REPORT OF

SUBSURFACE EXPLORATION AND
GEOTECHNICAL ENGINEERING ANALYSIS

UNIVERSITY OF THE DISTRICT OF COLUMBIA
STUDENT UNION BUILDING
WASHINGTON, DC

ECS PROJECT NO. 01:17393

FOR

CANNON DESIGN

MAY 3, 2011

May 3, 2011

Ms. Patricia Bou
Cannon Design
1100 Wilson Boulevard
Suite 2900
Arlington, Virginia 22209

ECS Project No. 01:17393

Reference: Report of Subsurface Exploration and Geotechnical Engineering, University of the District of Columbia Student Union Building, Connecticut Avenue, NW and Van Ness Street, NW, Washington, DC

Dear Ms. Bou:

As authorized by your acceptance of ECS Proposal No. 01:35733-GPR, with revised date of November 3, 2010, ECS Mid-Atlantic, LLC (ECS) has completed the subsurface exploration and geotechnical engineering analysis for the proposed Student Union Building for the University of the District of Columbia located at the intersection of Connecticut and Van ness Streets in Northwest Washington, DC.

A report, including the results of our subsurface exploration, boring data, laboratory testing, engineering recommendations, and a Boring Location Diagram are enclosed herein. The recommendations presented are intended for use by your office and for use by other professionals involved in the design and construction stages of the project described herein.

We appreciate the opportunity to be of service to Cannon Design on this project. If you have any questions with regard to the information and recommendations contained in this report, or if we may be of further service to you during the planning and/or construction phase of this project, please do not hesitate to contact the undersigned.

Respectfully,

ECS MID-ATLANTIC, LLC

DISTRICT OF COLUMBIA
MANOL ANDONYADIS
905320
LICENSED
PROFESSIONAL ENGINEER

Steve Adamchak
Staff Project Manager

Manol P. Andonyadis, P.E., LEED AP
Senior Principal Engineer

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REPORT

PROJECT

Subsurface Exploration and
Geotechnical Engineering Analysis
University of the District of Columbia
Student Union Building
Washington, DC

CLIENT

Cannon Design
1100 Wilson Boulevard
Suite 2900
Arlington, Virginia 22209

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DATE May 3, 2011

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PROJECT OVERVIEW

Introduction

This report presents the results of our subsurface exploration and geotechnical engineering analysis performed for the proposed Student Union Building for the University of the District of Columbia located at the intersection of Connecticut Avenue and Van Ness Street in Northwest Washington, DC. This study was conducted in general accordance with ECS Proposal No. 35733-GPR, with a revised date of November 3, 2011, and authorized by your office. In preparing this report, we have utilized information from our current subsurface exploration as well as information from nearby sites.

Site Location and Existing Site Conditions

The project site is located at the northwest quadrant of the intersection of Connecticut Avenue, NW and Van Ness Street, NW in Washington, DC. The site is bound to the north by a University of the District of Columbia access road, to the south by Van Ness Street, NW, to the east by Connecticut Avenue, NW, and to the west by existing University of the District of Columbia structures. An existing subsurface Washington Metropolitan Area Transit Authority (WMATA) facility is located east of and immediately adjacent to the proposed site. Based on a topographic survey prepared by Delon Hampton and Associates and provided by Cannon Design on January 31, 2011, existing site grades range from approximately EL. +256 feet in the north portion of the site to EL. +272 feet in the southwest portion of the site. The site is currently developed as a plaza area with brick pavers and trees in planters in the majority of the site with landscaped areas in the southern portion of the site.

Proposed Construction

Based on the conceptual drawings available at the time that this report was prepared as well as our recent conversations with the design team, the proposed development will consist of a three-story above-grade structure with one partial cellar level in the western portion of the structure. The structure will either be supported with cast-in-place concrete or structural steel members with maximum column loads on the order of 900 or 600 kips, respectively. The proposed finished floor elevations (FFE) for the cellar level and at-grade level (i.e. Level 1) are EL. +247.5 feet and EL. +261.5 feet, respectively.

Purpose and Scope of Work

The purpose of this exploration was to explore the subsurface conditions at the site and to develop engineering recommendations to guide the design and construction of the project. We accomplished these purposes by performing the following scope of services:

1. Reviewing the geotechnical reports prepared for nearby project sites by ECS;
2. Drilling soil borings using a conventional drill rig;
3. Performing laboratory tests on selected representative soil samples from the borings to evaluate pertinent engineering properties;
4. Analyzing the field and laboratory data from this exploration to develop appropriate engineering recommendations; and,
5. Preparing this geotechnical report of our findings and recommendations.

The conclusions and recommendations contained in this report are based on five (5) soil borings (Borings B-1 through B-5), conducted by ECS at the project site. Each of the borings was drilled within the limits of the proposed development to depths of approximately 34 feet to 45.5 feet below existing site grades, which corresponds to termination elevations of EL. +218 feet to EL. +226 feet. The subsurface exploration included split spoon soil sampling, rock coring, Standard Penetration Tests (SPT) and groundwater level observations in the boreholes. The results of the completed soil borings along with a Boring Location Diagram are included in the Appendix of this report.

The Boring Location Diagram was developed from the topographic survey drawing prepared by Delon Hampton and Associates, provided by Cannon Design on January 31, 2011. The boring locations were located in the field by a representative of ECS by measuring from existing site features. The boring locations in the field are considered to be within approximately 3 feet of the plan location.

EXPLORATION PROCEDURES

Subsurface Exploration Procedures

The soil borings were performed utilizing a truck-mounted auger-drilling rig, which uses continuous flight, hollow stem augers to advance the borehole. Drilling fluid was not used in this process. After completion of the borings, the boreholes were backfilled with the auger spoils generated during the drilling process.

Representative soil samples were obtained by means of the split-barrel sampling procedure in accordance with ASTM Specification D-1586. In this procedure, a 2-inch O.D., split-barrel sampler is driven into the soil a distance of either 18 inches or 24 inches by a 140-pound hammer falling 30 inches. Sampling was typically performed at 2.5 foot intervals in the upper 10 feet and every 5 feet thereafter. The number of blows required to drive the sampler through the last 12-inch interval is termed the Standard Penetration Test (SPT) N-value and is indicated for each sample on the boring logs. This value can be used as a qualitative indication of the in-place relative density of cohesionless soils. In a less reliable way, it also indicates the consistency of cohesive soils.

At boring location B-1, following drilling operations to auger refusal, rock samples were obtained in accordance with ASTM D-2113, using a diamond studded bit fastened to the end of a wire-line hollow tube core barrel. The core barrel was drilled into the rock up to five feet at a time and the samples were removed for measurement of sample recovery. The recovery is determined as the ratio of sample length recovered to the distance drilled. The core samples were stored in boxes and returned to our laboratory for identification and determination of the Rock Quality Designation (RQD). The RQD is determined as the ratio of intact rock in NQ core sections 4 inches or longer to the distance drilled. Percentages of recovery and RQD are given on the boring log included in the Appendix of this report.

The drill crew maintained a field log of the soils/rock encountered in the borings. After recovery, each sample was removed from the sampler and visually classified. Representative portions of each sample were then sealed and brought to our laboratory in Chantilly, Virginia for further visual examination and laboratory testing.

Laboratory Testing Program

Representative soil samples were selected and tested in our laboratory to check field classifications and to determine pertinent engineering properties. The laboratory testing program included visual classifications, moisture content tests, Atterberg Limits tests, and grain size distribution analysis tests. All data obtained from the laboratory tests is included in the Appendix of this report.

Each soil sample was classified on the basis of texture and plasticity in accordance with the Unified Soil Classification System. The group symbols for each soil type are indicated in parentheses following the soil descriptions on the boring logs. A brief explanation of the Unified System is included with this report. In addition, each rock core run was classified based on percent recovery (REC), Rock Quality Designation (RQD), weathering, discontinuity spacing, and hardness. The various soil / rock types were grouped into the major zones noted on the boring logs. The stratification lines designating the interfaces between earth materials, intermediate geo-materials, and rock on the boring logs and profiles are approximate; in situ, the transitions may be gradual, rather than distinct.

The soil samples will be retained in our laboratory for a period of 60 days, after which they will be discarded unless other instructions are received as to their disposition.

EXPLORATION RESULTS

Regional Geology

The site is located within the Piedmont Physiographic Province of Washington, DC The Piedmont is generally characterized as a gently rolling erosional surface underlain by Proterozoic and Palezoic igneous and metamorphic rocks. The Piedmont consists of residual soils which are predominately fine sandy silts and silty fine sands with mica which have developed in-place from the chemical and physical weathering of the underlying predominately micaceous schist and gneiss bedrock. The weathering occurs in an irregular fashion and creates a zone of decomposed, weathered rock which can possess rock-like qualities that can extend to a significant depth. The decomposed, weathered rock thickness can be highly variable in the Bethesda area over short horizontal distances.

Underlying the weathered rock is the parent bedrock material identified as the Wissahickon Formation. The upper zone of the Wissahickon Rock Formation at times consists of a thick, weak, mantle of rock termed poor quality or weathered rock on our boring logs.

The upper residual natural Silty SAND / Sandy SILT (SM/ML) soils are believed to be residual soil materials derived from the in-place weathering of the underlying parent bedrock. The transition zone between soil and rock is termed "Weathered Rock" on our boring logs. For the purposes of this report, Standard Penetration Test (SPT) N-values (during drilling and sampling) were used to discern the differing strata. The following penetration values were used to discern between soil, weathered rock, and rock:

- Stratum I - Fill Soils - SPT N-Values vary;
- Stratum II - Natural Residual Soils (SM/ML): SPT N-Values typically range between 4 to 22 blows per foot (bpf) with isolated areas of higher N-Values;
- Stratum III - Weathered SCHIST or GNEISS (Weathered Rock): The Weathered Rock is a "transition material" between the upper Residual Soil and lower Competent Rock. Typical SPT N-Values range between 98 bpf to more than 50 blows per 3 inches but some SPT N-Values may be somewhat denser (50 blows per 1 to 2 inches) or less dense (50 blows per 6 inches).
- Stratum IV - Schistic Gneiss (Competent Rock): Corable rock with Rock Recoveries of at least 75% sampled as Schistic Gneiss.

Soil Conditions

Ground cover at the project site, at the time of our field exploration program consisted of a thin layer of topsoil. Underlying the surficial materials, the subsurface profile can be subdivided into four different and distinct strata: (I) Fill, (II) Natural Residual Soils, (III) Weathered Rock, and (IV) Competent Rock. The following sections describe each soil strata in more detail.

Stratum I - Fill

Fill soils were observed at each boring location to depths ranging between 2 to 17 feet below the existing site grades (EL. +266 feet to EL. +243 feet). The Fill soils typically consisted of granular soils with varying amounts of sand, silt, clay and gravel. SPT N-Values in this material typically ranged between 3 bpf and 22 bpf, which is indicative of very loose to medium dense relative densities for the granular soils.

Stratum II – Natural Residual Soils

Natural Residual Soils were observed below the Stratum I - Fill soils. The Natural Residual Soils consisted of Micaceous Sandy SILT and Silty SAND (SM/ML). These soils were observed to extend to depths on the order of 7 to 32 feet below the existing site grades (EL. +228 feet to EL. +261 feet). SPT N-Values within this stratum typically ranged between 4 bpf and 38 bpf, which is indicative of very loose to dense relative densities.

Stratum III – Highly Weathered Rock

Weathered Rock was observed below the Stratum II - Natural Residual Soils. The Weathered Rock was typically sampled as Micaceous Silty SAND with rock fragments. These soils were observed to extend to a depth of 24.5 to 45 feet below existing site grades (EL. +245.5 feet to 218 feet). SPT N-values in this stratum ranged between 98 bpf to 50 blows over less than 1-inch of penetration.

Stratum IV – Competent Rock

Competent Rock was observed below the Stratum III - Weathered Rock. Competent Rock was encountered at the boring termination depth of Borings B-2 through B-5. At boring location B-1, Competent Rock was sampled using an NX core barrel and consisted of Schistic GNEISS. The competent rock recovered in the core barrels was typically weathered, soft and highly fractured.

Groundwater Observations

In auger drilling operations, water is not introduced into the boreholes, and the groundwater position can often be determined by observing water flowing into or out of the borings. Furthermore, visual observation of the soil samples retrieved during the auger drilling exploration can often be used in evaluating the groundwater conditions. Groundwater observations were made while drilling, after boring but before the augers were removed, and after the augers were removed prior to backfilling.

Based on the groundwater depths observed during the subsurface exploration, the groundwater level is approximately 14.5 feet to 33 feet below existing site grades. These depths correspond to groundwater elevations on the order of EL. +230 feet to EL. +245.5 feet, which is near the finished floor elevation of the cellar level, EL. +247.5 feet.

The highest groundwater observations are normally encountered in late winter and early spring and our current groundwater observations are not expected to be at the seasonal maximum water table. Variations in the location of the long-term water table may occur as a result of changes in precipitation, evaporation, surface water runoff, and other factors not immediately apparent at the time of our explorations.

ANALYSIS AND RECOMMENDATIONS

The conclusions and recommendations presented in this report should be incorpriated in the design and construction of the project to reduce possible soil and/or foundation related problems.

Based on our understanding of the proposed construction and the results of our subsurface exploration, the following key geotechnical issues were identified and analyzed to develop geotechnical recommendations for the proposed student union building and the associated site features:

1. The maximum column loads within the three-story structure of 900 kips;
2. The compressible nature of the undocumented fill material observed within the upper 2 to 17 feet of the soil borings performed within the footprint of the proposed structure;
3. The variability observed in the elevation of the top of the weathered rock stratum (i.e. EL. + 261 feet at boring location B-1 and EL. +227 feet at boring location B-2);
4. The load carrying capacity of the underlying residual soils, weathered rock, and competent rock materials;
5. The constructability of the foundation elements given the estimated design lengths;
6. The location/proximity of the subsurface WMATA structure's zone of influence within the proposed building footprint; and
7. The accelerated construction schedule for this project.

