Based on the results of classification tests and our experience with similar soils, a resilient modulus value of 4,000 lbs/sq.in has been estimated for use in pavement design for the clayey overburden soil. The subgrade soils should be prepared and inspected as described in Sections 5.2 and 5.3 of this report.

It is likely that the pavement subgrade in most areas will be wet, soft or yielding at the time of construction. Based on the limited space of the construction site and surrounding urban environment, excessively moist soils will need to be removed and replaced with well-compacted low-plasticity fill. It is anticipated that space to dry moist soil will not be available and that lime application would present challenges to maintaining the air quality of the surrounding campus area. Furthermore, it is anticipated that the organic content of the existing subgrade soils are not compatible with chemical stabilization and these soils are not considered suitable for support of pavement sections.

Based on our experience with soils of the type underlying this site, the natural subgrade soils at this site may yield and 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. In general, yielding subgrade problems are more prominent in cut areas (where saturated or nearly saturated silty and clayey soils are exposed by the excavation) or where little or no fill is to be placed.
Our experience on projects with similar subsurface conditions as those encountered on this project site indicates that settlement can occur due to future consolidation of the existing softer/looser soils and organic material without regard to loading. Changes in grading and surface water infiltration can initiate settlement along with degradation or collapse of materials within the fill. Therefore, in order to completely eliminate the risk of unacceptable differential settlement of the pavements it would be necessary to completely remove any existing soft, loose and organic materials that may be encountered and to replace them with well-compacted engineered fill.

In any case, it is recommended that the soils exposed at the pavement subgrade level should be carefully observed, tested and evaluated, including proofroll testing, to determine if there are any materials that need to be improved. Any remnants of previous construction that are exposed at the pavement subgrade level (such as utilities, pits, vaults, etc.) should 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.

The pavement subgrade surface should be uniformly sloped to facilitate drainage through the granular base and to avoid any ponding of water beneath the pavement. Storm water catch basins in pavement areas should be designed to allow water to drain from the aggregate base into the catch basins.

4.6 Site Grading and Drainage
Proper surface drainage should be provided at the site to minimize any increase in moisture content of the foundation soils or the below-grade wall backfill soils. The exterior grade should be sloped away from the structure to prevent ponding of water. Any roof drains or down spouts should be channeled or piped well away from the structure.

The soils encountered in the upper portion of the test borings are variable in composition, but are generally cohesive in nature and with relatively low permeability. Reliable disposal of significant amounts of storm water by infiltration measures in this upper zone will not be feasible. Additionally, the underlying sand and gravel layers are saturated and therefore also unreliable for reliable disposal of storm water.

It is our understanding that the proposed exterior detention ponds have been designed to release ground water into outlet structures at the bases of the ponds. Therefore, we understand that the ponds are not intended to maintain a constant pool of water, nor have they been designed to rely on infiltration for discharge of water. It should be noted that depending on seasonal ground water conditions, seepage of ground water into the ponds may occur. Therefore, the storm water outlets must be designed to accommodate a combined flow of storm water runoff and ground water seepage based on a possible high ground water elevation of about El 931. The basin must also be sized accordingly such that the intended detention capacity can be achieved during periods when the ground water level is higher than the invert of the ponds. Any fill placed along the side of the basin should be placed in accordance with Section 5.3.
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 ATC 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 Mass Excavation

It is our understanding that it will be necessary to make a mass excavation for the basement area to a depth of about 10 to 13 ft below the existing ground surface. Based on the basement location, it is anticipated that an open-cut excavation will be possible and an earth retention system is unnecessary (except near existing facilities such as surrounding pavements). It is recommended that the temporary excavation sideslopes be made no steeper than 2 (horizontal) to 1 (vertical). The actual slope configurations must be determined by the contractor responsible for the temporary excavation, construction means and methods and site safety. Some sloughing of loose material should be expected with such slopes and the slopes should be 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.

All existing footings, floors, utilities, etc., should be suitably protected from undermining due to excavation for the new structure. Depending on the relative depths and locations of the new and existing footings and the need to remove unsuitable soils at new footing locations, bracing or underpinning may be needed to protect the existing surrounding facilities. All federal, state and local safety regulations should be followed in this regard.

It is recommended that a baseline survey be made of all pertinent surrounding site features before construction is initiated. This will include establishing benchmarks and initial elevations on utility structures, 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, ground depressions, etc.) in the existing site features. 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 that could give an indication of the performance of the retention system or possible pending failure.
5.2 Site Preparation

Unless special ground improvement measures are taken, it will be necessary to remove all unsuitable soils as described in Section 4.1 beneath the parking garage structure and at least 15 ft beyond the outside limits of the parking garage structure to expose the medium dense sand and gravel soils or the stiff sandy silty clay soils. Removal of unsuitable soils will generally extend to about EL 923 to EL 918, although deeper removal may be required at some locations. The undercut excavation should be backfilled with well-graded granular fill as described in Section 5.3.

