FROEHLING & ROBERTSON, INC.

Report of Subsurface Exploration and Geotechnical Engineering Evaluation

Brockman Lot 15 Development

Amherst, Virginia

F&R Project No. 62P0009

Prepared For:

The Hollingsworth Companies

Two Centre Plaza

Clinton, Tennessee 37716

Prepared By:

Froehling & Robertson, Inc.

1734 Seibel Drive, N.E.

Roanoke, Virginia 24012

Phone: 540.344.7939

Fax: 540.344.3657

May 2012

SINCE 1881

FROEHLING & ROBERTSON, INC.

Engineering Stability Since 1881

1734 Seibel Drive, NE
Roanoke, Virginia 24012-5624 I USA
T 540.344.7939 I F 540.344.3657

F&R Project No.: 62P0009 2 May 2012

The Hollingsworth Companies
Two Centre Plaza
Clinton, TN 37716

Attention: Mr. Tom Wortham
Senior Vice President of Architecture & Business Development

Subject: Brockman Lot 15 Development
Amherst, Virginia

Mr. Wortham:

The purpose of this report is to present the results of the subsurface exploration program and geotechnical engineering analyses undertaken by Froehling & Robertson, Inc. (F&R) in connection with the Brockman Lot 15 Development project. Our services were performed in general accordance with F&R Proposal No. 1362-004G as authorized by The Hollingsworth Companies. The attached report presents our understanding of the project, reviews our exploration procedures, describes existing site and general subsurface conditions, and presents our evaluations, conclusions, and recommendations.

We have enjoyed working with you on this project, and we are prepared to assist you with the recommended quality assurance monitoring and testing services during construction. Please contact us if you have any questions regarding this report or if we may be of further service.

Sincerely,
FROEHLING & ROBERTSON, INC.

COMMONWEALTH OF VIRGINIA
ANDREW R. FRANK
Lic. No. 33731
5/2/12
PROFESSIONAL ENGINEER

Ben W. Silcox, E.I.T.
Staff Engineer

Andrew R. Frank, P.E.
Senior Geotechnical Engineer

Distribution: Addressee (1 original, 1 copy via e-mail: twortham@hollingsworthcos.com)

F:\Projects 62P\62P-0009 (Brockman Lot 15 Dev)\62P0009 Brockman Lot 15 - RPT_.doc

TABLE OF CONTENTS

APPENDICES

EXECUTIVE SUMMARY

This Executive Summary is provided as a brief overview of our geotechnical engineering evaluation for the project and is not intended to replace more detailed information contained elsewhere in this report. As an overview, this summary inherently omits details that could be very important to the proper application of the provided geotechnical design recommendations. This report should be read in its entirety prior to implementation into design and construction.

- The subsurface exploration program consisted of six test borings (designated as W-1 through W-6) performed on 13 April 2012. Site subsurface conditions generally consisted of surficial soils underlain by residual soils.
- The proposed building may be supported on a shallow foundation system designed for a maximum allowable bearing pressure of 2,500 pounds per square foot (psf) for footings bearing on approved residual soil or newly placed controlled structural fill subgrades. To reduce the possibility of localized shear failures, spread and strip footings should be a minimum of 3 feet and 2 feet wide, respectively.
- Based on the conditions encountered during our subsurface exploration, laboratory testing results, and our general experience in the vicinity, foundation supporting soils could have a moderate to high shrink-swell potential. Accordingly, we recommend that exterior footings be constructed at least 3 feet below adjacent exterior finished grades in order to reduce the effect of surface water migration into potentially highly plastic soils at the foundation bearing level and to bear below the normal frost depth of 2 feet.
- Based on the boring data as well as provided topographic, proposed grading and structural load information, we estimate total settlements on the order of 1 inch or less, with differential settlement of ½ to ⅔ the estimated total settlement. The magnitude of differential settlements will be influenced by the variation in excavation requirements across the building footprint, the distribution of loads, and the variability of underlying soils.
- Although no site retaining walls have been indicated, we envision that some loading dock walls may be required. Based on the current and previous subsurface exploration programs, the anticipated cut areas of the site will generally consists of soils described as clays and silts with moderate to high plasticity. Therefore, we suggest that a select cohesionless backfill material consisting of VDOT No. 57 crushed stone be considered.
- Based on the boring data and in general accordance with the IBC, a Site Class "D" should be used to develop the project's Seismic Design Category for further evaluations relative to Earthquake Load design.