Both shallow and deep foundation systems were evaluated for support of the proposed structures and the associated site features and amenities. The following sections outline our evaluation of each foundation system, followed by detailed recommendations.

Shallow foundation systems (i.e., spread footings) are generally more cost effective than a deep foundation system provided that the subgrade at the proposed bottom of footing elevation consists of competent bearing material and the settlement from the building loads is not excessive. The natural residual soils, weathered rock, or competent rock materials are suitable materials for support of the structure on a shallow foundation system. Due to the presence and variability of the very loose to loose fill material in the upper soil profile, support of the proposed at-grade levels of the proposed structures on a shallow foundation system is not entirely feasible. In several locations, the fill material extends beyond the anticipated bearing elevation of the portions of the structure with a cellar level. Based on the borings performed on the site, we anticipate that significant amounts of fill or loose natural residual soils will likely be encountered at the bearing elevation for the ground level and in some cases, the cellar level as well. Additionally, it is our understanding that approximately 70% of the structure will be within the WMATA Zone of Influence which may or may not require the use of deep foundation elements to bypass the WMATA Zone of Influence. As such, we recommend that deep foundation elements be utilized for the support of the structure. A deep foundation system bearing in the competent weathered rock material would be suitable for support of the proposed structures while bypassing both the undocumented fill material and the WMATA Zone of Influence. Based on the results of our analysis and our experience on other projects in close proximity to the project site, augered cast-in-place (ACIP) piles have been determined to be the optimal foundation system from an economic and constructability perspective.

Foundation Recommendation: Auger Cast-In-Place Piles

The use of 150 ton piles will result in economical 3-PC pile caps given the typical column loads in the structure. The following table summarizes the recommended ACIP piles.

Table 1: ACIP Pile Parameters

PILE DIAMETER (INCHES)	REINFORCING STEEL(1)	AXIAL PILE CAPACITY		GROUT STRENGTH (PSI)	ESTIMATED TIP ELEVATION(2) (FEET)
		AXIAL COMPRESSION (TONS; FS=2)	AXIAL TENSION (TONS, FS=3)		
18	6 #5 bars (upper 25 ft.) with #3 ties at 12" on-center, 1 #10 bar full length	150	30	4,000	+230 to +215

Notes: (1) The reinforcing steel provided in Table 1 accounts for geotechnical considerations only. More steel may be required for structural reasons.
(2) The estimated tip elevation should be refined after the completion of the recommended test pile program.
(3) Geotechnical Static computations indicate that the ACIP piles must be embedded a minimum of 15 feet into the weathered rock material.

Properly installed ACIP piles bearing in the weathered rock are anticipated to settle less than 1 inch with differential settlement between columns of less than ½ inch. Table 1 above demonstrates a significant variance in the estimated pile tip elevations within the building footprint. This is a function of the varying elevation of the competent rock elevation and the anticipated elevation where the ACIP pile installation equipment may refuse. A goal of the suggested test pile program is to clarify the required minimum pile tip elevation throughout the site and refine the recommendations based on field observations. Once indicator piles and subsequent load testing is complete, the geotechnical engineer should render opinions on the required pile tip elevations or acceptance criteria for the site.

ACIP piles generate significant amounts of their resistance from side friction and therefore, the minimum tip elevation will be required in order to achieve the recommended design capacity. As such, the ACIP contractor selected for the installation of the piles should be aware of the geologic conditions in selecting his equipment. It is our understanding the penetrating the weathered rock stratum may require the use of a Bower rig with greater crowd pressure than is generated from conventional gravity rigs. If the minimum tip elevations are not achieved due to auger refusal, the piles may be downgraded in capacity.

ACIP Pile Installation QA / QC

ACIP piles are subject to pile necking during installation due to the presence of very soft / very loose fill soils. Pile necking would reduce the sectional area of the pile, reducing the structural capacity of the pile and potentially causing structural failure of the pile element. In an effort to control the potential necking of the piles, a strict QA/QC program is required to verify that the pile integrity is not compromised during installation. The following sections briefly outline our recommendations during pile installation.

Materials and Equipment

Grout

The grout used shall consist of a mixture of Portland Cement, fluidifier, retarder, fine aggregate and water so proportioned and mixed as to produce a grout mix capable of being pumped. The pile grout shall have a minimum 28-day compressive strength of 6,000 psi. Mixing time after adding the fluidifier at the site shall be no less than 3 minutes. The grout shall be mixed in accordance with the applicable requirements of ASTM C94.

The Contractor shall not use any grout older than the maximum time specified by the supplier. If the pre-approved maximum time limit is in excess of 120 minutes, the supplier shall provide adequate documentation that the grout does not become detrimentally affected beyond this general local industry accepted standard time limit. The Contractor shall coordinate his grout delivery to meet the above requirement and to assure continuity of the work.

The viscosity of the grout should be controlled with a grout cone. This will reduce the variability of the grout and result in a more uniform compressive strength. It is recommended that the flow cone requirement be specified as a range rather than as a single value.

The grout shall be sampled and tested by an independent Testing Laboratory retained by the Owner. During indicator and test pile installation, sampling and casting of a set of six 2-inch cubes shall be made from each truck of grout delivered to the site. During production pile installation, sampling and casting of a set of six 2-inch cubes shall be made for every 50 cubic yards of grout delivered to the site and no less than once per day. For test piles and production pile grout cube sets, one cube shall be tested at 7 days, one at 14 days, one at 21 days, one at 28 days, and one at 56 days. One cube shall be kept as a reserve in case of low grout strength results. If the 56-day cube breaks at strength greater than or equal to the required, then the last cube shall also be tested at 56 days. If not, the last cube shall be tested at 84 days. Grout cubes shall be made and tested in accordance with ASTM C31, C109 and C469. The test results shall be submitted to the Owner, the Structural Engineer, and the Geotechnical Engineer for review within 3 days of completion of the testing.

Reinforcing Steel

The augercast piles shall have reinforcing steel cages as shown on the Structural Plans. Additionally, the steel cages shall have #3 bars spacers, or pre-approved equal, so as to maintain the cages centered within the pile shaft. The spacers shall be located at the tip and the top of the cages, with additional spacers located not more than 15 ft on-center for the full embedded length of the pile. The spacers shall be attached so as to prevent bending prior to placement in the pile shaft, and shall be approved by the Geotechnical Engineer prior to use. The size of the spacers shall be such that a minimum 3-inch grout cover inside the pile shaft is maintained. We recommend full length cages to assist in verifying the integrity of the pile shaft after auger withdrawal. If no pile necking has occurred, the reinforcing cage should be able to be installed with relative ease.

Grout Pump

The grout pump should be a positive displacement piston pump capable of developing sufficient displacement pressures to assure the continuous and complete filling of the augered pile shafts. The Contractor shall field-calibrate the pump discharge capacity in strokes per cubic foot prior to the installation of piles so that grout take can be monitored by the Geotechnical Engineer.

ACIP Pile Rig

The pile rig shall be capable of advancing and withdrawing the auger in a slow and steady continuous motion, and shall have sufficient torque and weight to advance the auger to the required depths outlined in Table 1. The auger shall have continuous flights that are uniform 18-inch-diameter throughout its length with no reduction in section at any point along the length. The auger shall have a 3-inch minimum I.D. hollow stem to facilitate grout injection. The auger shall be capable of installing up to 60 ft. long piles.

Pile Installation

Piles should be installed at locations laid out by a surveyor and as shown on the Foundation Plans prepared by the Structural Engineer. Pile centers shall be within 3 inches of those shown on the Foundation Plans at the pile cut-off elevation. The piles shall be cut-off to the specified elevation with the specified reinforcement extended as required above the cut-off elevation. Vertical piles shall be installed with deviations of no more than 1-inch in 5 feet from a vertical line.

The piles shall be installed by the rotation of the continuous flight auger into the ground to the tip elevation as outlined in this report. Once the tip elevation has been attained, a slow positive rotation shall be maintained and the auger initially withdrawn 0.5 ft to 1 ft. Grout should then be pumped through the auger tip until a minimum grout head of 10 ft is achieved. This will be estimated based on the pump calibration performed prior to pile installation. The auger shall then be advanced back to the tip elevation and steadily withdrawn in a continuous operation while grout is being injected without interruption. The rate of auger withdrawal and that of grout injection shall be coordinated such that the amount of grout pumped per foot of pile during auger retrieval is at least 115% of the theoretical volume per foot of pile. A positive grout pressure head above the tip of the auger shall be maintained at all times as verified by the return of slurry/grout from around the auger flights. If the auger jumps during withdrawal, if the pump skips a stroke, or if there is a break in the slurry/grout return as observed from the top of the augered shaft, the auger shall be lowered a minimum of 5 feet below the depth of questionable area and regrouted. The rate of auger withdrawal shall not be increased once grout return is observed at the ground surface. If the auger is withdrawn too rapidly, suction within the pile shaft could occur, exacerbating the potential for pile necking. If the minimum 115% grout volume is not achieved, the pile shall be redrilled and regrouted at the affected depths.

The augered shaft shall be completely filled to the ground surface with grout. Grout shall not be removed from the augered shafts by dipping or other means prior to setting of the grout. Installed piles shall be periodically checked by the Contractor to determine if the grout in the piles has settled. If the grout level drops more than about 1 ft, the top of the pile shall be purged and fresh grout shall be added to the top of the pile prior to the grout reaching its initial set.

Immediately upon completion of the grouting operation of each pile, the specified reinforcement shall be installed. Care shall be taken not to contaminate the pile grout with soil or other foreign material during reinforcing steel cage installation. The steel cages shall be maintained at the center of the grout-filled augered pile shaft at all times. If difficulty is encountered during installation of the reinforcement, the pile shall be redrilled and regrouted. If problems are still encountered, then the shaft shall be filled with grout and abandoned, and alternate pile location(s) shall be determined by the structural engineer.

In case there is a loss of grout upon pile grouting or if there is no return of grout from the shaft during pumping, the shaft shall be temporarily abandoned and shall be redrilled and regrouted after approximately 1 hour. If problems are still encountered, then the shaft shall be redrilled and regrouted the following day. If problems are still encountered, then a replacement pile shall be installed at a location determined by the Structural Engineer.

A minimum grout set time of 12 hours shall be allowed before any adjacent piles are installed unless otherwise directed by the Geotechnical Engineer. No piles closer than 9 ft center to center shall be installed the same day. If grout loss is experienced in a completed pile while drilling an adjacent pile, the construction of the adjacent pile shall be ceased and the completed pile shall be redrilled and regrouted. The adjacent pile shall not be installed until the next day.

General

All pile locations should be staked from a batter board system or by surveying techniques. Installation of auger cast piles is messy and staked locations frequently become covered with mud or grout, or are destroyed by other means. Measuring from previously installed piles is not acceptable. Piles should be spaced at least 3 pile diameters center-to-center.

It will be necessary that the work be observed by a qualified geotechnical engineer or his authorized representative. The representative should observe the pressures used to pump the grout into the hole and also the withdrawal (withdrawal rate) of the auger to determine that the pile is being properly constructed. In addition, pile depths and any abnormalities encountered during drilling should be recorded. Properly installed auger cast-in-place piles should have total settlement less than 1 inch.

We recommend that the geotechnical engineer or record be present during installation of auger cast piles to perform the following functions:

1. Monitor and record the depths to which drilling is completed and the rate of auger withdrawal.
2. Monitor and record the amount of grout that goes into the pile and the rate at which the grout is pumped.
3. Check and calibrate the equipment for controlling and measuring the flow rate of grout into the pile.

Indicator / Test Pile Program

We recommend the installation of eight (8) pre-production indicator piles. Two (2) of the indicator piles will be statically load tested under the observation of the geotechnical engineer to determine adequate capacity. Four indicator piles should be installed within the limits of the cellar level and four indicator piles should be installed outside of the limits of the cellar level of the structure. One of the indicator piles within the each of the levels should be statically load tested. The indicator piles shall be installed prior to installation of the production piles at permanent pile locations. The purpose of the test pile program is to determine the production pile tip elevations (pile lengths), confirm our assumption of pile capacity (which is related to our design safety factor), to allow observation of the subsurface conditions encountered by the augers, and to provide the drilling contractor with an opportunity to determine the equipment required to achieve the design tip elevations. The standard load test procedures (not the quick test) should be utilized.

Four strain transducers shall be installed on each indicator / test pile rebar cage near the pile toe. The details of the strain transducer installation shall be submitted to the geotechnical engineer for approval prior to the start of the test pile program. Strain and subsequent stress data should be collected during the static load tests and transmitted along with the pile top movements required by ASTM D 1143.

Floor Slab Design

Based on the subsurface conditions observed during the subsurface exploration and the anticipated finished floor elevations, we recommend that the floor slabs be designed as a slab-on-grade bearing on competent soils or compacted engineered fill over competent soils.

The FFE for the ground and cellar levels are anticipated to be at approximately EL +261.5 feet and EL +247.5 feet, respectively. Based on the topographic survey drawing prepared by Delon Hampton and Associates, we anticipate that grading to achieve these proposed FFE for the ground level may require up to approximately 2 to 3 feet of cut/fill. The cellar level will require approximately 15 feet of cut to achieve the proposed FFE elevation. Based on these estimates and the results of our subsurface exploration, the soils at the bottom of the slab elevations are anticipated to consist of newly placed fills or Stratum I soils for the ground level portions and existing fills, Natural Residual Soils, or Weathered Rock for the cellar level portions.

The existing fill, residual soils, and weathered rock is likely suitable for support of the slab-on-grade. We recommend that the slab subgrade be heavily proofrolled with a 20 ton loaded dump truck. If any soft or yielding soils are observed during this proofroll, then 2 feet of existing soils should removed and replaced with compacted structural fill in accordance with the recommendations included in this report. The existing silty/granular soils removed during undercutting are anticipated to be suitable for reuse as engineered compacted fill, provided that they meet the requirements of this report. Prior to placing the engineered fill, the approved subgrade soil should be properly compacted, proofrolled, and free of standing water, mud, and frozen soil.