After rough grade has been established, the exposed subgrade should be carefully observed by an engineer or a qualified soils technician by probing and testing as needed. Any organic material still in place, frozen, wet, soft or loose soil, cohesive soil and other undesirable materials should be removed. The exposed subgrade should furthermore be evaluated by probing to check for pockets of soft or loose material beneath a thin crust of better soil. Any unsuitable materials thus exposed should be further removed and replaced with well-compacted, engineered fill as outlined in Section 5.3.

The soils on the sides and at the base of the excavation should be carefully examined and tested to determine whether all unsuitable materials have been completely removed. Any unsuitable materials should be further undercut. Once all of the unsuitable materials have been completely removed to expose firm natural soils, the excavations can then be backfilled.

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 at all possible.

5.3 Fill Compaction

All engineered fill beneath pavements, footings and slabs-on-grade should be compacted to a dry density of at least 98 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.

It is also recommended that only well-graded granular material, such as pit-run sand and gravel or INDOT No. 53 crushed limestone, should be used to fill undercut excavations below spread footings, floor slabs and any other excavations of limited lateral dimensions where proper compaction of cohesive materials is difficult and compaction can only be accomplished with small vibratory equipment. Lean concrete (2,000 lbs./sq.in. minimum compressive strength) can also be used as fill beneath spread footings.

5.4 Foundation Excavations

It is essential that the soil at the base of each spread footing excavation should be carefully observed, tested and evaluated by an engineer or a qualified geotechnical field technician working under the direction of the geotechnical engineer-of-record (ATC) to identify any unsuitable materials that must be removed and replaced and to verify that each spread footing will bear on firm natural soils or engineered fill that is placed over firm natural soils after first removing any unsuitable materials as described in Sections 4.1 and 4.2. All miscellaneous uncontrolled fill, utilities, soft cohesive soils, loose granular soils and any otherwise
undesirable materials must be removed at footing locations so that the footings will bear on satisfactory material compatible with the design of the footing. It appears likely that it will be necessary to remove and replace unsuitable materials at all spread footing locations on this project. At the time of such observation, 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. The necessary depth of penetration will be established by the geotechnical engineer or field technician.

Where undercutting is required to remove unsuitable materials the proposed spread footing bearing elevation may be re-established by backfilling after all undesirable materials have been removed. The undercut excavation beneath each spread footing should extend to suitable bearing soils as described in Sections 4.1 and 4.2.

All existing facilities (e.g., utilities, tunnels, pavements, etc.) should be suitably protected from undermining due to excavation for the new structure. Depending on the relative depths and locations of the new excavations and the need to remove unsuitable soils at footing locations, bracing or underpinning will likely be needed to protect some of the existing facilities. All federal, state and local safety regulations should be followed in this regard.

Soils exposed in the bases of all satisfactory foundation excavations should be protected against any detrimental change in condition such as from disturbance, rain and freezing. Surface run-off water should be drained away from the excavation 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 recommended that a concrete “mud mat” be placed at the bases of the footing excavations to protect the subgrade soils from deterioration due to seepage of water, construction activity, etc., and to aid in the proper placement of reinforcing steel.

5.5 Construction Dewatering

Significant dewatering measures will be required well in advance of making any excavations, including the basement excavation and excavations for non-basement foundations, in order to adequately depress the ground water level below the deepest excavations. No excavations should be initiated until it is demonstrated that the ground water has been sufficiently depressed well below the deepest excavation level. It will not be possible to pump water directly from the base of the excavation without causing deterioration of the subgrade soil or causing a quick condition in the base of the excavation. It will be necessary to depress the ground water level well in advance of excavation using a series of multiple wells or well-points and the ground water level must be maintained at least 3 ft below the deepest excavation level. Furthermore, ground water levels higher than those measured at the time of this investigation should be expected due to variations in the ground water level. The contractor should be prepared for variable ground water conditions, including cases as described above, and variable temporary dewatering conditions. It is recommended that an experienced specialty dewatering contractor be retained to provide temporary dewatering measures for the excavations at the basement level and non-basement foundation excavations.
6 FIELD INVESTIGATION

Eight test borings were drilled for this investigation at the approximate locations shown on the Boring Plan (Figure 2 in the Appendix). The test borings drilled for this study were extended to depths of 18.5 ft to 60.0 ft below the existing grade. One test boring that was drilled for a previous preliminary investigation at this site is also included in this study. Split-barrel samples were obtained by the Standard Penetration Test procedures (ASTM D-1586) at 2.5 ft to 5.0 ft intervals. The test borings were backfilled with auger cuttings and plugged with concrete at completion of drilling. Topsoil was placed over the concrete plug and leveled to the surrounding grade.