1.0 INTRODUCTION

1.1 Project Information

Our understanding of the project is based on information provided by Mr. Tom Wortham of The Hollingsworth Companies, our previous subsurface exploration experience at the site, and our experiences with similar projects. The proposed project will consist of the development of Lot 15 within the Brockman Business Park in Amherst, Virginia (see Site Vicinity Map, Drawing No. 1).

Included in the provided information was an e-mailed drawing (filename: Boring Location Sketch.pdf) that portrayed site topography (10-foot contours), spot finished grades, and the requested locations of six test borings. The provided drawing indicates that the proposed project will consist of an approximate 360 feet by 300 feet building with parking and driveways to the west and south of the planned building. Based on discussion with Mr. Wortham, we understand that the proposed structure is a pre-fabricated metal building with maximum column and continuous wall loads on the order of 60 kips and less than 2 kips per linear foot, respectively.

Based on the provided drawing, existing site grades appear to range from an estimated 705 feet to 690 feet in the proposed building area and about 682 feet to 700 feet in proposed parking areas. The provided drawing further indicates a planned finished floor elevation (FFE) of 696 feet and spot pavement grades to the west of the building of 690 feet to 692 feet. Based on the provided topographic and spot grading information, it appears that maximum cuts and fills on the order of 10 feet will be needed to develop the proposed spot grades. Furthermore, it appears maximum fill depths of about 6 feet will be required within the proposed building footprint.

1.2 Previous Subsurface Information

F&R performed a previous preliminary subsurface exploration at the site which consisted of six test borings (designated as B-1 through B-6) as well as some associated laboratory testing. We note that previous borings B-1 through B-3 are within planned development areas for the current project. The preliminary exploration was performed in October 2003 under the name *Brockman Business Park – Right Now Site (Site #1)* and F&R Project No. E62-203G. We have included the Boring Location Plan, Boring Logs, and laboratory test results from the 2003 preliminary subsurface exploration program in Appendix C. Qualification of the previously performed test borings and laboratory testing can be found in the October 2003 report.

Historic subsurface information (especially borings B-1 through B-3) in conjunction with the current exploration test borings and our experience at the project site and in the vicinity, were used to develop geotechnical recommendations for the proposed Brockman Lot 15 Development project. In utilizing the previously explored test boring data, the owner and others should realize that although individual test borings are representative of the subsurface conditions at a given boring location on the date shown, it is not necessarily indicative of the subsurface conditions at other locations or at other times.

1.3 Scope of Services

The purposes of our involvement on this project were to 1) provide general descriptions of the subsurface soil conditions encountered at the locations explored, 2) provide foundation design recommendations, and 3) comment on other geotechnical aspects of the proposed development. In order to accomplish the above objectives, we undertook the following scope of services:

1) Visited the site to observe existing surface conditions and features and mark boring locations.
2) Coordinated utility clearance with Miss Utility services.
3) Reviewed and summarized readily available geologic information relative to the project site.
4) Executed a subsurface exploration consisting of six test borings performed in the vicinity of the requested locations; four within the proposed new building footprint and two in planned pavement areas. The building borings were drilled to the planned depth of 25 feet each while the pavement borings were drilled to planned depths of 10 feet and 15 feet (125 linear feet total).
5) Performed one soil classification (Atterberg limits and wash #200) test on a selected split-spoon sample to aid in visual-manual classifications.
6) Provided a Seismic Site Class Definition per the 2009 International Building Code (IBC) based on interpretation of the Standard Penetration Test data.
7) Evaluated the findings of the test borings and laboratory test results relative to shallow foundation design, lateral earth pressure recommendations, and provided appropriate design criteria.
8) Prepared this written report summarizing our work on the project, providing foundation design recommendations as well as lateral earth design criteria, and discussing geotechnical related aspects of the proposed construction. Copies of the test boring logs and laboratory test results are included in the attached Appendices.