We recommend that the floor slab be isolated from the foundation pile caps so that differential settlement of the structure will not induce stresses on the floor slab. Also, in order to minimize the crack width of any shrinkage cracks that may develop near the surface of the slab, we recommend mesh reinforcement be included in the design of the floor slab. The mesh should be in the top half of the slab to be effective.

We also recommend the slabs-on-grade be underlain by a minimum of 6 inches of VDOT No. 57 stone. This granular layer will facilitate the fine grading of the subgrade and help prevent the rise of water through the floor slab. If available, a clean sand may be substituted for the gravel layer. When loads on the floor slab are in excess of 500 psf, we recommend additional reinforcing steel be placed in the floor slab.

Before the placement of concrete, a 6-mil vapor barrier should be placed on top of the granular material to provide additional moisture protection. However, special attention should be given to the surface curing of the slab in order to minimize uneven drying of the slab and associated cracking.

Below Grade Wall Design and Drainage

Below-grade walls should be designed to withstand lateral earth pressures and surcharge loads. We recommend that the laterally restrained walls (i.e. basement walls) be designed for a linearly increasing lateral fluid equivalent active earth pressure of 60 pcf which does not include hydrostatic water pressures. The wall design should also account for any surcharge loads (including adjacent structure foundations) within a 45 degree slope from the base of the wall.

This lateral earth pressure assumes that the below grade walls are fully drained (i.e., no hydrostatic pressures) and does not include any surcharge loads. Any surcharge loads imposed within a 45 degree slope of the base of the wall should be considered in the below grade wall design. The influence of these surcharge loads on the below grade walls should be based on an at-rest pressure coefficient, k_0, of 0.5.

To minimize excessive pressures against the retaining walls, and to reduce the settlement of the wall backfill, it is recommended that the wall backfill (if required) be compacted to 95% of the maximum dry density determined in accordance with ASTM Specification D-698, Standard Proctor Method. Where the fill will be supporting pavement or other structures, the fill should also be compacted to near 95% of this specification, except that the upper 1 foot should be compacted to 100% of the maximum dry density referenced above. Backfill materials which are placed behind below-grade walls should be free of organic materials and debris, free-draining (or with proper drainage provisions), non-frost susceptible, and should not include any highly plastic clays or silts (CH or MH). It is imperative that no CH or MH soils be used as backfill, due to the shrink-swell potential of these materials. The wall backfill should also have a Liquid Limit less than 40 and Plasticity Index of less than 15. The fill placed adjacent to the below grade walls should not be over compacted. Heavy earthwork equipment should maintain a minimum horizontal distance away from the below grade walls of 1 foot per foot of vertical wall height. Lighter compaction equipment should be used close to the below grade walls.

Suitable manmade drainage materials may be used in lieu of the free draining granular backfill, adjacent to the below grade walls. Examples of suitable materials include Enka-Mat, Mirafi, or J-Drain drainage composites. These materials should be covered with a filter fabric having an Apparent Opening Size (AOS) consistent with the size of the soils to be retained. The material should be placed in accordance with the manufacturer's recommendations and connected to either the perimeter drainage system or the underslab granular mat, which in turn should be properly drained. The ground surface adjacent to the below grade walls should be kept properly graded to prevent ponding of water adjacent to below grade walls.

Underslab Subdrainage

We recommend that the below grade areas for the structure be provided with a perimeter and underslab subdrainage system (i.e., a "drained" basement condition). A sketch titled "Below Grade Wall Waterproofing and Underslab Drainage Diagram" is included in the Appendix and is intended to graphically depict our recommendations for this report section. The system may consist of perforated, closed joint drain tiles located around the interior perimeter of the below grade areas, as close as feasible to the exterior wall, below the finished floor level. A network of interior pipes is also needed. Since an earth retention system will likely be required for construction, it is anticipated that "lot line" construction will be used. Weep holes (which convey drainage from behind the walls to the underslab subdrainage system) should be placed at a spacing of no greater than 8 feet on center, generally designed to align between the soldier piles of the earth retention system. The weep holes should be a minimum of four inches in diameter, and should freely drain from the exterior drainage medium to be collected by the interior perimeter drain line just inside the base of the wall. The drain lines should be surrounded by 6 inches of gravel or clean sand material having a gradation compatible with the size of the opening utilized in the drain lines and the surrounding soils to be retained.

We recommend that the perimeter and underslab drain system for the proposed structure be designed to flow to one permanent sump at a location to be determined by the design team. We recommend that the permanent sump be designed with a full duplex capability (i.e., two pumps per pit), with each individual pump rated at no less than 50 gallons per minute (gpm). With this configuration, under emergency conditions, the individual sump would have the capacity to pump 100 gpm. The contractor should monitor the pumping rate of the construction dewatering system in order to verify that the permanent sump pump has been adequately sized. Smaller or conversely larger pumps may ultimately be needed based on actual flow rates at the time of construction. Once the plans are further developed, please contact ECS so that we can refine our pumping estimates.

Lateral drain lines under the floor slab should be placed at no more than 30 feet on center. Underslab drain lines should have a minimum diameter of 4 inches, and they should be slotted or appropriately perforated. Clean out access should be installed at all sharp bends and at approximately every 100 feet for straight runs. A grit collection chamber should be installed upstream of the sump to reduce the amount of granular materials reaching the pumps. Per IBC, underslab drain lines should be sloped at a minimum of 0.5% and be underlain and covered by a minimum of 2 inches and 4 inches of gravel, respectively.

If requested, ECS can provide a proposal for the underslab drainage system design.

Adjacent Construction Monitoring and WMATA

Any buildings within a 3H:1V zone of influence from the edge of excavation and dewatering system should be monitored for settlement and lateral deflection during construction. The installation of a minimum number of three dimensional monitoring points on the existing adjacent structures located south and east of the project site should be considered. Typically, the monitoring points are created by taking ongoing survey shots, periodically during the construction dewatering, excavation and construction to grade to see if there are any building impacts.

While it is unlikely that significant settlement of adjacent structures and streets will occur if proper workmanship is employed during construction, it is prudent to perform such monitoring to defend against unfounded claims of structural damage by adjacent property owners. By having data available, such claims can be appropriately addressed.

Also of note is the presence of the METRO tunnel and station located near the project site, at the intersection of Old Georgetown Road and Wisconsin Avenue. It appears that the METRO is on the order of 120 feet from our site and may be out of the zone of influence of the proposed structure; however, once the plans are further along, WMATA will likely require a detailed adjacent construction impact analysis and monitoring plan during construction. ECS can provide a proposal for these services at your request.

Seismic Design Considerations

The subsurface exploration completed at this site included the drilling of a total of five (5) borings to depths of approximately 34 feet and 45 feet. The International Building Code (IBC) 2006 requires site classification for seismic design based on the upper 100 feet of a soil profile. Where site specific data are not available to a depth of 100 feet, appropriate soil and rock properties are permitted to be estimated by the registered design professional preparing the soils report based on known geologic conditions. The seismic site class definitions for the weighted average of shear wave velocity in the upper 100 feet of the soil and rock profile are presented in Table 1613.5.2 of the 2006 IBC Code and in the table below.

Site Class	Soil Profile Name	Shear Wave Velocity, Vs, (ft./s)
A	Hard Rock	Vs > 5,000 fps
B	Rock	2,500 < Vs ≤ 5,000 fps
C	Very dense soil and soft rock	1,200 < Vs ≤ 2,500 fps
D	Stiff Soil Profile	600 ≤ Vs ≤ 1,200 fps
E	Soft Soil Profile	Vs < 600 fps

Considering the soil and rock profile encountered at this site, we recommend a seismic site classification of Site Class D.

METRO Impact

The WMATA "Adjacent Construction Design Manual", Revision 4, requires any developer of a site adjacent to, or over, an existing WMATA facility to submit their design for the construction of these structures for their review. If the planned development is located within the WMATA zone of influence, WMATA will require plans and supporting calculations of the proposed construction for their review in order to determine what effect, it any, the adjacent development will impose on their structure.

In addition, METRO will impose specific design requirements for the earth retention system. These are outlined in great detail in the METRO "Adjacent Construction Design Manual" which can be obtained from the METRO official. The earth retention contractors in this area, which are predominately design build contractors, should be familiar with this manual, and must design it in strict compliance with the METRO guidelines.

METRO will not allow the imposition of additional loads on the METRO tunnel by the new building construction. Based on the recommendation that the structure is supported on ACIP piles installed to an estimated tip elevation below the WMATA zone of influence, it is our opinion that the METRO tunnel will not experience an increase in loading as a result of the proposed buildings and will most likely experience a decrease in effective stress. However, a detailed stress analysis submission will be required to be submitted to METRO for their review.

Once additional information has been provided to our office with regard to the exact location of the foundation elements as well as the foundation loads that will be imposed by the new building, we can perform a stress analysis, which considers the location of the METRO tunnel, the excavation depth, and the foundation loads. Included in this WMATA analysis will be a preliminary monitoring action plan submittal to WMATA which will detail our monitoring protocol during construction at this site, which will also require their approval.

PROJECT CONSTRUCTION

Subgrade Preparation and Earthwork Operations

Initial preparation of the site should consist of complete removal of existing pavements, pavers, sidewalks, and foliage that will not be part of the new construction. Further excavation to the design subgrade level should be limited to about 1 foot above the design subgrade. This will allow any equipment required to excavate footing foundations the ability to negotiate the site on material that will ultimately be removed. We recommend this plan in order to limit undercutting that may be necessary due to the surface disturbance caused by construction traffic and exposure to weather.

Upon removal of the protective layer and excavation to the floor subgrade, the building slab areas should be observed by the Geotechnical Engineer of Record or his authorized representative. Any soft or unsuitable materials encountered during the observation process should be removed and replaced with an approved backfill material in accordance with the earthwork specifications presented in this report.

All excavations should be adequately sloped or braced in order to protect construction personnel and equipment working at the site. OSHA safety regulations should be followed in all cases. If any problems are encountered during the earthwork operations, or if site conditions deviate from those encountered during our subsurface exploration, the Geotechnical Engineer should be notified immediately.

Fill Placement

All fills should consist of an approved material, free of organic matter and debris, cobbles greater than 4-inches and have a Liquid Limit and Plasticity Index less than 40 and 20, respectively. Unacceptable fill materials include topsoil and organic materials (OH, OL), and high plasticity silts and clays (CH, MH). Under no circumstances should high plasticity soils be used as fill material in proposed structural areas or close to site slopes. The on-site Stratum II - Natural Residual Soils appear to be suitable for reuse as backfill material. Wall backfill will require a maximum Liquid Limit and Plasticity Index of 40 and 15, respectively.

The on-site silty sand and silt soils may require moisture content adjustments, such as the application of discing or other drying techniques or spraying of water to the soils prior to their use as compacted fill (termed manipulation). The planning of earthwork operations should recognize and account for increased costs associated with manipulation of the on-site materials considered for reuse as compacted fill.

Fill materials should be placed in lifts not exceeding 8-inches in loose thickness and moisture conditioned to within ±2 percentage points of the optimum moisture content. Soil bridging lifts should not be used, since excessive settlement of overlying structures will likely occur. Controlled fill soils should be compacted to a minimum of 95% of the maximum dry density obtained in accordance with ASTM Specification D-698, Standard Proctor Method. However, the upper one foot of soil supporting slabs-on-grade, pavements, sidewalks, or gutters should be compacted to a minimum of 100% of the maximum dry density obtained in accordance with ASTM Specification D-698, Standard Proctor Method.

The expanded footprint of the proposed pavement and fill areas should be well defined, including the limits of the fill zones at the time of fill placement. Grade control should be maintained throughout the fill placement operations. All fill operations should be observed on a full-time basis by a qualified soil technician to determine that the specified compaction requirements are being met. A minimum of one compaction test per 2,500 square foot area should be tested in each lift placed. The elevation and location of the tests should be clearly identified at the time of fill placement.

Compaction equipment suitable to the soil type used as fill should be used to compact the fill material. Theoretically, any equipment type can be used as long as the required density if achieved. Ideally, a steel drum roller would be most efficient for compacting and sealing the surface soils. All areas receiving fill should be graded to facilitate positive drainage from building pad and pavement areas of any free water associated with precipitation and surface runoff.

It should be noted that prior to the commencement of fill operations and/or utilization of any off-site borrow materials, the Geotechnical Engineer of Record should be provided with representative samples to determine the material's suitability for use in a controlled compacted fill and to develop moisture-density relationships. In order to expedite the earthwork operations, if off-site borrow materials are required, it is recommended they be comprised of a select granular material which will provide suitable support and be easily compacted and well drained.

Fill materials should not be placed on frozen soils or frost-heaved soils and/or soils which have been recently subjected to precipitation. All frozen soils should be removed prior to continuation of fill operations. Borrow fill materials, if required, should not contain frozen materials at the time of placement. All frost-heaved soils should be removed prior to placement of controlled, compacted fill, granular subbase materials, foundation or slab concrete, and asphalt pavement materials.

Earth Retention System

A temporary excavation system will be required for construction of the cellar level of the proposed development. A free draining system consisting of soldier piles and wood lagging is recommended. The system should be braced externally using tiebacks, if possible. Spacing of the soldier piles and braces should be determined by a structural analysis. However, we recommend that the maximum center line to center line spacing of the soldier piles not exceed 8 feet. In addition, wooden lagging should have a minimum thickness of 3 inches. The final design of the system should be performed by a specialist in this area and is not part of the scope of this report. The earth retention system should be designed for both global stability as well as stability at the face of the excavation.

If tiebacks are used, we recommend a "performance test" be performed on 10% of randomly selected tiebacks (or a minimum of three tiebacks, whichever is greater). The performance test evaluates the tieback load carrying capacity, deflections during loading, and movements with respect to time. We recommend tiebacks be tested and accepted / rejected based on PTI standards.

In areas where tiebacks are not feasible, an internal bracing system of rakers or cross lot bracing would be required. Rakers should be braced against toe blocks or other reaction points that have been designed to carry the load.