Logs of all test 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 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 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, grain size distribution tests, organic content tests (loss-on-ignition tests), marl content tests (Ca/MgCO₃ 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 and summary sheets 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 and bedrock 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 are solely for the purpose of planning the design of the proposed facilities. 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 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.

ATC assumes no responsibility for any construction procedures, temporary excavations (including utility trenches), temporary dewatering or site safety during or after construction. 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, streets, etc.).
Appendix

Figure 1: Vicinity Map
Figure 2: Boring Plan
Figure 3: Design Illustration – Lateral Earth Pressure Against Below-Grade Wall Assuming Undrained Backfill with Hydrostatic Pressure
Figure 4: Uplift Consideration of Submerged Below-Grade Structure

Test Boring Logs
“Field Classification System for Soil Exploration”
Unconfined Compressive Strength Test Report
Grain Size Distribution Test Reports
“Important Information About Your Geotechnical Engineering Report”
LATERAL EARTH PRESSURE AGAINST BELOW-GRADE WALL
ASSUMING UNDRAINED BACKFILL W/ HYDROSTATIC PRESSURE

PROPOSED EAST PARKING GARAGE
BALL STATE UNIVERSITY
NORTH NEW YORK AVENUE AND STUDEBAKER DRIVE
MUNCIE, INDIANA

\[ h_w = \text{DEPTH FROM DESIGN HIGH GROUND WATER LEVEL TO BASE OF WALL (ft.)} \]

\[ \bar{\gamma}_s = \text{SUBMERGED SOIL UNIT WEIGHT (lbs./cu. ft.)} \]

\[ \gamma_s = \text{TOTAL SOIL UNIT WEIGHT (lbs./cu.ft.)} \]

\[ \gamma_w = \text{UNIT WEIGHT OF WATER (lbs./cu. ft.)} \]

\[ K_0 = \text{COEFFICIENT OF AT-REST LATERAL EARTH PRESSURE} \]
\[ P_u = \gamma_w H \]

\[ U = \gamma_w HBL \]

**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.)
- \( \gamma_w \) = 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.)
- \( B \) = WIDTH OF STRUCTURE BASE (ft.)
- \( L \) = LENGTH OF STRUCTURE BASE (ft.)

**DESIGN ILLUSTRATION – UPLIFT CONSIDERATION OF SUBMERGED BELOW-GRADE STRUCTURE**

PROPOSED EAST PARKING GARAGE
BALL STATE UNIVERSITY
NORTH NEW YORK AVENUE AND STUDEBAKER DRIVE
MUNCIE, INDIANA
Ground surface elevation estimated from handheld GPS and Delaware County GIS mapping.

Sample No. 1:
- Organic content = 9.2%
- Ca/MgCO₃ content = 36%

Sample No. 2:
- Organic content = 20.8%
- Ca/MgCO₃ content = 16%

Sample No. 5:
- Finer than No. 200 sieve = 3.7%

Charged augers with water at 11.5 ft.
Cobbles encountered at 12 ft.

Sample No. 6:
- Unconfined compressive strength = 1.8 tsf
- Dry density = 121.3 pcf
Borehole backfilled with auger cuttings, plugged with concrete from -2 to -1 ft and covered with topsoil.

---

### SOIL CLASSIFICATION

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<th>Stratum Depth, ft</th>
<th>Depth, ft.</th>
<th>Sample No.</th>
<th>Sample Type</th>
<th>Sample Graphics</th>
<th>Sampler Graphics</th>
<th>Groundwater</th>
<th>Standard Penetration Test</th>
<th>Blows per 6 in. Increments</th>
<th>Moisture Content, %</th>
<th>Pocket Penetrometer Post</th>
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### Drill and Sampling Information

- Hollow Stem Augers
- Continuous Flight Augers
- Casing Advancer
- Mud Drilling
- Hand Auger

### Test Data

- **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: 3.0 ft.
  - At Completion: 6.4 ft.
  - After -- hours: -- ft.
  - Cave Depth: 12.8 ft.

---

### Test Boring Log

- **Client**: Champlin Architecture
- **Project Name**: Proposed East Parking Garage
- **Project Location**: Ball State University
  - N. New York Ave. & Studebaker Dr., Muncie, Indiana

- **Date Started**: 2/14/18
- **Hammer Wt.**: 140 lbs.
- **Date Completed**: 2/14/18
- **Hammer Drop**: 30 in.
- **Drill Foreman**: R. Zarobinski
- **Inspector**: S. Rushfeldt
- **Boring Method**: HSA

---

### Surface Elevation

- 927

---

### Remarks

- Ground surface elevation estimated from handheld GPS and Delaware County GIS mapping.

---

### Sample Log

- Sample No. 1
  - Organic content = 9.2%
  - Ca/MgCO₃ content = 36%

- Sample No. 2
  - Organic content = 20.8%
  - Ca/MgCO₃ content = 16%

- Sample No. 5
  - Finer than No. 200 sieve = 3.7%

- Charged augers with water at 11.5 ft.