Our geotechnical scope of services did not include survey services, quantity estimates, preparation of plans or specifications, formal slope stability analyses, pavement design, detention pond considerations, evaluations of earthquake motions, or the identification and evaluation of wetland or other environmental aspects of the project site.

2.0 SUBSURFACE EXPLORATION PROCEDURES

The subsurface exploration program consisted of six test borings; four borings within the proposed new building footprint (designated as W-1 through W-4), and two borings in the planned pavement areas (designated as W-5 and W-6) in the general locations requested by The Hollingsworth Companies. The borings were performed on 13 April 2012 at the approximate locations shown on the attached Boring Location Plan (Drawing No. 2, Appendix B).

F&R personnel marked the boring locations in the field using a hand-held Global Positioning System (GPS) unit. Ground surface elevations at the borings locations were interpolated from the provided existing topographic information. No claim is made as to the accuracy of the information contained in the provided documents or the variable accuracy of the hand-held GPS unit. In consideration of the methods used in their determination, the test boring locations and elevations shown on the attached Boring Location Plan, Composite Subsurface Profile (Drawing No. 3), and Boring Logs should be considered approximate.

The test borings were performed in accordance with generally accepted practice using a truck-mounted CME-55 rotary drill rig equipped with an automatic hammer. Hollow-stem augers were advanced to pre-selected depths, the center plug was removed, and representative soil samples were recovered with a standard split-spoon sampler (1 3/8 in. ID, 2 in. OD) in general accordance with ASTM D 1586, the Standard Penetration Test (SPT). Utilizing an automatic hammer, a weight of 140 pounds is freely dropped from a height of 30 inches to drive the split-spoon sampler into the soil. The number of blows required to drive the split-spoon sampler three consecutive 6-inch increments is recorded, and the blows of the last two increments are summed to obtain the Standard Penetration Resistance (N-value). The N-value provides a general indication of in-situ soil conditions and has been correlated with certain engineering properties of soils.

In some soils it is not always practical to drive a split-spoon sampler the full three consecutive 6-inch increments. Whenever more than 50 blows are required to drive the sampler over a 6-inch increment, or the sampler is observed not to penetrate after 50 blows, the condition is called split-spoon refusal. Split-spoon refusal conditions may occur because of obstructions or because the earth materials being tested are very dense or very hard. When split-spoon refusal occurs, often little or no sample is recovered. The SPT N-value for split-spoon refusal conditions is typically estimated as greater than 100 blows per foot (bpf). Where the sampler is observed not to penetrate after 50 blows, the N-value is reported as 50/0. Otherwise, the depth of penetration after 50 blows is reported in inches, i.e. 50/5, 50/2, etc.

Subsurface water level readings were taken in each of the borings immediately upon completion of the soil drilling process. Upon completion of drilling, the boreholes were backfilled with auger cuttings (soil). Periodic observation and maintenance of the boreholes should be performed due to potential subsidence at the ground surface, as the borehole backfill could settle over time.

Representative portions of the split-spoon soil samples obtained throughout the exploration program were placed in glass jars and transported to our laboratory. In the laboratory, the soil samples were classified by a member of our professional staff in general accordance with techniques outlined in the visual-manual identification procedure (ASTM D 2488) and the Unified Soil Classification System. The soil descriptions and classifications discussed in this report and shown on the attached boring logs are generally based on visual observation and should be considered approximate.

Copies of the boring logs are provided and classification procedures are further explained in the attached Appendix B. Split-spoon soil samples recovered on this project will be stored at F&R's office for a period of sixty days. After sixty days, the samples will be discarded unless prior notification is provided to us in writing.

3.0 SITE AND SUBSURFACE CONDITIONS

3.1 Site Description

The project site is located at the end of Brockman Park Drive in the Brockman Business Park, which exists on the north side of Route 60, approximately ¾ of a mile southeast of its intersection with Route 29 in Amherst, Virginia. The site is generally grass-covered along its southern portion and heavily wooded along its northern portion. Based on the provided drawing, existing site grades appear to range from an estimated 705 feet to 690 feet in the proposed building area and about 682 feet to 700 feet in the proposed parking areas.