Underpinning may be necessary in areas of the site where the proposed excavation will end below adjacent footings subgrade levels. Underpinning may consist of concrete piers bearing on suitable very dense materials below the proposed subgrade levels. As an alternative, it may also be feasible to provide underpinning using a system of bracketed soldier piles. As in the case for concrete pier underpinning, the soldier beam pile system should also bear below the proposed excavation subgrade.

The contractor should avoid stockpiling excavated materials immediately adjacent to the excavation walls. We recommend that stockpile materials be kept back from the excavation a minimum distance equal to one-half the excavation depth to avoid surcharging the excavation walls. If this is impractical due to space constraints, the excavation walls should be retained with bracing designed for the anticipated surcharge loading.

Earth Retention System/Support of Excavation (SOE) Performance Requirements

We recommend the following specification for use in the construction documents associated with the earth retention system.

Part 1 – General

1. Contractor/Designer shall design and construct a temporary Support of Excavation (SOE) system sufficient to support the project's below grade construction.

Part 2 – Submittals

1. SOE design plans sealed by a licensed Professional Engineer for the jurisdiction the work is performed in.
2. All supporting calculations for the SOE design, including global stability calculations.
3. Subsurface data utilized for the SOE design.
4. The braced excavation contractor shall submit the anticipated movement amounts (vertically and laterally) of each portion of the excavation support system to the owner's engineering consultant. These anticipated movements will also serve as the basis for evaluating the performance of the excavation support system. If creep movements are anticipated, the contractor shall state the total expected magnitude and rate during the time frame the SOE system is required to support the excavation. The contractor's estimated excavation support movements shall be subject to review and acceptance by the owner's engineering consultant before they are used as the performance standard.
5. Jack calibration data for any equipment utilized to tension tieback anchors. Calibration records must be current within a 12 month period of the time of anchor stressing.
6. Proposed Performance Test Locations and elevations (for tieback anchors).
8. If not stated on the plans, the method of soldier pile installation.

Part 3 – Performance Requirements

1. The performance of the braced excavation system will be monitored (measured) by the owner's engineering consultants. These measurements will serve as the basis for determining the performance and adequacy of the excavation support system. The initial baseline measurements and periodic movement data will be provided to all parties involved in construction. The initial baseline measurements shall be obtained before significant portions of the below grade excavation work occur, and preferably before any excavation work begins. The contractor may make his own independent measurements; however, the owner's engineering consultant's measurements will serve as the basis for performance evaluation.

2. If the movements of the excavation support system exceed the contractor's estimate, additional support for the excavation support system shall be provided by the contractor on an urgent basis, at no additional cost to the owner. If the excavation support system is creeping (inward or downward), and the owner's engineers projected estimate of total movement (within the performance time period of the excavation support system) exceeds the total movement estimates provided by the contractor, then additional support shall be added to the braced excavation system to halt the creeping, also on an urgent basis, at no additional cost to the owner.

Part 4 – Monitoring by Owner's Engineering Consultant

1. Prior to or very near the commencement of below grade excavation work, baseline data of the position of the SOE system will be obtained. Baseline measurements and subsequent movement evaluation will be performed with either total station, laser technology or optical surveying equipment. Total station technology is capable of making precise measurements of movement (±0.125 inches). Reflector "targets" will be attached to the SOE system by the Owner's Engineering Consultant, with the full cooperation and assistance of the SOE contractor. The Owner's Engineering Consultant, with the assistance of the SOE contractor, shall replace any previously established targets if they are damaged during construction.

2. Monitoring Frequency. The SOE monitoring frequency is recommended as follows:

   - Once to twice weekly during construction of all below-grade levels.

   - Monitoring frequency will remain at once to twice per week until the structural engineer (SE) indicates that all below-grade level walls and floors are constructed and capable of resisting the below-grade soil and water pressures.

   - Monitoring ceases after below grade construction ends and SE indicates that all below grade-level walls and floors are constructed and capable of resisting the below-grade soil and water pressures.

3. Reporting.

   - The results of the monitoring readings will be transmitted verbally to either the general contractor's representative or the SOE contractor's representative during the field work. Any significant movements since the prior readings will be identified.

- Written reports containing the monitoring data and corresponding graphical presentation of said data will be provided by the Engineer to all interested parties, electronically and in hardcopy form, on a weekly or twice monthly basis.

Construction Dewatering

While we do not expect a deep well system will be required during construction, we do recommend that positive site drainage be maintained during construction. Depending on the fluctuation of groundwater levels due to rainfall and other factors, it may be necessary to control groundwater through the use of sump pit and pumping systems. In addition, trenching and sump pumping may be required to dewater localized areas and remove groundwater from the site. This system may consist of multiple trenches and sumps including pumps placed in perforated 55-gallon barrels, installed during the excavation process.

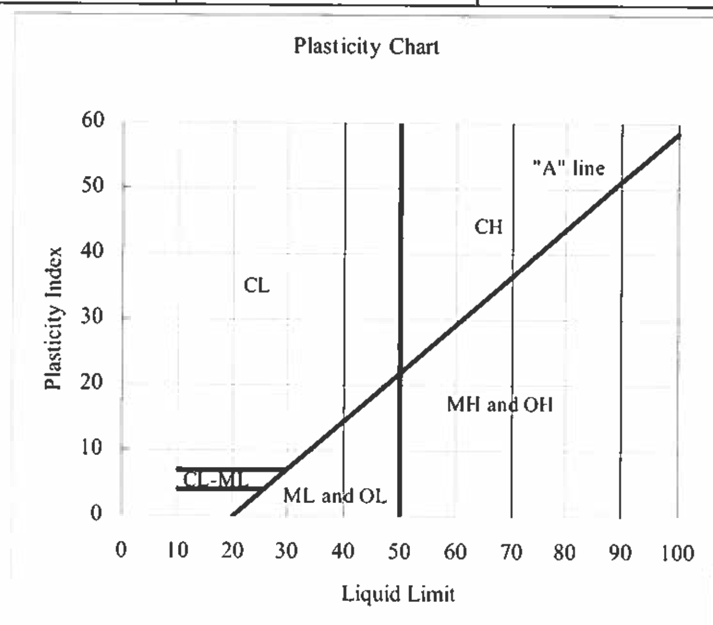
We anticipate that some localized areas within the excavations may not be completely dry and will require the use of small trenches, sump pits and pumps to facilitate the placement of the foundations. Installation and operation of the dewatering system should occur before the initiation of excavation operations at the site. A totally dry subgrade should not be anticipated; however, the surface of the subgrade should be sufficiently dewatered to provide an adequate surface on which to construct the foundations and floor slabs.

Closing

We recommend that the construction activities be monitored by a qualified geotechnical engineering firm to provide the necessary overview and to check the suitability of the subgrade soils for supporting the footings. We would be pleased to provide these services. If you have any questions with regard to this information or need any further assistance during the design and construction of the project please feel free to contact us.

APPENDIX

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D 2487)

Major Divisions			Group Symbols		Typical Names	Laboratory Classification Criteria		
Coarse-grained soils (More than half of material is larger than No. 200 Sieve size)	Gravels (More than half of coarse fraction is larger than No. 4 sieve size)	Clean gravels (Little or no fines)	GW		Well-graded gravels, gravel-sand mixtures, little or no fines	Determine percentages of sand and gravel from grain-size curve. Depending on percentage of fines (fraction smaller than No. 200 sieve size), coarse-grained soils are classified as follows: Less than 5 percent GW, GP, SW, SP; More than 12 percent GM, GC, SM, SC; 5 to 12 percent Borderline cases requiring dual symbols [b]	$C_u = D_{60}/D_{10}$ greater than 4 $C_c = (D_{30})^2/(D_{10} x D_{60})$ between 1 and 3	
			GP		Poorly graded gravels, gravel-sand mixtures, little or no fines		Not meeting all gradation requirements for GW	
		Gravels with fines (Appreciable amount of fines)	GM[a]	d	Silty gravels, gravel-sand mixtures		Atterberg limits below "A" line or P.I. less than 4	Above "A" line with P.I. between 4 and 7 are borderline cases requiring use of dual symbols
				u				
			GC		Clayey gravels, gravel-sand-clay mixtures		Atterberg limits below "A" line or P.I. less than 7	
	Sands (More than half of coarse fraction is smaller than No. 4 sieve size)	Clean sands (Little or no fines)	SW		Well-graded sands, gravelly sands, little or no fines		$C_u = D_{60}/D_{10}$ greater than 6 $C_c = (D_{30})^2/(D_{10} x D_{60})$ between 1 and 3	
			SP		Poorly graded sands, gravelly sands, little or no fines		Not meeting all gradation requirements for SW	
		Sands with fines (Appreciable amount of fines)	SM[a]	d	Silty sands, sand-silt mixtures		Atterberg limits above "A" line or P.I. less than 4	Limits plotting in CL-ML zone with P.I. between 4 and 7 are borderline cases requiring use of dual symbols
				u				
			SC		Clayey sands, sand-clay mixtures		Atterberg limits above "A" line with P.I. greater than 7	
Fine-grained soils (More than half material is smaller than No. 200 Sieve)	Silts and clays (Liquid limit less than 50)		ML		Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity			
			CL		Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays			
			OL		Organic silts and organic silty clays of low plasticity			
	Silts and clays (Liquid limit greater than 50)		MH		Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts			
			CH		Inorganic clays of high plasticity, fat clays			
			OH		Organic clays of medium to high plasticity, organic silts			
	Highly Organic soils		Pt		Peat and other highly organic soils			

[a] Division of GM and SM groups into subdivisions of d and u are for roads and airfields only. Subdivision is based on Atterberg limits; suffix d used when L.L. is 28 or less and the P.I. is 6 or less; the suffix u used when L.L. is greater than 28.

[b] Borderline classifications, used for soils possessing characteristics of two groups, are designated by combinations of group symbols. For example GW-GC, well-graded gravel-sand mixture with clay binder. (From Table 2.16 - Winterkorn and Fang, 1975)

REFERENCE NOTES FOR BORING LOGS

I. Drilling Sampling Symbols

SS	Split Spoon Sampler	ST	Shelby Tube Sampler
RC	Rock Core, NX, BX, AX	PM	Pressuremeter
DC	Dutch Cone Penetrometer	RD	Rock Bit Drilling
BS	Bulk Sample of Cuttings	PA	Power Auger (no sample)
HSA	Hollow Stem Auger	WS	Wash sample
REC	Rock Sample Recovery %	RQD	Rock Quality Designation %

II. Correlation of Penetration Resistances to Soil Properties

Standard Penetration (blows/ft) refers to the blows per foot of a 140 lb. hammer falling 30 inches on a 2-inch OD split-spoon sampler, as specified in ASTM D 1586. The blow count is commonly referred to as the N-value.

A. Non-Cohesive Soils (Silt, Sand, Gravel and Combinations)

Density		*Relative Properties*	
Under 4 blows/ft	Very Loose	Adjective Form	12% to 49%
5 to 10 blows/ft	Loose	With	5% to 12%
11 to 30 blows/ft	Medium Dense		
31 to 50 blows/ft	Dense		
Over 51 blows/ft	Very Dense		

Particle Size Identification		
Boulders		8 inches or larger
Cobbles		3 to 8 inches
Gravel	Coarse	1 to 3 inches
	Medium	½ to 1 inch
	Fine	¼ to ½ inch
Sand	Coarse	2.00 mm to ¼ inch (dia. of lead pencil)
	Medium	0.42 to 2.00 mm (dia. of broom straw)
	Fine	0.074 to 0.42 mm (dia. of human hair)
Silt and Clay		0.0 to 0.074 mm (particles cannot be seen)

B. Cohesive Soils (Clay, Silt, and Combinations)

Blows/ft	*Consistency*	*Unconfined Comp. Strength Q_p (tsf)*	*Degree of Plasticity*	*Plasticity Index*
Under 2	Very Soft	Under 0.25	None to slight	0 – 4
3 to 4	Soft	0.25-0.49	Slight	5 – 7
5 to 8	Medium Stiff	0.50-0.99	Medium	8 – 22
9 to 15	Stiff	1.00-1.99	High to Very High	Over 22
16 to 30	Very Stiff	2.00-3.00		
31 to 50	Hard	4.00–8.00		
Over 51	Very Hard	Over 8.00		

III. Water Level Measurement Symbols

WL	Water Level	BCR	Before Casing Removal	DCI	Dry Cave-In
WS	While Sampling	ACR	After Casing Removal	WCI	Wet Cave-In
WD	While Drilling	▽	Est. Groundwater Level	▼	Est. Seasonal High GWT

The water levels are those levels actually measured in the borehole at the times indicated by the symbol. The measurements are relatively reliable when augering, without adding fluids, in a granular soil. In clay and plastic silts, the accurate determination of water levels may require several days for the water level to stabilize. In such cases, additional methods of measurement are generally applied.

CLIENT	JOB #	BORING #	SHEET
Cannon Design	17393	B-1	1 OF 2

PROJECT NAME: University of DC Student Union Building

ARCHITECT-ENGINEER:

SITE LOCATION: Connecticut Avenue NW and Van Ness Stree, Washington, DC 20008

CALIBRATED PENETROMETER TONS/FT.^{2}: 1 2 3 4 5+

PLASTIC LIMIT % X — WATER CONTENT % ● — LIQUID LIMIT % △

ROCK QUALITY DESIGNATION & RECOVERY: RQD% — — — REC.% ——— 20% 40% 60% 80% 100%

⊗ STANDARD PENETRATION BLOWS/FT.: 10 20 30 40 50+

DESCRIPTION OF MATERIAL — ENGLISH UNITS

BOTTOM OF CASING — LOSS OF CIRCULATION 100%

SURFACE ELEVATION: 268

DEPTH (FT)	SAMPLE NO.	SAMPLE TYPE	SAMPLE DIST. (IN)	RECOVERY (IN)	DESCRIPTION OF MATERIAL	ELEVATION (FT)	STANDARD PENETRATION BLOWS/FT.
0	1	SS	18	10	Topsoil Depth 3"	268	6 (2-3-3)
					Silty Fine to Medium SAND, Trace Gravel and Brick Fragments, Brown, Moist, Medium Dense (SM-FILL)		
	2	SS	18	14		265	8 (2-3-5)
5	3	SS	18	14	Silty Fine to Medium Micaceous SAND, Light Tan and Brown, Moist, Loose (SM)		18 (3-5-13)
	4	SS	12	12	Weathered ROCK, Sampled as Silty Fine to Coarse Micaceous SAND, With Gravel, Light Tan and Gray, Moist, Very Dense	260	50/6 (23-50/6)
10						255	
	5	SS	3	3			50/3 (50/3)
15						250	
	6	SS	1	1			50/1 (50/1)
20	7	RC	60	29	Schist, Gray and Brown, Moderately to Highly Weathered, Soft, Highly Fractured [REC=48% RQD=8%]	245	
25	8	RC	60	24	Schist, Gray and Brown, Moderately to Highly Weathered, Soft, Highly Fractured [REC=40% RQD=8%]	240	
30	9	RC	60	18	Schist, Gray and Brown, Moderately to Highly Weathered, Soft, Highly Fractured [REC=30% RQD=18%]		

CONTINUED ON NEXT PAGE.