- Sample No. 6
  - Unconfined compressive strength = 1.8 tsf
  - Dry density = 121.3 pcf

- Borehole backfilled with auger cuttings, plugged with concrete from -2 to -1 ft and covered with topsoil.
# TEST BORING LOG

**CLIENT** Champlin Architecture  
**PROJECT NAME** Proposed East Parking Garage  
**PROJECT LOCATION** Ball State University, N. New York Ave. & Studebaker Dr., Muncie, Indiana

**DATE** 2/14/18  
**HOUR**  
**WEIGHT** 140 lbs.

**SOIL CLASSIFICATION**

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<tr>
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**TEST DATA**

- **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**
  - 3.0 ft.
  - 6.5 ft.
  - -- ft.
  - 13.1 ft.

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

---

**Ground surface elevation estimated from handheld GPS and Delaware County GIS mapping.**

**Sample No. 1:**
- Organic content = 15.9%
- Ca/MgCO3 content = 22%
- Cobbles encountered at 8 ft.

**Sample No. 6:**
- Finer than No. 200 sieve = 7.3%
- Cobbles encountered at 16 ft.
- Borehole backfilled with auger cuttings, plugged with concrete from -2 to -1 ft and covered with topsoil.
# Groundwater Remarks

- **Sample No. 1:**
  - Organic content = 4.3%
  - Ca/MgCO3 content = 8%

- **Sample No. 3:**
  - LL=22, PL=13, PI=9

- Borehole backfilled with auger cuttings, plugged with concrete from -2 to -1 ft and covered with topsoil.

- Bottom of Test Boring at 29.3 ft

---

**SOIL CLASSIFICATION**

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- Ground surface elevation estimated from handheld GPS and Delaware County GIS mapping.

- Organic content = 4.3%
- Ca/MgCO3 content = 8%
- Sample No. 3:
  - LL=22, PL=13, PI=9

---

**Surface Elevation 930**

- **Drill Foreman:** R. Zarobinski
- **Inspector:** S. Rushfeldt
- **Boring Method:** HSA
- **Spoon Sampler OD:** 2.0 in.
- **Shelby Tube OD:** -- in.

---

**Depth to Groundwater**

- **Note on Drilling Tools:** 13.0 ft.
- **At Completion:** 7.0 ft.
- **After -- hours:** -- ft.
- **Cave Depth:** 7.0 ft.
### SOIL CLASSIFICATION

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<td>SS</td>
<td>4-10-11</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>6</td>
<td>SS</td>
<td>SS</td>
<td>10-21-24</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>7</td>
<td>SS</td>
<td>SS</td>
<td>9-17-23</td>
<td></td>
</tr>
<tr>
<td>909.0</td>
<td>20.0</td>
<td>8</td>
<td>SS</td>
<td>SS</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

- **11 in. Topsoil**
  - Black, moist, very loose, CLAYEY SAND (SC) with marl, wood fragments, organics and trace gravel

- **Gray, wet, loose to dense, SAND and GRAVEL (SW-SM) with trace silt**
  - sandy silt seam at 14 ft

- **Bottom of Test Boring at 20.0 ft**

### Remarks
- Ground surface elevation estimated from handheld GPS and Delaware County GIS mapping.
- Sample No. 1:
  - Organic content = 13.6%
  - Ca/MgCO3 content = 28%
- Sample No. 2:
  - Organic content = 7.3%
  - Ca/MgCO3 content = 47%

- Cobbles encountered at 11.5 ft.
- Cobbles encountered at 16 ft.
- Borehole backfilled with auger cuttings, plugged with concrete from -2 to -1 ft and covered with topsoil.

### Test Data

<table>
<thead>
<tr>
<th>Sample Type</th>
<th>Depth to Groundwater</th>
<th>Boring Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>SS</td>
<td>6.0 ft.</td>
<td>HSA - Hollow Stem Augers</td>
</tr>
<tr>
<td>ST</td>
<td>6.5 ft.</td>
<td>CFA - Continuous Flight Augers</td>
</tr>
<tr>
<td>CA</td>
<td>10.1 ft.</td>
<td>HA - Hand Auger</td>
</tr>
<tr>
<td>RC</td>
<td></td>
<td>MD - Mud Drilling</td>
</tr>
<tr>
<td>CU</td>
<td></td>
<td>CA - Casing Advancer</td>
</tr>
<tr>
<td>CT</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