3.2 Regional Geology

The site lies within the Blue Ridge physiographic province of Virginia. Available geologic references report that the proposed site is underlain by Middle Proterozoic (Early or Pre-Grenville) aged rocks generally consisting of layered quartzofeldspathic augen gneiss and flasher gneiss. The soils resulting from in-situ weathering of the rocks, without significant transportation, are called residual soils.

The residual soil profile generally grades downward gradually from fine-grained plastic soils near the ground surface to coarse-grained soils at greater depth. A transitional zone of partially weathered rock of varying thickness occurs between the coarse-grained residual soils and the underlying bedrock. Partially weathered rock is defined, for engineering purposes, as residual material with standard penetration resistances in excess of 100 blows per foot. Weathering of the parent bedrock is generally more rapid near fracture zones and therefore, the bedrock surface may be irregular. Irregular patterns of differential weathering may also result in zones of rock and partially weathered rock embedded within the more completely weathered coarse-grained soils.

3.3 Subsurface Conditions

3.3.1 General

The subsurface conditions discussed in the following paragraphs and those shown on the boring logs represent an estimate of the subsurface conditions based on interpretation of the boring data using normally accepted geotechnical engineering judgments. The transitions between different soil strata are usually less distinct than those shown on the boring logs and subsurface profile. Although individual test borings are representative of the subsurface conditions at the boring locations on the dates shown, they are not necessarily indicative of subsurface conditions at other locations or at other times. Data from the specific test borings for the current study are shown on the attached boring logs in Appendix B. In addition, a composite subsurface profile has been provided to conceptually illustrate conditions encountered across the site. Boring data from the specific test borings for the 2003 study as well as associated laboratory testing results are provided in Appendix C.

Below the existing ground surface, the borings generally encountered surficial soils underlain by residual soils with a lens of partially weathered rock in boring W-1. These materials are generally discussed in the following paragraphs.

3.3.2 Surficial Soils

Surficial soils were encountered in each of the borings to a depth of approximately 2 to 4 inches. Surficial soils are typically a dark-colored soil material containing roots, fibrous matter, and/or other organic components, and are generally unsuitable for engineering purposes. We note that no laboratory testing has been performed to determine the organic content or horticultural properties of the observed surficial soil materials. Therefore, the term "surficial soils" is not intended to indicate suitability for landscaping and/or other purposes. The surficial soil depths provided in this report are based on driller observations and should be considered approximate. Actual surficial soil depths should be expected to vary across the site.

3.3.3 Residual Soils

Residual soils, formed by the in-place weathering of the parent rock, were encountered below the surficial soils in each of the borings. Sampled residual soils were generally described as clays (CL/CH), silts (ML/MH), and sands (SM). Standard penetration resistances within the sampled residuum ranged from 6 to 29 blows per foot (bpf) with a typical range of 8 to 18 bpf.

3.3.4 Partially Weathered Rock

Partially weathered rock (PWR) is a transitional material between soil and rock, which retains the relic structure of the rock and has very hard or very dense consistencies. PWR was encountered as an approximate 5-foot thick lens within the residual soil profile at a depth of approximately 17 feet below existing site grade in boring W-1. The sampled PWR was described as very dense silty sand (SM) with a penetration resistance of 50 blows per 4 inches (50/4) of split-spoon penetration. We note that PWR was also encountered in previous boring B-6, but this boring is located outside of the currently planned development area.

3.3.5 Subsurface Water

Subsurface water for the purposes of this report is defined as water encountered below the existing ground surface. Measurable subsurface water was not encountered in any of the test borings immediately upon completion of the soil drilling process. Fluctuations in subsurface water levels and soil moisture can be anticipated with changes in precipitation, run-off, and season.