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES IN-SITU THE TRANSITION MAY BE GRADUAL

WL DRY — WS OR WD	BORING STARTED 03/18/11	DRILLER: Connelly
WL(BCR) DRY — WL(ACR) DRY	BORING COMPLETED 03/18/11	CAVE IN DEPTH ● 19.0'
WL	RIG CME 75 FOREMAN S. Effland	DRILLING METHOD 2.25" Hollow Stem Auger

CLIENT	JOB #	BORING #	SHEET
Cannon Design	17393	B-1	2 OF 2

ECS

PROJECT NAME: University of DC Student Union Building

ARCHITECT-ENGINEER:

SITE LOCATION: Connecticut Avenue NW and Van Ness Stree, Washington, DC 20008

-O- CALIBRATED PENETROMETER TONS/FT.² 1 2 3 4 5+

PLASTIC LIMIT % X — WATER CONTENT % ● — LIQUID LIMIT % △

ROCK QUALITY DESIGNATION & RECOVERY: RQD% — — — REC.% —— 20% 40% 60% 80% 100%

⊗ STANDARD PENETRATION BLOWS/FT. 10 20 30 40 50+

DESCRIPTION OF MATERIAL — ENGLISH UNITS

BOTTOM OF CASING — LOSS OF CIRCULATION 100%

SURFACE ELEVATION 268

DEPTH (FT)	SAMPLE NO.	SAMPLE TYPE	SAMPLE DIST. (IN)	RECOVERY (IN)	DESCRIPTION OF MATERIAL	ELEVATION (FT)
30	9	RC	60	18	Schist, Gray and Brown, Moderately to Highly Weathered, Soft, Highly Fractured [REC=30% RQD=18%]	235
35	10	RC	60	34	Schist, Gray and Brown, Moderately to Highly Weathered, Soft, Closely Jointed to Highly Fractured [REC=56% RQD=36%]	230
40	11	RC	60	34	Schist, Gray and Brown, Moderately to Highly Weathered, Soft, Closely Jointed to Highly Fractured [REC=56% RQD=30%]	225
45					END OF BORING @ 44.5'	220
50						215
55						210
60						

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES IN-SITU THE TRANSITION MAY BE GRADUAL

WL DRY — WS OR WD	BORING STARTED 03/18/11	DRILLER: Connelly
WL(BCR) DRY — WL(ACR) DRY	BORING COMPLETED 03/18/11	CAVE IN DEPTH @ 19.0'
WL	RIG CME 75 FOREMAN S. Effland	DRILLING METHOD 2.25" Hollow Stem Auger

CLIENT	JOB #	BORING #	SHEET
Cannon Design	17393	B-2	1 OF 2

PROJECT NAME: University of DC Student Union Building

ARCHITECT-ENGINEER:

SITE LOCATION: Connecticut Avenue NW and Van Ness Stree, Washington, DC 20008

ECS

-○- CALIBRATED PENETROMETER TONS/FT.² 1 2 3 4 5+

PLASTIC LIMIT % X — WATER CONTENT % ● — LIQUID LIMIT % △

ROCK QUALITY DESIGNATION & RECOVERY RQD% — — — REC.% —— 20% 40% 60% 80% 100%

⊗ STANDARD PENETRATION BLOWS/FT. 10 20 30 40 50+

DESCRIPTION OF MATERIAL — ENGLISH UNITS

BOTTOM OF CASING — LOSS OF CIRCULATION 100%

SURFACE ELEVATION 260

DEPTH (FT)	SAMPLE NO.	SAMPLE TYPE	SAMPLE DIST. (IN)	RECOVERY (IN)	DESCRIPTION OF MATERIAL	STANDARD PENETRATION BLOWS/FT.
0	1	SS	18	3	Brick Depth 2"	14 (13-6-8)
					Gravelly Fine to Medium SAND, With Silt, Trace Asphalt, Brown, Moist, Medium Dense, (SW-FILL)	
	2	SS	18	12		22 (5-6-16)
5					Silty Fine SAND, Trace Gravel and Mica, Gray , Moist, Medium Dense, (SM-FILL)	
	3	SS	18	12	Fine to Medium SAND, With Brick Fragments, Trace Silt, Red, Moist, Medium Dense, (SP-FILL)	17 (10-7-10)
	4	SS	18	12	Clayey SILT, With Sand, Brown, Moist, Loose, (ML-FILL)	5 (2-2-3)
10						
	5	SS	18	10		4 (2-2-2)
15						
					Silty Fine to Medium Micaceous SAND, With Gravel, Brown, Moist to Wet, Loose to Dense, (SM)	
	6	SS	18	8		6 (5-3-3)
20						
	7	SS	18	12		15 (7-7-8)
25						
	8	SS	18	14		38 (14-18-20)
30						

ELEVATION (FT): 255, 250, 245, 240, 235

CONTINUED ON NEXT PAGE.

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES IN-SITU THE TRANSITION MAY BE GRADUAL

▽WL 18.00 — WS OR WD	BORING STARTED 03/16/11	DRILLER: Connelly
▼WL(BCR) 24.00 ▼WL(ACR) 24.00	BORING COMPLETED 03/16/11	CAVE IN DEPTH ● 25.0'
▽WL	RIG CME 75 FOREMAN S. Effland	DRILLING METHOD 2.25" Hollow Stem Auger

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(03/23/2011) Skdemchak

CLIENT	JOB #	BORING #	SHEET
Cannon Design	17393	B-2	2 OF 2

ECS

PROJECT NAME: University of DC Student Union Building

ARCHITECT-ENGINEER:

SITE LOCATION: Connecticut Avenue NW and Van Ness Stree, Washington, DC 20008

—O— CALIBRATED PENETROMETER TONS/FT.² 1 2 3 4 5+

PLASTIC LIMIT % X — WATER CONTENT % ● — LIQUID LIMIT % △

ROCK QUALITY DESIGNATION & RECOVERY: RQD% — — — REC.% —— 20% 40% 60% 80% 100%

⊗ STANDARD PENETRATION BLOWS/FT. 10 20 30 40 50+

DESCRIPTION OF MATERIAL — ENGLISH UNITS

BOTTOM OF CASING — LOSS OF CIRCULATION 100%

SURFACE ELEVATION 260

DEPTH (FT)	SAMPLE NO.	SAMPLE TYPE	SAMPLE DIST. (IN)	RECOVERY (IN)	DESCRIPTION OF MATERIAL	ELEVATION (FT)	STANDARD PENETRATION
30					Silty Fine to Medium Micaceous SAND, With Gravel, Brown, Moist to Wet, Loose to Dense, (SM)		
					Weathered ROCK, Sampled as Silty Fine to Coarse Micaceous SAND, With Gravel, Tan to Brown, Moist, Very Dense		
35	9	SS	18	16		225	(23-48-50) 98
40	10	SS	10	10		220	(49-50/4) 50/4
	11	SS	0	0	AUGER REFUSAL @ 42.0'		(50/0) 50/0
45						215	
50						210	
55						205	
60							

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES IN-SITU THE TRANSITION MAY BE GRADUAL

WL 18.00 — WS OR WD	BORING STARTED 03/16/11	DRILLER: Connelly
WL(BCR) 24.00 WL(ACR) 24.00	BORING COMPLETED 03/16/11	CAVE IN DEPTH ● 25.0'
WL	RIG CME 75 FOREMAN S. Effland	DRILLING METHOD 2.25" Hollow Stem Auger

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(03/23/2011) Sdomchak

CLIENT	JOB #	BORING #	SHEET
Cannon Design	17393	B-3	1 OF 2

PROJECT NAME: University of DC Student Union Building

ARCHITECT-ENGINEER:

SITE LOCATION: Connecticut Avenue NW and Van Ness Stree, Washington, DC 20008

-○- CALIBRATED PENETROMETER TONS/FT.² 1 2 3 4 5+

PLASTIC LIMIT % X — WATER CONTENT % ● — LIQUID LIMIT % △

ROCK QUALITY DESIGNATION & RECOVERY: RQD% — — — REC.% —— 20% 40% 60% 80% 100%

⊗ STANDARD PENETRATION BLOWS/FT. 10 20 30 40 50+

DESCRIPTION OF MATERIAL — ENGLISH UNITS

BOTTOM OF CASING — LOSS OF CIRCULATION 100%

SURFACE ELEVATION 263

DEPTH (FT)	SAMPLE NO.	SAMPLE TYPE	SAMPLE DIST. (IN)	RECOVERY (IN)	DESCRIPTION OF MATERIAL	ELEVATION (FT)	STANDARD PENETRATION BLOWS/FT.
0	1	SS	18	12	Brick Depth 2"		8 (12-4-4)
					Silty Fine to Medium SAND, Trace Gravel, Brown, Moist, Very Loose to Loose, (SM-FILL)	260	
	2	SS	18	14			4 (1-1-3)
5	3	SS	18	12			3 (1-1-2)
					Sandy SILT, With Clay, Trace Asphalt and Brick Fragments, Brown, Moist, Loose (ML-FILL)	255	
10	4	SS	18	5			5 (3-2-3)
					Silty Fine Micaceous SAND, Reddish Brown, Gray at 23.5 ft, Moist, Very Loose to Loose, (SM)	250	
15	5	SS	18	14			6 (3-3-3)
						245	
20	6	SS	18	3			4 (3-2-2)
						240	
25	7	SS	18	6			9 (7-6-3)
					Gravelly Fine to Medium SAND, Reddish Brown, Moist, Medium Dense (SW)	235	
30	8	SS	18	12			12 (8-6-6)

CONTINUED ON NEXT PAGE.

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES IN-SITU THE TRANSITION MAY BE GRADUAL

WL 33.00	WS OR WD	BORING STARTED 03/21/11	DRILLER: Connelly
WL(BCR) 28.00 WL(ACR) 24.00		BORING COMPLETED 03/21/11	CAVE IN DEPTH ● 29.0'
WL		RIG CME 75 FOREMAN S. Effland	DRILLING METHOD 2.25" Hollow Stem Auger

(03/23/2011)

Skdandrak

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CLIENT	JOB #	BORING #	SHEET
Cannon Design	17393	B-3	2 OF 2
PROJECT NAME	ARCHITECT-ENGINEER		
University of DC Student Union Building			

SITE LOCATION
Connecticut Avenue NW and Van Ness Stree, Washington, DC 20008

CALIBRATED PENETROMETER TONS/FT.² — 1, 2, 3, 4, 5+

PLASTIC LIMIT % X — WATER CONTENT % ● — LIQUID LIMIT % △

ROCK QUALITY DESIGNATION & RECOVERY — RQD% — REC.% — 20% 40% 60% 80% 100%

⊗ STANDARD PENETRATION BLOWS/FT. — 10, 20, 30, 40, 50+

DESCRIPTION OF MATERIAL — ENGLISH UNITS

BOTTOM OF CASING — LOSS OF CIRCULATION 100%

SURFACE ELEVATION 263

DEPTH (FT)	SAMPLE NO.	SAMPLE TYPE	SAMPLE DIST. (IN)	RECOVERY (IN)	DESCRIPTION OF MATERIAL	WATER LEVELS	ELEVATION (FT)	STANDARD PENETRATION BLOWS/FT.
30–32					Gravelly Fine to Medium SAND, Reddish Brown, Moist, Medium Dense (SW)			
32–45					Weathered ROCK, Sampled as Silty Fine Sand, With Gravel, Gray and Brown, Moist, Very Dense,	▽ (33.0)	230	
33.5	9	SS	5	3				(50/5) 50/5
38.5	10	SS	6	6			225	(50/6) 50/6
43.5	11	SS	3	3			220	(50/3) 50/3
45.0					AUGER REFUSAL @ 45.0'			

Depth scale: 30, 35, 40, 45, 50, 55, 60. Elevation scale: 230, 225, 220, 215, 210, 205.

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES IN-SITU THE TRANSITION MAY BE GRADUAL

▽WL 33.00 — WS OR WD	BORING STARTED 03/21/11	DRILLER: Connelly
▼WL(BCR) 28.00 ▼WL(ACR) 24.00	BORING COMPLETED 03/21/11	CAVE IN DEPTH ● 29.0'
▽WL	RIG CME 75 FOREMAN S. Effland	DRILLING METHOD 2.25" Hollow Stem Auger

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(03/23/2011)

SAdamchak

CLIENT	JOB #	BORING #	SHEET
Cannon Design	17393	B-4	1 OF 2

PROJECT NAME: University of DC Student Union Building

ARCHITECT-ENGINEER:

SITE LOCATION: Connecticut Avenue NW and Van Ness Stree, Washington, DC 20008

CALIBRATED PENETROMETER TONS/FT.² 1 2 3 4 5+

PLASTIC LIMIT % — WATER CONTENT % — LIQUID LIMIT %

ROCK QUALITY DESIGNATION & RECOVERY: RQD% REC.% 20% 40% 60% 80% 100%

STANDARD PENETRATION BLOWS/FT. 10 20 30 40 50+

DESCRIPTION OF MATERIAL — ENGLISH UNITS

BOTTOM OF CASING — LOSS OF CIRCULATION 100%

SURFACE ELEVATION 263

DEPTH (FT)	SAMPLE NO.	SAMPLE TYPE	SAMPLE DIST. (IN)	RECOVERY (IN)	DESCRIPTION OF MATERIAL	ELEVATION (FT)	STANDARD PENETRATION BLOWS/FT.
0	1	SS	18	12	Brick Depth 2"		12 (22-6-6)
					Fine to Medium SAND, With Gravel and Silt, Brown, Moist, Medium Dense, (SW-FILL)	260	
	2	SS	18	10			13 (8-6-7)
5	3	SS	18	10	Silty Fine to Medium Micaceous SAND, Trace Gravel, Brown, Moist, Loose to Medium Dense, (SM)		7 (2-3-4)
	4	SS	18	12		255	7 (3-3-4)
10							
	5	SS	18	12		250	22 (6-11-11)
15							
	6	SS	18	16		245	22 (8-11-11)
20							
	7	SS	11	11	Weathered ROCK, Sampled as Silty Fine Sand, With Gravel, Gray and Brown, Moist, Very Dense	240	(21-50/5) 50/5
25							
	8	SS	11	11		235	(40-50/5) 50/5
30							

CONTINUED ON NEXT PAGE.