- **Note on Drilling Tools**: Noted on Drilling Tools
- **At Completion**: 6.0 ft.
- **After**: 6.5 ft.
- **Cave Depth**: 10.1 ft.
<table>
<thead>
<tr>
<th>SOIL CLASSIFICATION</th>
<th>SURFACE ELEVATION 929</th>
</tr>
</thead>
<tbody>
<tr>
<td>10 in. Topsoil</td>
<td>928.2 0.8</td>
</tr>
<tr>
<td>Black, moist, soft, Silty Clay (CL) with trace organics</td>
<td>923.0 6.0</td>
</tr>
<tr>
<td>Gray, wet, loose to very dense, Sand and Gravel (SW-SM) with trace silt</td>
<td>901.0 28.0</td>
</tr>
<tr>
<td>Gray, moist, hard, Sandy Clay (CL) with trace gravel</td>
<td>899.0 30.0</td>
</tr>
</tbody>
</table>

Bottom of Test Boring at 30.0 ft

<table>
<thead>
<tr>
<th>Depth to Groundwater</th>
<th>Boring Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>Noted on Drilling Tools 6.0 ft.</td>
<td>HSA - Hollow Stem Augers</td>
</tr>
<tr>
<td>At Completion 7.0 ft.</td>
<td>CFA - Continuous Flight Augers</td>
</tr>
<tr>
<td>After 9.5 ft.</td>
<td>CA - Casing Advancer</td>
</tr>
<tr>
<td>Cave Depth 9.5 ft.</td>
<td>MD - Mud Drilling</td>
</tr>
<tr>
<td></td>
<td>HA - Hand Auger</td>
</tr>
</tbody>
</table>
## Drill and Sampling Information

<table>
<thead>
<tr>
<th>Stratum / Depth</th>
<th>Stratum Elevation</th>
<th>Sample Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0 ft.</td>
<td>931.0</td>
<td>SS</td>
</tr>
<tr>
<td>0.9 ft.</td>
<td>930.1</td>
<td>SS</td>
</tr>
<tr>
<td>3.0 ft.</td>
<td>928.0</td>
<td>SS</td>
</tr>
<tr>
<td>6.0 ft.</td>
<td>925.0</td>
<td>SS</td>
</tr>
<tr>
<td>8.5 ft.</td>
<td>922.5</td>
<td>SS</td>
</tr>
<tr>
<td>11.0 ft.</td>
<td>920.0</td>
<td>SS</td>
</tr>
<tr>
<td>16.0 ft.</td>
<td>915.0</td>
<td>SS</td>
</tr>
<tr>
<td>18.5 ft.</td>
<td>912.5</td>
<td>SS</td>
</tr>
<tr>
<td>29.0 ft.</td>
<td>900.0</td>
<td>SS</td>
</tr>
<tr>
<td>33.5 ft.</td>
<td>897.5</td>
<td>SS</td>
</tr>
<tr>
<td>40.0 ft.</td>
<td>891.0</td>
<td>SS</td>
</tr>
</tbody>
</table>

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

**Remarks:**
- Ground surface elevation estimated from handheld GPS and Delaware County GIS mapping.
- Organic content = 4.1%
- Ca/MgCO3 content = 6%
- Organic content = 1.3%
- Ca/MgCO3 content = 34%
- Charged augers with water at 11 ft.
- Borehole backfilled with auger cuttings, plugged with concrete from -2 to -1 ft and covered with topsoil.
- Cobble encountered at 21 ft.
**TEST BORING LOG**

**CLIENT** Champlin Architecture  
**PROJECT NAME** Proposed East Parking Garage  
**PROJECT LOCATION** Ball State University  
**JOB #** 170GC00601

**N. New York Ave. & Studebaker Dr., Muncie, Indiana**

<table>
<thead>
<tr>
<th>Soil Classification</th>
<th>Stratum Elevation</th>
<th>Stratum Depth, ft</th>
<th>Depth, Scale, ft</th>
<th>Sample No.</th>
<th>Sample Type</th>
<th>Standard Penetration Test, Blows per 6 in. increments</th>
<th>Moisture Content, %</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Gray, wet, dense, GRAVEL (GW-GM) with trace sand and silt</td>
<td>882.5</td>
<td>48.5</td>
<td>13</td>
<td>SS</td>
<td>13-17-18</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gray, wet, very dense, SILTY SAND (SM) with little gravel</td>
<td>881.0</td>
<td>50.0</td>
<td>14</td>
<td>SS</td>
<td>22-23-30</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gray, slightly moist, hard, SANDY CLAY (CL) with trace gravel</td>
<td>871.0</td>
<td>60.0</td>
<td>16</td>
<td>SS</td>
<td>24-29-33</td>
<td>9.2</td>
<td>4.5+</td>
<td></td>
</tr>
</tbody>
</table>

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

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

---

**Champlin Architecture**
7988 Centerpoint Drive, Suite 100  
Indianapolis, IN 46256  
(317) 849-4990  
Fax (317) 849-4278
**Ground surface elevation estimated from handheld GPS and Delaware County GIS mapping.**

**Sample No. 1:**
- Organic content = 5.4%
- Ca/MgCO3 content = 12%

**Sample No. 2:**
- Organic content = 8.6%
- Ca/MgCO3 content = 6%

**Sample No. 4:**
- Finer than No. 200 sieve = 8.4%

- Charged augers with water at 9 ft.