3.4 Laboratory Testing Program

Laboratory testing was performed in general accordance with applicable ASTM International (ASTM) standards. A split-spoon sample from boring W-3 was tested for moisture content (ASTM D 2216), Atterberg limits (ASTM D 4318), and percent passing #200 sieve (ASTM D 1140). The results of the laboratory tests are summarized in the following table.

USCS Soil Classification Test Summary

Boring No.	Sample Depth (feet)	Sample Type	Moisture Content (%)	% Finer than No. 200	Atterberg Limits			USCS Classification
					L.L.	P.L.	P.I.	
W-3	3.5 – 5	SS	39.9	76.7	62	49	13	Red brown elastic SILT (MH) with sand

4.0 DESIGN RECOMMENDATIONS

4.1 General

The following evaluations and recommendations are based on our observations at the site, interpretation of the field and laboratory data obtained during this exploration as well as the previous 2003 exploration, and our experience with similar subsurface conditions and projects. Soil penetration data has been used to develop an allowable bearing pressure and estimate associated settlements using established correlations. Subsurface conditions in unexplored locations may vary from those encountered. If structure locations, loadings, or elevations are changed, we should be notified and requested to confirm and, if necessary, re-evaluate our recommendations.

Determination of an appropriate foundation system for a given structure is dependent on the proposed structural loads, soil conditions, and construction constraints such as proximity to other structures, etc. The subsurface exploration aids the geotechnical engineer in determining the soil stratum appropriate for structural support. This determination includes considerations with regard to both allowable bearing capacity and compressibility of the soil strata. In addition, since the method of construction greatly affects the soils intended for structural support, consideration must be given to the implementation of suitable methods of site preparation, fill compaction, and other aspects of construction.

4.2 Foundation Design

Based on the structural information discussed in the project information section of this report, the proposed building may be supported on a shallow foundation system bearing on approved undisturbed residual soils or newly placed controlled structural fill (see Section 5.3, Controlled Structural Fill recommendations). Based on the anticipated structural loads and the subsurface conditions encountered in our test borings, we recommend that foundations be designed for a maximum allowable bearing pressure of 2,500 pounds per square foot (psf) for footings bearing on approved subgrades. To reduce the possibility of localized shear failures, spread and strip footings should be a minimum of 3 feet and 2 feet wide, respectively.

4.3 Shrink-Swell and Frost Depth Considerations

Based on the conditions encountered during our subsurface exploration, laboratory testing results, and our general experience in the vicinity, foundation supporting soils could have a moderate to high shrink-swell potential. Accordingly, we recommend that exterior footings be constructed at least 3 feet below adjacent exterior finished grades in order to reduce the effect of surface water migration into potentially highly plastic soils at the foundation bearing level and to bear below the normal frost depth of 2 feet.

4.4 Estimated Settlements

Based on the boring data as well as provided topographic, proposed grading and structural load information, we estimate total settlements on the order of 1 inch or less, with differential settlement of ½ to ⅔ the estimated total settlement. The magnitude of differential settlements will be influenced by the variation in excavation requirements across the building footprint, the distribution of loads, and the variability of underlying soils.

Our settlement analysis was performed on the basis of structural and grading assumptions discussed in the project information section of this report. Actual settlements experienced by the structure and the time required for these soils to settle will be influenced by undetected variations in subsurface conditions, actual structural loads, final grading plans, and the quality of fill placement and foundation construction.

4.5 Ground Floor Slabs

Ground floor slabs may be designed as a slab-on-grade supported by undisturbed residual soils or newly placed controlled structural fill subgrades. Slab-on-grade support is contingent upon successful completion of the subgrade evaluation process as described in Site Preparation (Section 5.1).

A vapor retarder should be used beneath ground floor slabs that will be covered by tile, wood, carpet, impermeable floor coatings, and/or if other moisture-sensitive equipment or materials will be in contact with the floor. However, the use of vapor retarders may result in excessive curling of floor slabs during curing. We refer the floor slab designer to ACI 302.1R-96, Sections 4.1.5 and 11.11, for further discussion on vapor retarders, curling, and the means to reduce concrete shrinkage and curling.