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES IN-SITU THE TRANSITION MAY BE GRADUAL

WL 30.00 — WS OR WD	BORING STARTED 03/18/11	DRILLER: Connelly
WL(BCR) 33.00 WL(ACR) 20.00	BORING COMPLETED 03/18/11	CAVE IN DEPTH ● 34.0'
WL	RIG CME 75 FOREMAN S. Effland	DRILLING METHOD 2.25" Hollow Stem Auger

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(03/21/2011) SAdamchak

CLIENT	JOB #	BORING #	SHEET
Cannon Design	17393	B-4	2 OF 2

PROJECT NAME: University of DC Student Union Building

ARCHITECT-ENGINEER:

SITE LOCATION: Connecticut Avenue NW and Van Ness Stree, Washington, DC 20008

CALIBRATED PENETROMETER TONS/FT.² — 1, 2, 3, 4, 5+

PLASTIC LIMIT % X — WATER CONTENT % ● — LIQUID LIMIT % △

ROCK QUALITY DESIGNATION & RECOVERY — RQD% — REC.% — 20% 40% 60% 80% 100%

⊗ STANDARD PENETRATION BLOWS/FT. — 10, 20, 30, 40, 50+

DESCRIPTION OF MATERIAL — ENGLISH UNITS

BOTTOM OF CASING — LOSS OF CIRCULATION 100%

SURFACE ELEVATION 263

DEPTH (FT)	SAMPLE NO.	SAMPLE TYPE	SAMPLE DIST. (IN)	RECOVERY (IN)	DESCRIPTION OF MATERIAL	ELEVATION (FT)	STANDARD PENETRATION BLOWS/FT.
30					Weathered ROCK, Sampled as Silty Fine Sand, With Gravel, Gray and Brown, Moist, Very Dense	230 (water level)	
	9	SS	11	11			(48-50/5) 50/5
35						225	
	10	SS	5	5			(50/5) 50/5
40						220	
	11	SS	3	3			(50/3) 50/3
45	12	SS	0	0			(50/0) 50/0
					AUGER REFUSAL @ 45.0'	215	
50						210	
55						205	
60							

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES IN-SITU THE TRANSITION MAY BE GRADUAL

WL 30.00 — WS OR WD	BORING STARTED 03/18/11	DRILLER: Connelly
WL(BCR) 33.00 WL(ACR) 20.00	BORING COMPLETED 03/18/11	CAVE IN DEPTH ● 34.0'
WL	RIG CME 75 FOREMAN S. Effland	DRILLING METHOD 2.25" Hollow Stem Auger

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(03/21/2011) Skidmochak

CLIENT	JOB #	BORING #	SHEET
Cannon Design	17393	B-5	1 OF 2

PROJECT NAME: University of DC Student Union Building

ARCHITECT-ENGINEER:

SITE LOCATION: Connecticut Avenue NW and Van Ness Stree, Washington, DC 20008

CALIBRATED PENETROMETER TONS/FT.² 1 2 3 4 5+

PLASTIC LIMIT % — WATER CONTENT % — LIQUID LIMIT %

ROCK QUALITY DESIGNATION & RECOVERY: RQD% — REC.% — 20% 40% 60% 80% 100%

STANDARD PENETRATION BLOWS/FT. 10 20 30 40 50+

DESCRIPTION OF MATERIAL — ENGLISH UNITS

BOTTOM OF CASING — LOSS OF CIRCULATION 100%

SURFACE ELEVATION 260

DEPTH (FT)	SAMPLE NO.	SAMPLE TYPE	SAMPLE DIST. (IN)	RECOVERY (IN)	DESCRIPTION OF MATERIAL	ELEVATION (FT)	STANDARD PENETRATION BLOWS/FT.
0	1	SS	18	16	Brick Depth 2"	260	14 (10-8-6)
					Clayey Fine to Medium SAND, With Gravel, Brown, Moist, Medium Dense, (SC-FILL)		
	2	SS	18	18	Silty Fine to Medium SAND, Trace Gravel, Moist, Brown, Loose to Very Dense, (SM)		11 (6-5-6)
5	3	SS	18	18		255	5 (2-3-2)
	4	SS	18	18			6 (2-3-3)
10						250	
	5	SS	18	18			18 (6-7-11)
15						245	
	6	SS	18	18			(27-39-54) 93
20						240	
	7	SS	18	18			12 (6-5-7)
25						235	
					Weathered ROCK,		
	8	SS	6	6			(50/6) 50/6
30							

CONTINUED ON NEXT PAGE.

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES IN-SITU THE TRANSITION MAY BE GRADUAL

WL 14.50 — WS OR WD	BORING STARTED 03/16/11		DRILLER: Connelly
WL(BCR) 22.00 WL(ACR) 15.00	BORING COMPLETED 03/16/11		CAVE IN DEPTH ● 25.0'
WL	RIG CME 75	FOREMAN S. Effland	DRILLING METHOD 2.25 Hollow Stem Auger

(03/23/2011)

SAdamchak

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CLIENT	JOB #	BORING #	SHEET
Cannon Design	17393	B-5	2 OF 2

PROJECT NAME: University of DC Student Union Building

ARCHITECT-ENGINEER:

SITE LOCATION: Connecticut Avenue NW and Van Ness Stree, Washington, DC 20008

CALIBRATED PENETROMETER TONS/FT.^{2}: 1 2 3 4 5+

PLASTIC LIMIT % X — WATER CONTENT % ● — LIQUID LIMIT % △

ROCK QUALITY DESIGNATION & RECOVERY: RQD% — — — REC.% —— 20% 40% 60% 80% 100%

STANDARD PENETRATION BLOWS/FT.: 10 20 30 40 50+

DESCRIPTION OF MATERIAL — ENGLISH UNITS

BOTTOM OF CASING — LOSS OF CIRCULATION 100%

SURFACE ELEVATION: 260

DEPTH (FT)	SAMPLE NO.	SAMPLE TYPE	SAMPLE DIST. (IN)	RECOVERY (IN)	DESCRIPTION OF MATERIAL	WATER LEVELS	ELEVATION (FT)	STANDARD PENETRATION
30					Weathered ROCK,			
	9	SS	1	1				(50/1) 50/1
35					AUGER REFUSAL @ 34.0'		225	
40							220	
45							215	
50							210	
55							205	
60								

THE STRATIFICATION LINES REPRESENT THE APPROXIMATE BOUNDARY LINES BETWEEN SOIL TYPES IN-SITU THE TRANSITION MAY BE GRADUAL

WL 14.50 WS OR WD	BORING STARTED 03/16/11	DRILLER: Connelly
WL(BCR) 22.00 WL(ACR) 15.00	BORING COMPLETED 03/16/11	CAVE IN DEPTH ● 25.0'
WL	RIG CME 75 FOREMAN S. Effland	DRILLING METHOD 2.25 Hollow Stem Auger

(03/23/2011)

SAdamchek

ECS Mid-Atlantic, LLC
Chantilly, VA
Laboratory Testing Summary

Printed on (date): April 25, 2011

Project No. 17393 **Project Name.** University of DC Student Union Building

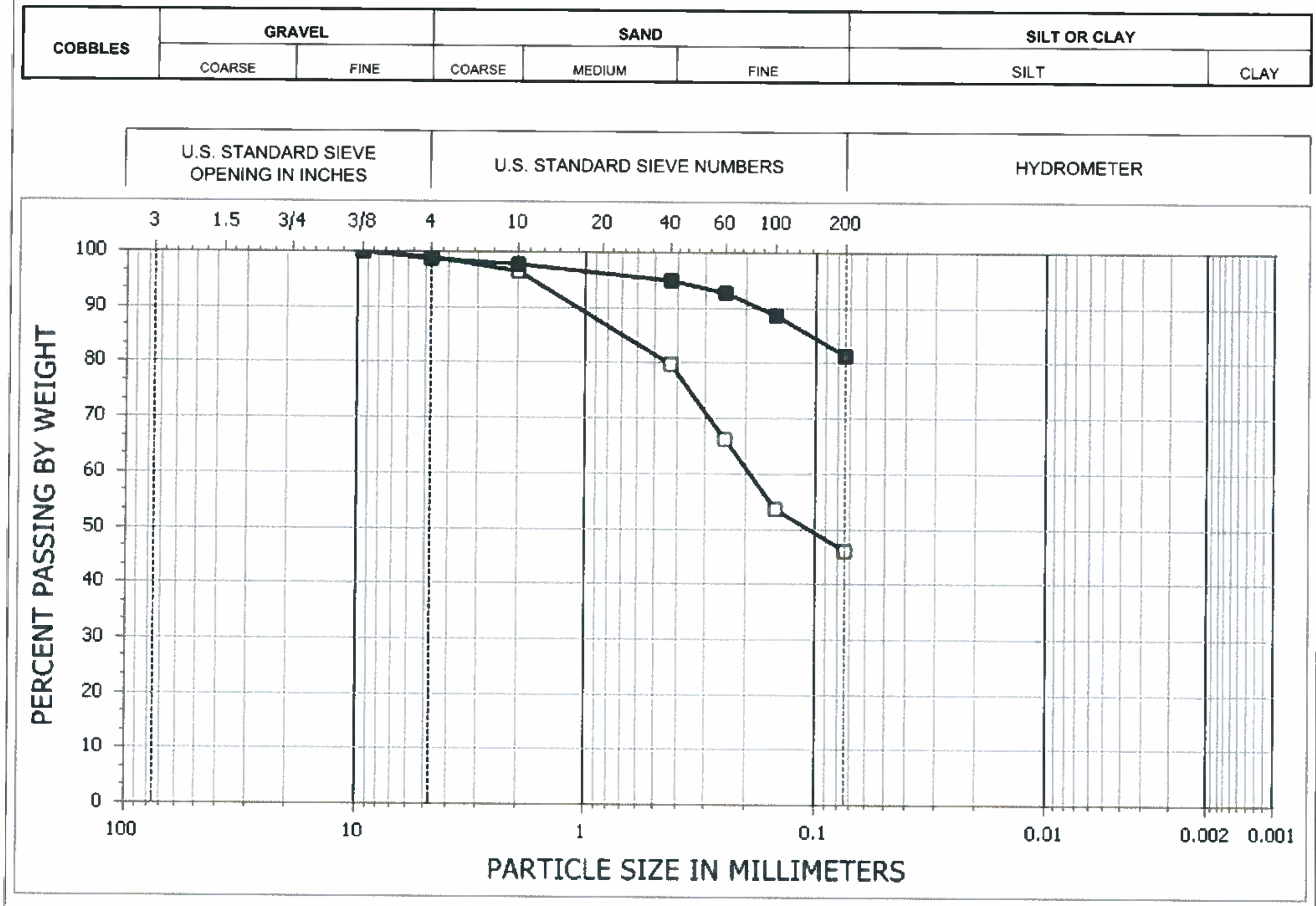
Project Engineer Steven J. Adamchak **Principal Engineer** Scott S. Stannard **Summary By** Steven J. Adamchak

Boring Number	Sample Number	Depth (feet)	MC[1] (%)	Soil Type[2]	Atterberg Limits[3]			Percent Passing No. 200 Sieve[4]	Moisture - Density (Corr.)[5]		CBR Value[6]	Other
					LL	PL	PI		Maximum Density (pcf)	Optimum Moisture (%)		
B-1	S-1	0.00 - 1.50	18.1									
B-1	S-3	5.00 - 6.50	18.9	SM				28.4				
B-1	S-5	13.50 - 13.75	27.5									
B-2	S-2	2.50 - 4.00	6.3									
B-2	S-4	8.50 - 10.00	22.2	CL	29	17	12	81.3				
B-3	S-2	2.50 - 4.00	6.5									
B-4	S-2	2.50 - 4.00	21.7									
B-4	S-4	8.50 - 10.00	18.2	SM				46.2				
B-5	S-1	0.00 - 1.50	9.1									
B-5	S-3	5.00 - 6.50	2.8	SM				47.6				

Notes: 1. ASTM D 2216, 2. ASTM D 2487, 3. ASTM D 4318, 4. ASTM D 1140, 5. See test reports for test method, 6. See test reports for test method

Definitions: MC: Moisture Content, Soil Type: USCS (Unified Soil Classification System), LL: Liquid Limit, PL: Plastic Limit, PI: Plasticity Index, CBR: California Bearing Ratio, OC: Organic Content (ASTM D 2974)