**Sample No. 7:**
- Finer than No. 200 sieve = 11.9%

**Borehole backfilled with auger cuttings, plugged with concrete from -2 to -1 ft and covered with topsoil.**

### Soil Classification

<table>
<thead>
<tr>
<th>Stratum Elevation</th>
<th>Stratum Depth, ft</th>
<th>Depth Scale, ft</th>
<th>Sample No.</th>
<th>Sample Type</th>
<th>Standard Penetration Test, Blows per 6 in. Increments</th>
<th>Moisture Content, %</th>
<th>Pocket Penetrometer</th>
</tr>
</thead>
<tbody>
<tr>
<td>10.8 in. Topsoil</td>
<td>929.1</td>
<td>0.9</td>
<td>1</td>
<td>SS</td>
<td>1-2-2</td>
<td>57.5</td>
<td>0.25</td>
</tr>
<tr>
<td>Dark gray, moist, soft to very soft, SILTY CLAY (CL) with trace organics and marl</td>
<td>923.0</td>
<td>7.0</td>
<td>2</td>
<td>SS</td>
<td>2-1-2</td>
<td>50.4</td>
<td>&lt;0.25</td>
</tr>
<tr>
<td>Brown, wet, medium dense, SAND (SP-SM) with trace gravel and silt</td>
<td>916.5</td>
<td>13.5</td>
<td>3</td>
<td>SS</td>
<td>2-3-6</td>
<td>8-5-10</td>
<td></td>
</tr>
<tr>
<td>Brown and gray, wet, medium dense, GRAVEL (GP-GM) with little sand and trace silt</td>
<td>914.0</td>
<td>16.0</td>
<td>4</td>
<td>SS</td>
<td>8-9-10</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gray, wet, medium dense, SAND and GRAVEL (SP-SM) with little silt</td>
<td>910.0</td>
<td>20.0</td>
<td>8</td>
<td>SS</td>
<td>8-12-14</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Depth to Groundwater**

- Noted on Drilling Tools: **6.5 ft.**
- At Completion: **7.0 ft.**
- After **--** hours: **-- ft.**
- Cave Depth: **8.0 ft.**

**Boring Method**

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

<table>
<thead>
<tr>
<th>Stratum Elevation</th>
<th>Stratum Depth, ft</th>
<th>Depth Scale, ft</th>
<th>Sample No.</th>
<th>Sample Type</th>
<th>Sample Graphics</th>
<th>Stratum Characterization</th>
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</thead>
<tbody>
<tr>
<td>9.28 ft.</td>
<td>0.8</td>
<td>928.2</td>
<td>1</td>
<td>SS</td>
<td>X</td>
<td>9 in. Topsoil — Brown, very moist, soft, SANDY SILTY CLAY (CL) and marl with trace gravel and root material</td>
</tr>
<tr>
<td>9.25 ft.</td>
<td>3.5</td>
<td>925.5</td>
<td>2</td>
<td>SS</td>
<td>X</td>
<td>3-6-8 Groundwater Remarks — Charged augers with water at 9 ft.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3</td>
<td>SS</td>
<td>X</td>
<td>Groundwater Remarks — Charged augers with water at 9 ft.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>4</td>
<td>SS</td>
<td>X</td>
<td>Groundwater Remarks — Charged augers with water at 9 ft.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>5</td>
<td>SS</td>
<td>X</td>
<td>Groundwater Remarks — Charged augers with water at 9 ft.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>6</td>
<td>SS</td>
<td>X</td>
<td>Groundwater Remarks — Charged augers with water at 9 ft.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>7</td>
<td>SS</td>
<td>X</td>
<td>Groundwater Remarks — Charged augers with water at 9 ft.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>8</td>
<td>SS</td>
<td>X</td>
<td>Groundwater Remarks — Charged augers with water at 9 ft.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>9</td>
<td>SS</td>
<td>X</td>
<td>Groundwater Remarks — Charged augers with water at 9 ft.</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>10</td>
<td>SS</td>
<td>X</td>
<td>Groundwater Remarks — Charged augers with water at 9 ft.</td>
</tr>
</tbody>
</table>

**Groundwater Remarks:**
- Charged augers with water at 9 ft.

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

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

**Remarks:**
- Ground surface elevation estimated from handheld GPS and Delaware County GIS mapping.
- Sample No. 1:
  - Organic content = 1.8%
  - Ca/MgCO3 content = 40%
- Charged augers with water at 9 ft.
- Borehole backfilled with auger cuttings, plugged with concrete from -2 to -1 ft and covered with topsoil.

**Sample Information:**
- Date Started: 2/15/18
- Date Completed: 2/15/18
- Hammer Wt.: 140 lbs.
- Hammer Drop: 30 in.
- Spoon Sampler OD: 2.0 in.
- Rock Core Dia.: -- in.
- Shelby Tube OD: -- in.