Proper jointing of the ground floor slab is also essential to minimize cracking. ACI suggests that unreinforced, plain concrete slabs may be jointed at spacings of 24 to 36 times the slab thickness, up to a maximum spacing of 18 feet. Floor slab construction should incorporate isolation joints along bearing walls and around column locations to allow minor movements to occur without damage. Utility or other construction excavations in the prepared floor subgrade should be backfilled to a controlled fill criterion to provide uniform floor support.

4.6 Lateral Earth Pressures

While no planned site retaining walls have been indicated at this time, we envision that some loading dock walls may be required, and therefore, the following information is provided to aid in analysis of soil loads on below grade walls.

Earth pressures on walls below grade are influenced by structural design of the walls, conditions of wall restraint, methods of construction and/or compaction, and the strength of the materials being restrained. The most common conditions assumed for earth retaining wall design are the active and at-rest conditions. Active conditions apply to relatively flexible earth retention structures, such as freestanding walls, where some movement and rotation may occur to mobilize soil shear strength. Walls that are rigidly restrained, such as basement, pit, and tunnel walls, require design using at-rest earth pressures.

A third condition, the passive state, represents the maximum possible pressure when a structure is pushed against the soil, and is used in wall foundation design to help resist active or at-rest pressures. Because significant wall movements are required to develop the passive pressure, the total calculated passive pressure should be reduced for design purposes.

Based on the current and previous subsurface exploration programs, the anticipated cut areas of the site will generally consists of soils described as clays and silts with moderate to high plasticity. We do not recommend the use of the onsite clays and silts as backfill behind the loading or other retaining walls. We recommend that a low-plasticity and/or granular material be used as backfill behind retaining walls. Since an applicable borrow source has not been identified at the time this report was written and because structural design requires the use of established earth pressure parameters prior to construction, we suggest that a select cohesionless backfill material consisting of VDOT No. 57 crushed stone be considered. The select material should be extended laterally from back heel of the wall footing, a minimum distance of 0.5 times the wall height at the top of the wall (see Extent of Select Backfill for Retaining Walls, Drawing No. 4, Appendix D).

Number 57 crushed stone should be placed in lifts no greater than 2 feet in thickness and compacted with a backhoe bucket or similar. In addition, we recommend that the No. 57 crushed stone backfill be placed using a separation geotextile at the interface between the coarse-grained crushed stone backfill and existing residual/new fill soils.

The recommended lateral earth pressure coefficients and equivalent fluid pressure parameters for design of retaining or below grade walls using a select VDOT No. 57 crushed stone backfill are provided in the following table:

VDOT No. 57 CRUSHED STONE

Earth Pressure Conditions	Coefficient	Recommended Equivalent Fluid Pressure (pcf)
Active (K_a)	0.22	24
At-Rest (K_o)	0.36	40

* A crushed stone unit weight of 110 pounds per cubic foot should be used for design calculations.

Our recommendations assume that the ground surface above the wall is level. The recommended equivalent fluid pressures assume that constantly functioning drainage systems are installed between walls and crushed stone backfill to prevent the accidental buildup of hydrostatic pressures and lateral stresses in excess of those stated. If a functioning drainage system is not installed, then lateral earth pressures should be determined using the buoyant weight of the soil. Hydrostatic pressures calculated with the unit weight of water (62.4 pcf) should be added to these earth pressures to obtain the total stresses for design.

Regardless of the select backfill material chosen, the following friction and passive earth pressure coefficients are provided for use in evaluating the foundation member's resistance to sliding in the in-situ soils at the site. Based on our experience with similar subsurface conditions, we recommend a coefficient of friction value of 0.34 between foundation concrete and underlying soil subgrade. For soils similar to those encountered in the test boring, we recommend a passive earth pressure coefficient of $K_p = 2.77$ and a moist unit weight of 120 pounds per cubic feet (pcf). Please note that significant movement is required to develop the passive pressure. Therefore, the total calculated passive pressure should be reduced by one-half to two-thirds for design purposes.