Grain Size (ASTM D 422) Test Summary



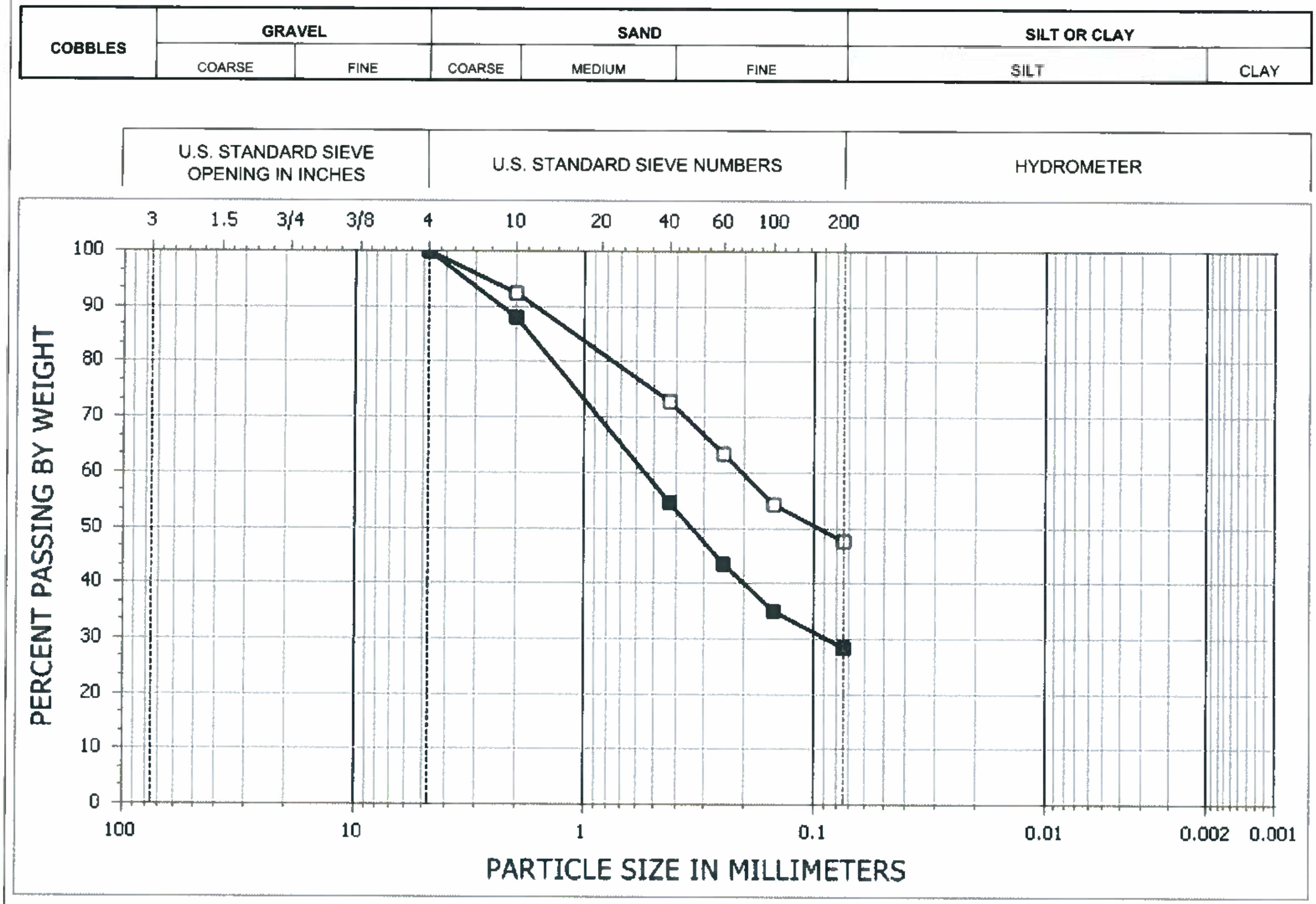
Boring Number Sample Number	Depth (feet)	Test Symbol	LL	PI	Description
B-4 / S-4	8.50 - 10.00	□	----	----	Silty Sand Tr/Mica Brown (SM)
B-2 / S-4	8.50 - 10.00	■	29	12	Lean Clay w/Sand Yellowish Brown (CL)

Project No.	17393
Project Name:	University of DC Student Union Building
PM:	Steven J. Adamchak
PE:	Scott S. Stannard
Printed on(date):	April 25, 2011

ECS Mid-Atlantic, LLC

Chantilly, VA

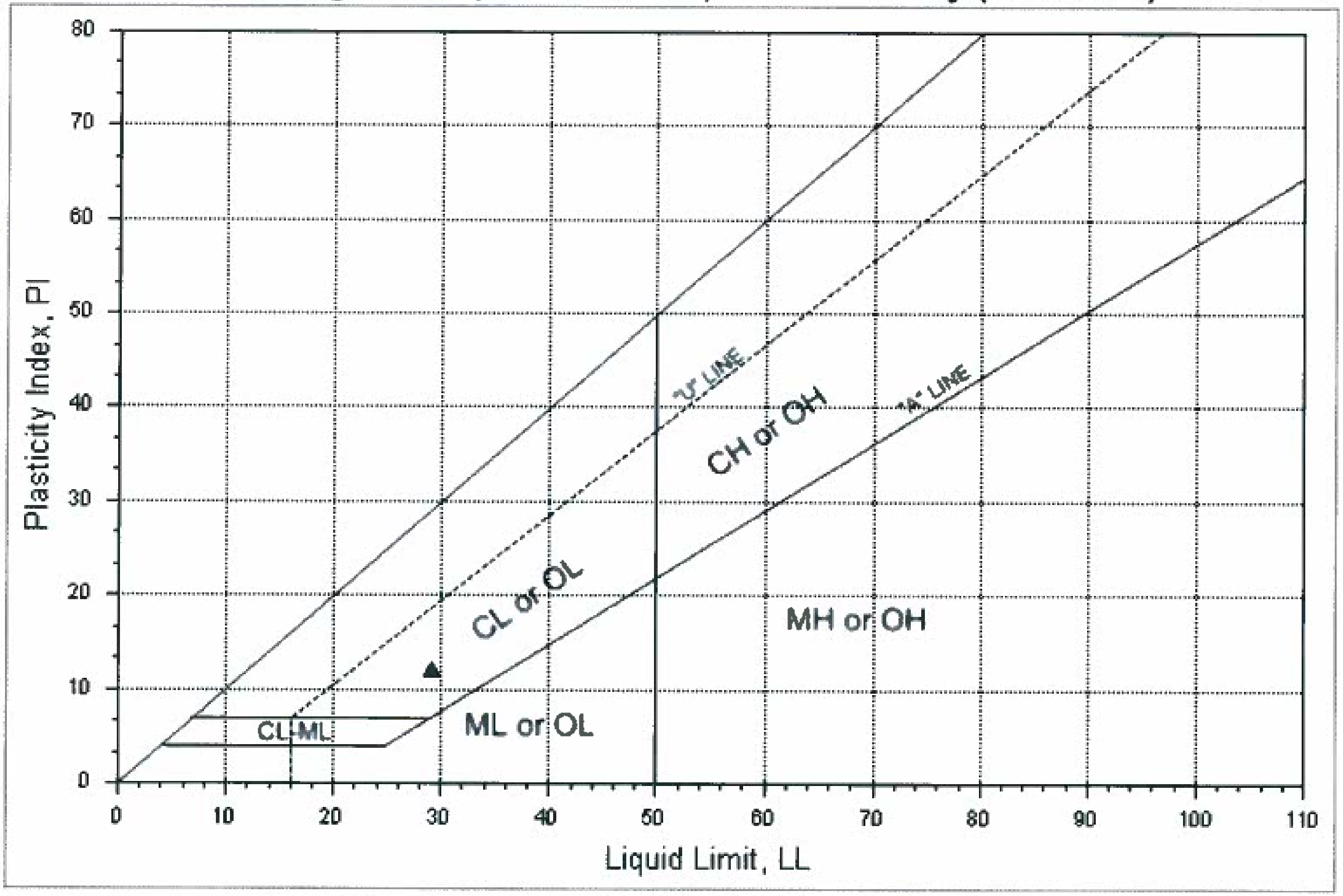
Grain Size (ASTM D 422) Test Summary



Boring Number Sample Number	Depth (feet)	Test Symbol	LL	PI	Description
B-5 / S-3	5.00 - 6.50	□	----	----	Silty Sand Brown (SM)
B-1 / S-3	5.00 - 6.50	■	----	----	Silty Sand Tr/Mica L/Brown (SM)

Project No.	17393	**ECS Mid-Atlantic, LLC**
Project Name:	University of DC Student Union Building	
PM:	Steven J. Adamchak	**Chantilly, VA**
PE:	Scott S. Stannard	
Printed on(date):	April 25, 2011	

Atterberg Limits (ASTM D 4318) Test Summary (Method A)



All samples are prepared using 'DRY' method unless otherwise noted

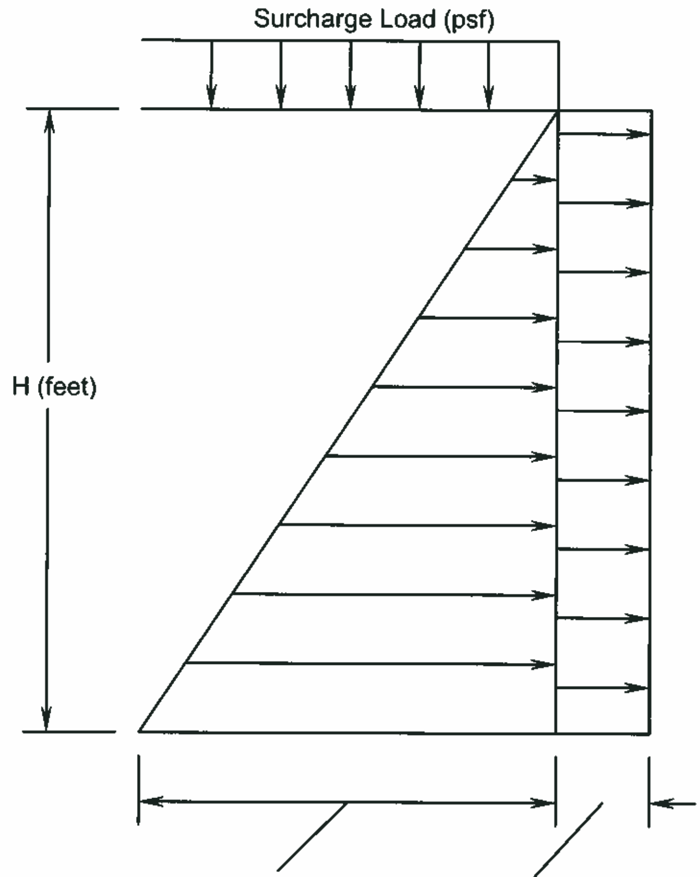
Boring Number Sample Number	Depth (feet)	Test Symbol	Description	MC (%)	LL	PL	PI	% Passing #200 Sieve	% Sample Retained on #40 Sieve	Notes
B-2 / S-4	8.50 - 10.00	▲	Lean Clay w/Sand Yellowish Brown (CL)	22.2	29	17	12	81.3	5.0	

Project No.	17393
Project Name:	University of DC Student Union Building
PM:	Steven J. Adamchak
PE:	Scott S. Stannard
Printed on(date):	April 25, 2011

ECS Mid-Atlantic, LLC

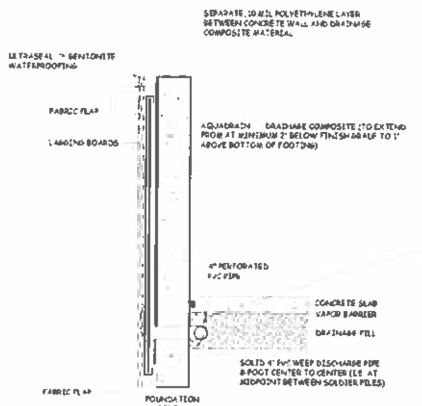
Chantilly, VA

LATERAL EARTH PRESSURE DIAGRAM

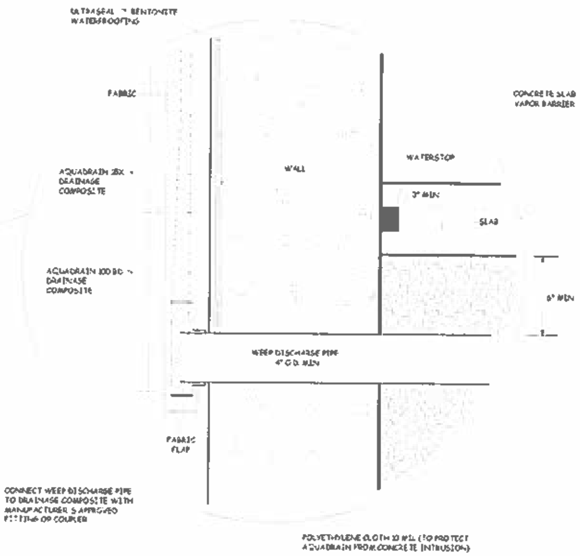


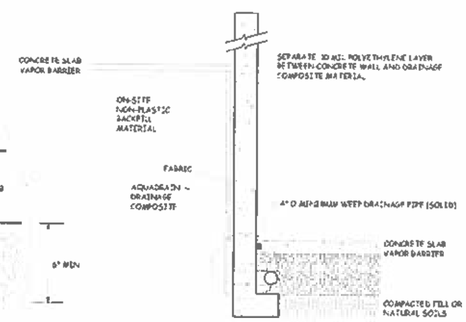
Lateral Earth Pressure = 60 H psf
(For below grade walls restrained from movement at top and bottom, drained conditions presumed)

Horizontal Pressure from Surcharge
= 0.5 x Vertical Surcharge



EXTERIOR WALL DRAIN DETAIL (TYPICAL) - DRAINED CONDITION (1.0)

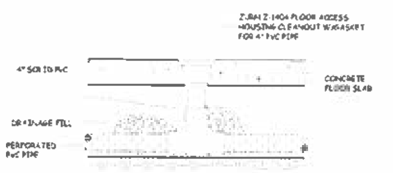




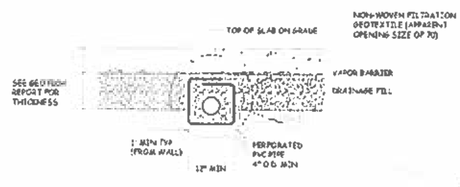
INTERIOR BELOW GRADE WALL DETAIL (2.0)