**Project Information:**
- Client: Champlin Architecture
- Project Name: Proposed East Parking Garage
- Project Location: Ball State University, N. New York Ave. & Studebaker Dr., Muncie, Indiana
- Job #: 170GC00601

---

**Test Boring Log**

---

**Client:** Champlin Architecture  
**Project Name:** Proposed East Parking Garage  
**Project Location:** Ball State University, N. New York Ave. & Studebaker Dr., Muncie, Indiana  
**Job #:** 170GC00601
Ball State University  

Proposed Future Parking Garage Site  

Riverside Avenue and New York Avenue  

Muncie, Indiana

DRILLING and SAMPLING INFORMATION

Date Started: 4/24/17  
Date Completed: 4/24/17  
Drill Foreman: Z. Vaughan  
Inspector: S. Rushfeldt  
Boring Method: HSA

TEST DATA

Ground surface elevation estimated from Google Earth.  
Sample No. 1:  
Organic Content = 10.2%

Sample No. 7 was not obtained due to augering through boulder.  
Auger refusal on apparent boulder at 18.5 ft.

SOIL CLASSIFICATION

<table>
<thead>
<tr>
<th>Stratum</th>
<th>Stratum Depth, ft</th>
<th>Sample No.</th>
<th>Sample Type</th>
<th>Moisture Content, %</th>
</tr>
</thead>
<tbody>
<tr>
<td>929.7</td>
<td>0.3</td>
<td>1</td>
<td>SS</td>
<td>4-4-5</td>
</tr>
<tr>
<td>926.5</td>
<td>3.5</td>
<td>2</td>
<td>SS</td>
<td>20.9</td>
</tr>
<tr>
<td>919.0</td>
<td>11.0</td>
<td>3</td>
<td>SS</td>
<td>6-8-9</td>
</tr>
<tr>
<td>911.5</td>
<td>18.5</td>
<td>4</td>
<td>SS</td>
<td>8-10-10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5</td>
<td>SS</td>
<td>8-20-30</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6</td>
<td>SS</td>
<td>11-14-17</td>
</tr>
<tr>
<td></td>
<td></td>
<td>7</td>
<td>SS</td>
<td>17-26-31</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8</td>
<td>SS</td>
<td>50/0</td>
</tr>
</tbody>
</table>

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:  
- 3.5 ft.  
- 3.0 ft.  
- -- ft.  
- 4.0 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)

<table>
<thead>
<tr>
<th>Density</th>
<th>Particle Size Identification</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Loose</td>
<td>Boulders - 8 inch diameter or more</td>
</tr>
<tr>
<td>Loose</td>
<td>Cobble - 3 to 8 inch diameter</td>
</tr>
<tr>
<td>Medium Dense</td>
<td>Gravel - Coarse - 1 to 3 inch</td>
</tr>
<tr>
<td>Dense</td>
<td>Gravel - Coarse - 1 to 3 inch</td>
</tr>
<tr>
<td>Very Dense</td>
<td>Sand - Coarse 2.00mm to ¼ inch</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Relative Proportions</th>
<th>Descriptive Term</th>
<th>Percent</th>
</tr>
</thead>
<tbody>
<tr>
<td>Medium</td>
<td>(dia. of broom straw)</td>
<td></td>
</tr>
<tr>
<td>Trace</td>
<td>1 - 10</td>
<td></td>
</tr>
<tr>
<td>Little</td>
<td>11 - 20</td>
<td></td>
</tr>
<tr>
<td>Some</td>
<td>21 - 35</td>
<td></td>
</tr>
<tr>
<td>And</td>
<td>36 - 50</td>
<td></td>
</tr>
</tbody>
</table>

COHESIVE SOILS
(Clay, Silt and Combinations)

<table>
<thead>
<tr>
<th>Consistency</th>
<th>Plasticity</th>
<th>Degree of Plasticity</th>
<th>Plasticity Index</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very Soft</td>
<td>Degree of Plasticity</td>
<td>None to slight</td>
<td>0 - 4</td>
</tr>
<tr>
<td>Soft</td>
<td>Degree of Plasticity</td>
<td>Slight</td>
<td>5 - 7</td>
</tr>
<tr>
<td>Medium Stiff</td>
<td>Degree of Plasticity</td>
<td>Medium</td>
<td>8 - 22</td>
</tr>
<tr>
<td>Stiff</td>
<td>Degree of Plasticity</td>
<td>High to Very High</td>
<td>over 22</td>
</tr>
<tr>
<td>Very Stiff</td>
<td>Degree of Plasticity</td>
<td>High to Very High</td>
<td>over 22</td>
</tr>
<tr>
<td>Hard</td>
<td>Degree of Plasticity</td>
<td>High to Very High</td>
<td>over 22</td>
</tr>
</tbody>
</table>