Heavy equipment should not operate within 5 feet of below grade walls to prevent lateral pressures in excess of those cited. If footings or other surcharge loads are located a short distance outside below grade walls, they may also exert appreciable additional lateral pressures. Surcharge loads should be evaluated using the appropriate active or at-rest pressure coefficients provided above. The effect of surcharge loads should be added to the recommended earth pressures to determine total lateral stresses.

These retaining/below grade wall recommendations should not be correlated for use in the design of mechanically stabilized earth (MSE) walls. We recommend that soil parameters for any MSE wall design be established through appropriate laboratory testing directed by the wall designer.

4.7 Seismic Site Class Definition

The following recommendations are based on Sections 1613.5.2 and 1613.5.5 of the 2009 International Building Code (IBC). Our scope of services did not include a seismic conditions survey to determine site-specific shear wave velocity information. IBC provides a methodology for interpretation of Standard Penetration Test resistance values (N-values) to determine a Site Class Definition. However, this method requires averaging N-values over the top 100 feet of the subsurface profile. We note that the test borings for this project were extended to a maximum depth of 25 feet below existing site grades.

The available subsurface data from our exploration indicates an N-value range of about 6 to greater than 100 bpf within the upper 25 feet below existing site grades. Based on the boring data and in general accordance with sections 1613.5.2 and 1613.5.5 of the IBC, a Site Class Definition "D" may be used to develop the project's Seismic Design Category for further evaluations relative to Earthquake Load design.

We note that the above provided Site Class Definition is based on information available at the time this report was written. Should this classification be so onerous to the project cost that further study is warranted, we can perform a site-specific geo-physical survey to attain sufficient detail to refine the project's seismic Site Class Definition. This additional testing would be beyond the currently authorized scope of services for this project.

5.0 CONSTRUCTION RECOMMENDATIONS

5.1 Site Preparation

Before proceeding with construction, any surficial soils, roots, and any other deleterious non-soil materials should be stripped or removed from the proposed construction area. During the clearing and stripping operations, positive surface drainage should be maintained to prevent the accumulation of water. Underground utilities should be re-routed to locations a minimum of 10 feet outside of the proposed new structure footprint.

After stripping, areas intended to support new fill, pavements, floor slabs, and foundations should be carefully evaluated by a geotechnical engineer. At that time, the engineer may require proofrolling of the subgrade with a 20- to 30-ton loaded truck or other pneumatic-tired vehicle of similar size and weight. Proofrolling should be performed during a time of good weather and not while the site is wet, frozen, or severely desiccated. The purpose of the proofrolling is to locate soft, weak, or excessively wet soils present at the time of construction.

The proofrolling observation is a good opportunity for the geotechnical engineer to locate inconsistencies intermediate of the test boring locations in the existing subgrade. Any unsuitable materials observed during the evaluation and proofrolling operations should be undercut and replaced with compacted fill and/or stabilized in-place. The possible need for, and extent of, undercutting and/or in-place stabilization required can best be determined by the geotechnical engineer at the time of construction. Once the site has been properly prepared, at-grade construction may proceed.

5.2 Foundation Construction

All foundation subgrades should be observed, evaluated, and verified for the design bearing pressure by the geotechnical engineer after excavation and prior to reinforcement steel placement. If low consistency soils are encountered during foundation construction, localized undercutting and/or in-place stabilization of foundation subgrades may be required. The actual need for, and extent of, undercutting should be based on field observations made by the geotechnical engineer at the time of construction.

Excavations for footings should be made in such a way as to provide bearing surfaces that are firm and free of loose, soft, wet, or otherwise disturbed soils. Foundation concrete should not be placed on frozen or saturated subgrades. If such materials are allowed to remain below foundations, settlements will increase. Foundation excavations should be concreted as soon as practical after they are excavated. If an excavation is left open for an extended period, a thin mat of lean concrete should be placed over the bottom to minimize damage to the bearing surface from weather or construction activities. Water should not be allowed to pond in any excavation.