NOTES

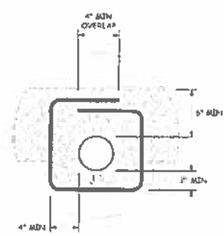
1) PRODUCTS SPECIFIED MAY BE SUBSTITUTED WITH AN EQUIVALENT PRODUCT, UPON REVIEW AND APPROVAL OF ECS
2) GEOTEXTILE FILTER FABRIC TO CONTACT LAGGING OR SOIL, NOT THE CONCRETE WALL
3) A GEOTEXTILE WRAPPED ADS DRAIN LINE MAY NOT BE SUBSTITUTED IN LIEU OF DETAIL 4.0
4) 4" MINIMUM DIAMETER WEEP HOLES (SOLID PVC) TO BE LOCATED AT 8 FOOT CENTER TO CENTER (I.E. AT MIDPOINT BETWEEN SOLDIER PILES, ADJUSTED IN THE FIELD AS REQUIRED)
5) DRAINAGE COMPOSITE ON EXTERIOR OF BELOW GRADE WALLS TO BE CONTINUOUS AROUND WALLS AND ALL SIDES OF EXCAVATION
6) SEE MANUFACTURER'S DETAIL FOR CONNECTION BETWEEN DRAINAGE PANELS
7) A NON-WOVEN FILTRATION GEOTEXTILE (MIRAFI 140N OR EQUIVALENT) SHOULD BE PLACED ON THE ENTIRE SUBGRADE IF STATED IN GEOTECHNICAL REPORT, PRIOR TO THE PLACEMENT OF THE UNDERSLAB DRAINAGE SYSTEM STONE. THE GEOTEXTILE SHOULD HAVE AN APPARENT OPENING SIZE OF 70 AND SHOULD BE PLACED IN ACCORDANCE WITH THE MANUFACTURER'S RECOMMENDATIONS. THE SAME FABRIC SHALL BE USED TO SURROUND THE PERFORATED DRAINAGE PIPE
8) MINIMUM 6 INCH GRAVEL COVER REQUIRED BETWEEN BOTTOM OF SLAB AND TOP OF PVC DRAIN PIPE. MIN. 2 INCHES OF GRAVEL MUST LIE BELOW THE INVERT OF ALL DRAIN PIPES
9) ULTRASEAL BENTONITE WATERPROOFING TO BE INSTALLED IN ACCORDANCE WITH MANUFACTURER'S RECOMMENDATIONS AND MEMBRANE TO BE HORIZONTALLY ORIENTED. ALTERNATIVE PRODUCT MAY BE SUBSTITUTED UPON REVIEW AND APPROVAL OF ECS
10) MAINTENANCE OF THE UNDERSLAB DRAINAGE SYSTEM WILL BE REQUIRED. THE BUILDING ENGINEER WILL NEED TO DERIVE A SCHEDULE OF MAINTENANCE
11) 4 INCH PERFORATED PVC DRAINS MUST BE SLOPED MINIMUM 0.5% TO APPROPRIATE DRAINAGE COLLECTION POINT

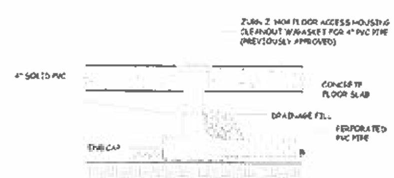


CLEANOUT DETAIL (TYPICAL) - PERFORATED PIPE (3.0)



UNDERSLAB DRAIN DETAIL (TYP) (4.0)





CLEANOUT DETAIL W/END CAP - PERFORATED PIPE (5.0)

DRAINAGE LAYER STEP DETAIL (TYP) (6.0)

BELOW-GRADE WALL WATERPROOFING AND UNDERSLAB DRAINAGE DIAGRAM

ECS LLC MID-ATLANTIC

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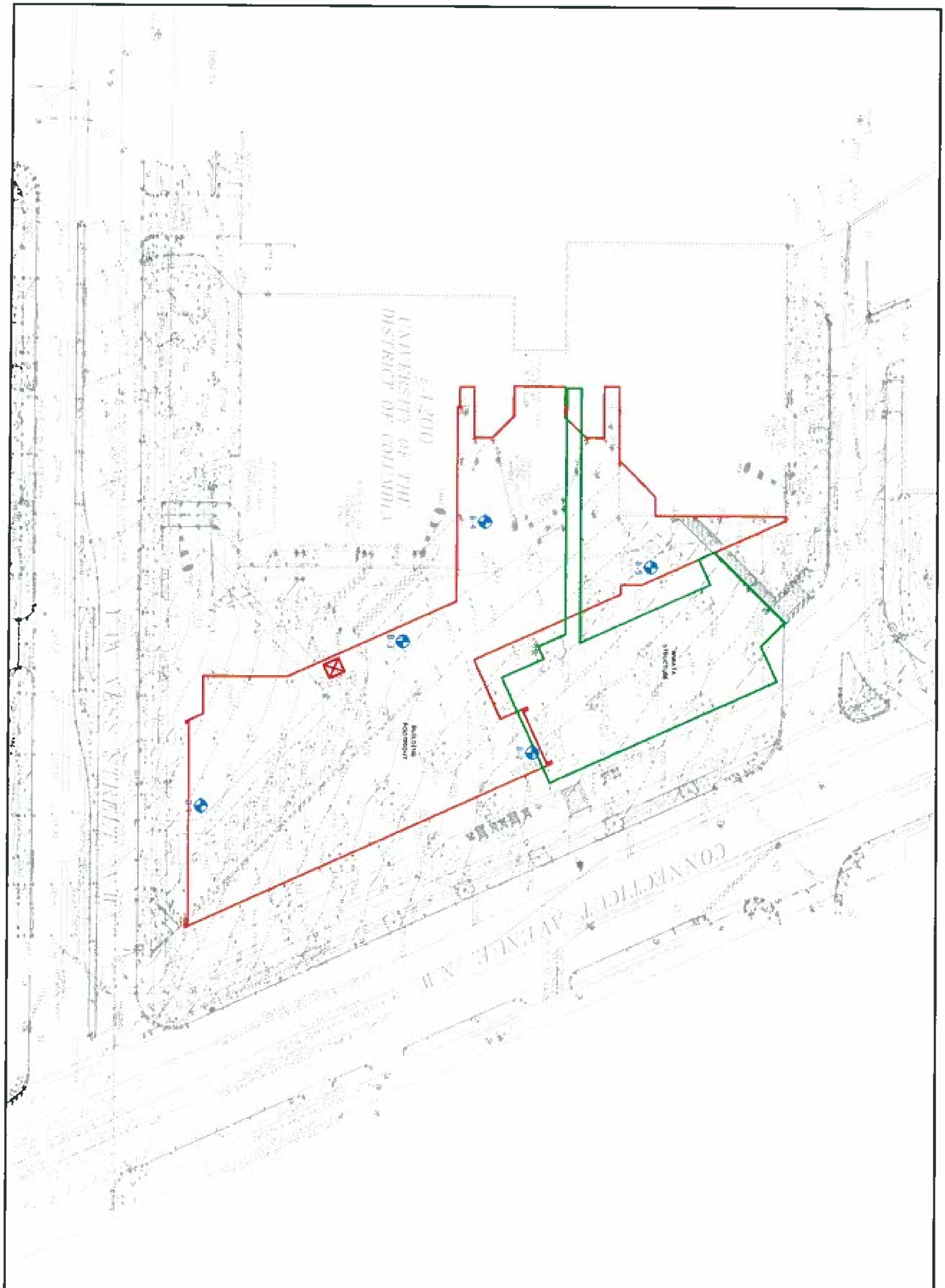

DATE	[illegible]-28-11
SHEET	1 OF 1
PROJECT NO	17393
SCALE	1"=50'
ENGINEER	SJA
DRAFTING	RAC
ECS REVISIONS	

BORING LOCATION DIAGRAM

UNIVERSITY OF DC

UNIVERSITY OF DC STUDENT UNION

WASHINGTON, DC

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SOIL CLASSIFICATION LEGEND

FILL - FILL (POSS/PROB) OF ALL TYPES
GW - WELL GRADED GRAVEL
GM - SILTY GRAVEL
GP - POORLY GRADED GRAVEL
GC - CLAYEY GRAVEL
SW - WELL GRADED SAND
ST - SHELBY TUBE
ML - LOW PLASTICITY SILT
CL - LOW PLASTICITY CLAY
MH - HIGH PLASTICITY SILT
RC - ROCK CORE
SM - SILTY SAND
SP - POORLY GRADED SAND
SC - CLAYEY SAND
PM - PRESSURE METER
CH - HIGH PLASTICITY CLAY
OH - HIGH PLASTICITY ORGANIC SILTS AND CLAYS
OL - LOW PLASTICITY ORGANIC SILTS AND CLAY
PT - PEAT
WR - WEATHERED ROCK

SURFACE MATERIALS

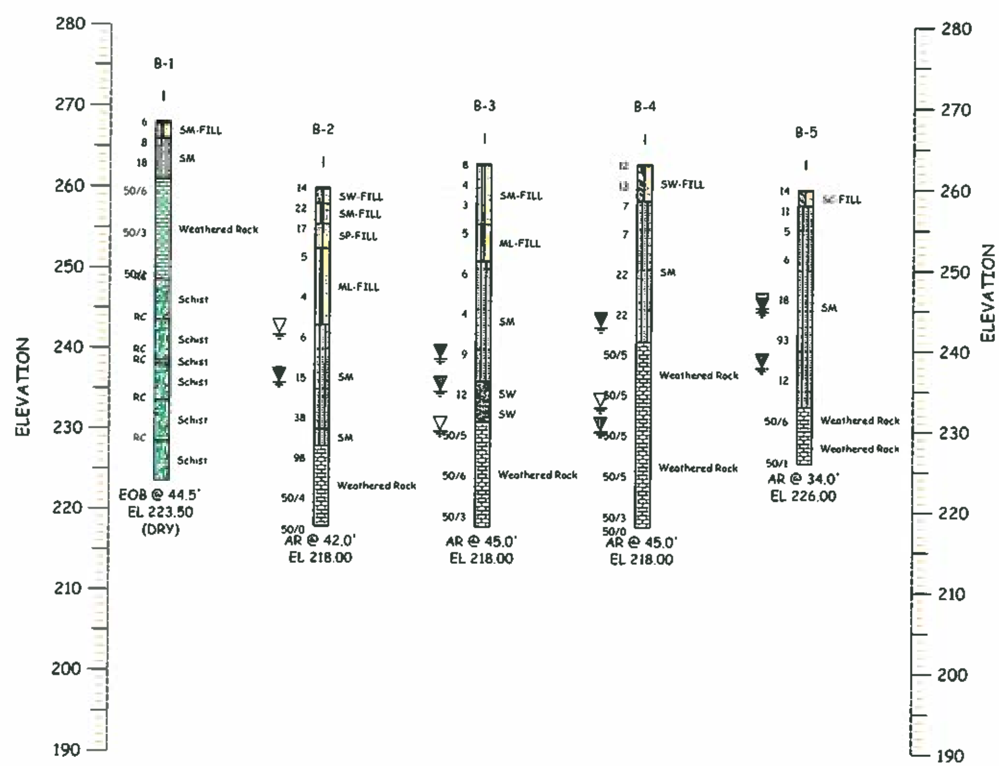
TOPSOIL
ASPHALT
GRAVEL
CONCRETE
VOID

ROCK TYPES

IGNEOUS
METAMORPHIC
SEDIMENTARY

SYMBOL LEGEND

WATER LEVEL - DURING DRILLING/SAMPLING
WATER LEVEL - BEFORE CASING REMOVAL
WATER LEVEL - AFTER CASING REMOVAL
WATER LEVEL - AFTER 24 HOURS



SCALE
VERTICAL SCALE 1"=15'

UNIVERSITY OF DC STUDENT UNION
WASHINGTON, DC

ECS

GENERALIZED SUBSURFACE PROFILE
UNIVERSITY OF DC

ECS REVISIONS

ENGINEER	DRAFTING
SJA	RAC
SCALE	AS NOTED
PROJECT NO	17393
SHEET	1 OF 1
DATE	03-28-11

Analysis

Several factors are considered when recommending the ACIP pile capacities. Specific factors for ACIP pile design for this project are described in the subsequent paragraph. The primary design factors for ACIP piles is the length of the pile and the diameter. ECS does not recommend altering our original pile length recommendations which were primarily based on the ability to auger through the weathered rock materials and the WMATA zone of influence concerns. As such, the reduced pile capacities are based on decreased pile diameters.

Foundation Recommendations

ACIP Piles Foundations

The following table summarizes our recommended pile designs.

Table 1: ACIP Pile Parameters

PILE DIAMETER (INCHES)	REINFORCING STEEL(1)	AXIAL PILE CAPACITY		GROUT STRENGTH (PSI)	ESTIMATED TIP ELEVATION(2) (FEET)
		AXIAL COMPRESSION (TONS; FS=2)	AXIAL TENSION (TONS, FS=3)		
14	4 #4 bars (upper 25 ft.) with #3 ties at 12" on-center, 1 #10 bar full length	90	10	4,000	+230 to +215
16	5 #4 bars (upper 25 ft.) with #3 ties at 12" on-center, 1 #10 bar full length	120	15	4,000	+230 to +215
18	6 #5 bars (upper 25 ft.) with #3 ties at 12" on-center, 1 #10 bar full length	150	30	4,000	+230 to +215

Notes: (1) The reinforcing steel provided in Table 1 accounts for geotechnical considerations only. More steel may be required for structural reasons.

(2) The estimated tip elevation should be refined after the completion of the recommended test pile program.

(3) Geotechnical Static computations indicate that the ACIP piles must be embedded a minimum of 15 feet into the weathered rock material.

The above table provides flexibility in the selection of the ACIP piles. Should more than one pile size be selected, we recommend that a standard load test be performed for each pile size selected at both the cellar level and ground level. The remainder of our ACIP pile recommendations provided in our Geotechnical Report should be utilized no matter which pile size is selected.