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.
### UNCONFINED COMPRESSION TEST

#### Sample No.

<table>
<thead>
<tr>
<th>Sample No.</th>
<th>Unconfined strength, psi</th>
<th>Undrained shear strength, psi</th>
<th>Failure strain, %</th>
<th>Strain rate, in./min.</th>
<th>Water content, %</th>
<th>Wet density, pcf</th>
<th>Dry density, pcf</th>
<th>Saturation, %</th>
<th>Void ratio</th>
<th>Specimen diameter, in.</th>
<th>Specimen height, in.</th>
<th>Height/diameter ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>24.54</td>
<td>12.27</td>
<td>15.0</td>
<td>2.00</td>
<td>14.4</td>
<td>138.8</td>
<td>121.3</td>
<td>99.9</td>
<td>0.3898</td>
<td>1.53</td>
<td>3.04</td>
<td>1.98</td>
</tr>
</tbody>
</table>

#### Description:

<table>
<thead>
<tr>
<th>LL =</th>
<th>PL =</th>
<th>PI =</th>
<th>Assumed GS= 2.7</th>
<th>Type: Split spoon</th>
</tr>
</thead>
</table>

#### Project No.: 170GC00601

#### Date Sampled:

#### Remarks:

#### Figure QU9884D

#### Client: Champlin Architecture

#### Project: East Parking Garage

#### Source of Sample: 9884 Depth: 13.5-15’

#### Sample Number: B-101, #6

---

**ATC Group Services LLC**
Indianapolis, Indiana
**Particle Size Distribution Report**

**Material Description**
Gray Sand and Gravel with trace Silt

**Atterberg Limits**
- PL = NP
- LL = NP
- PI = NP

**Coefficients**
- $D_{90} = 27.6879$
- $D_{85} = 22.0764$
- $D_{50} = 17.37$
- $D_{30} = 1.4881$
- $D_{15} = 0.5611$
- $C_u = 17.37$
- $C_c = 0.85$

**Classification**
AASHTO=

**Remarks**

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* (no specification provided)

**Source of Sample:** 09884  
**Depth:** 11.0'-12.5'  
**Sample Number:** B-101; S-5  
**Date:**

---

**ATC Group Services LLC**  
**Indianapolis, Indiana**

**Client:** Champlin Architecture  
**Project:** East Parking Garage  
**Project No:** 170GC00601  
**Figure**
# Particle Size Distribution Report

**Material Description**
Gray Sand with trace Gravel and Silt

**Atterberg Limits**
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<tr>
<th>PL</th>
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<th>PI</th>
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<tbody>
<tr>
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**Coefficients**
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**Classification**
USCS = SW-SM  AASHTO =

**Remarks**

---

**Sieve Analysis**

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**Source of Sample:** 09884  **Depth:** 13.5'-15.0'
**Sample Number:** B-102; S-6

---

**Client:** Champlin Architecture  **Project:** East Parking Garage
**Project No:** 170GC00601  **Figure**
Gray Sand and Gravel with trace Silt

PL = NP 
LL = NP 
P1 = NP

D90 = 16.6661 
D50 = 13.5704 
D30 = 4.1655

Classification: SW-SM

Remarks:

Source of Sample: 09884 
Depth: 8.5'-10.0'
Sample Number: B-105; S-4

Client: Champlin Architecture
Project: East Parking Garage
Project No: 170GC00601

Figure
### Particle Size Distribution Report

**Material Description**
Brown Sand with trace Gravel and Silt

**Atterberg Limits**
\[ \begin{align*}
\text{PL}= & \quad \text{NP} \\
\text{LL}= & \quad \text{NP} \\
\text{Pl}= & \quad \text{NP}
\end{align*} \]

**Coefficients**
\[ \begin{align*}
D_{90} &= 3.4159 \\
D_{85} &= 2.7950 \\
D_{50} &= 1.1875 \\
D_{10} &= 0.1045 \\
C_u &= 14.48 \\
C_c &= 0.83
\end{align*} \]

**Classification**
SP-SM

**Remarks**

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**Source of Sample:** 09885  
**Depth:** 8.5'-10.0'  
**Sample Number:** B-107; S-4  
**Client:** Champlin Architecture  
**Project:** East Parking Garage  
**Project No:** 170GC00601  
**Date:**
Particle Size Distribution Report

Material Description
Gray Sand and Gravel with trace Silt

<table>
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<th>SPEC.*</th>
<th>PASS? (X=NO)</th>
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* (no specification provided)

| Source of Sample: 09885 | Depth: 16.0'-17.5' | Date: |
| Sample Number: B-107; S-7 | |

ATC Group Services LLC
Indianapolis, Indiana

Client: Champlin Architecture
Project: East Parking Garage
Project No: 170GC00601
Figure
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-engineering 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.