5.3 Controlled Structural Fill

Based on the boring data, controlled structural fill may be constructed using the non-organic on-site soils. If needed, off-site borrow materials should generally have a classification of CL, ML, SM, or SC as defined by the Unified Soil Classification System (USCS). Other materials may be suitable for use as controlled structural fill material and should be individually evaluated by the geotechnical engineer. Controlled structural fill should be free of boulders, organic matter, debris, or other deleterious materials and should have a maximum particle size no greater than 3 inches. In addition, we recommend a minimum standard Proctor (ASTM D 698) maximum dry density of 90 pounds per cubic feet for fill materials.

Fill materials should be placed in horizontal lifts with maximum height of 8 inches loose measure. New fill should be adequately keyed into stripped and scarified subgrade soils and should, where applicable, be benched into the existing slopes. During fill operations, positive surface drainage should be maintained to prevent the accumulation of water. We recommend that structural fill be compacted to at least 95 percent of the standard Proctor maximum dry density. In confined areas such as utility trenches, portable compaction equipment and thin lifts of 3 to 4 inches may be required to achieve specified degrees of compaction. Each lift of fill should be tested in order to confirm that the recommended degree of compaction is attained.

In general, we recommend that the moisture content of fill materials be maintained within three percentage points of the optimum moisture content as determined from the standard Proctor density test. We recommend that the contractor have equipment on site during earthwork for both drying and wetting of fill soils. Moisture control may be especially difficult during winter months or extended periods of rain. Attempts to work the soils when wet can be expected to result in deterioration of otherwise suitable soil conditions or of previously placed and properly compacted fill. Where construction traffic or weather has disturbed the subgrade, the upper 8 inches of soils (or more if warranted) intended for structural support should be scarified and re-compacted. Each lift of fill should be tested in order to confirm that the recommended degree of compaction is attained.

5.4 Subsurface Water Conditions

Subsurface water for the purposes of this report is defined as water encountered below the existing ground surface. Based on the subsurface water data obtained during our exploration program, we generally anticipate that subsurface water will not be encountered during anticipated earthwork or shallow foundation excavations at the site. However, the contractor should be prepared to dewater should water levels vary from those encountered during the drilling program. Fluctuations in subsurface water levels and soil moisture can be anticipated with changes in precipitation, runoff, and season.

6.0 CONTINUATION OF SERVICES

We recommend that we be given the opportunity to review the foundation plan, grading plan, and project specifications when construction documents approach completion. This review evaluates whether the recommendations and comments provided herein have been understood and properly implemented. We also recommend that Froehling & Robertson, Inc. be retained for professional and construction materials testing services during construction of the project. Our continued involvement on the project helps provide continuity for proper implementation of the recommendations discussed herein.

7.0 LIMITATIONS

This report has been prepared for the exclusive use of The Hollingsworth Companies or their agent, for specific application to the Brockman Lot 15 Development project in Amherst, Virginia, in accordance with generally accepted soil and foundation engineering practices. No other warranty, express or implied, is made. Our conclusions and recommendations are based on design information furnished to us, the data obtained from the previously described subsurface exploration programs, and generally accepted geotechnical engineering practice. The conclusions and recommendations do not reflect variations in subsurface conditions which could exist intermediate of the boring locations or in unexplored areas of the site. Should such variations become apparent during construction, it will be necessary to re-evaluate our conclusions and recommendations based upon on-site observations of the conditions.

Regardless of the thoroughness of a subsurface exploration, there is the possibility that conditions between borings will differ from those at the boring locations, that conditions are not as anticipated by the designers, or that the construction process has altered the soil conditions. Therefore, experienced geotechnical engineers should evaluate earthwork, pavement, and foundation construction to verify that the conditions anticipated in design actually exist. Otherwise, we assume no responsibility for construction compliance with the design concepts, specifications, or recommendations.

In the event that changes are made in the design or location of the proposed structure, the recommendations presented in the report shall not be considered valid unless the changes are reviewed by our firm and conclusions of this report modified and/or verified in writing. If this report is copied or transmitted to a third party, it must be copied or transmitted in its entirety, including text, attachments, and enclosures. Interpretations based on only a part of this report may not be valid. This report contains 14 pages of text and the attached appendices.

APPENDIX A

Important Information About Your

Geotechnical Engineering Report

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

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

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

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

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual

subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

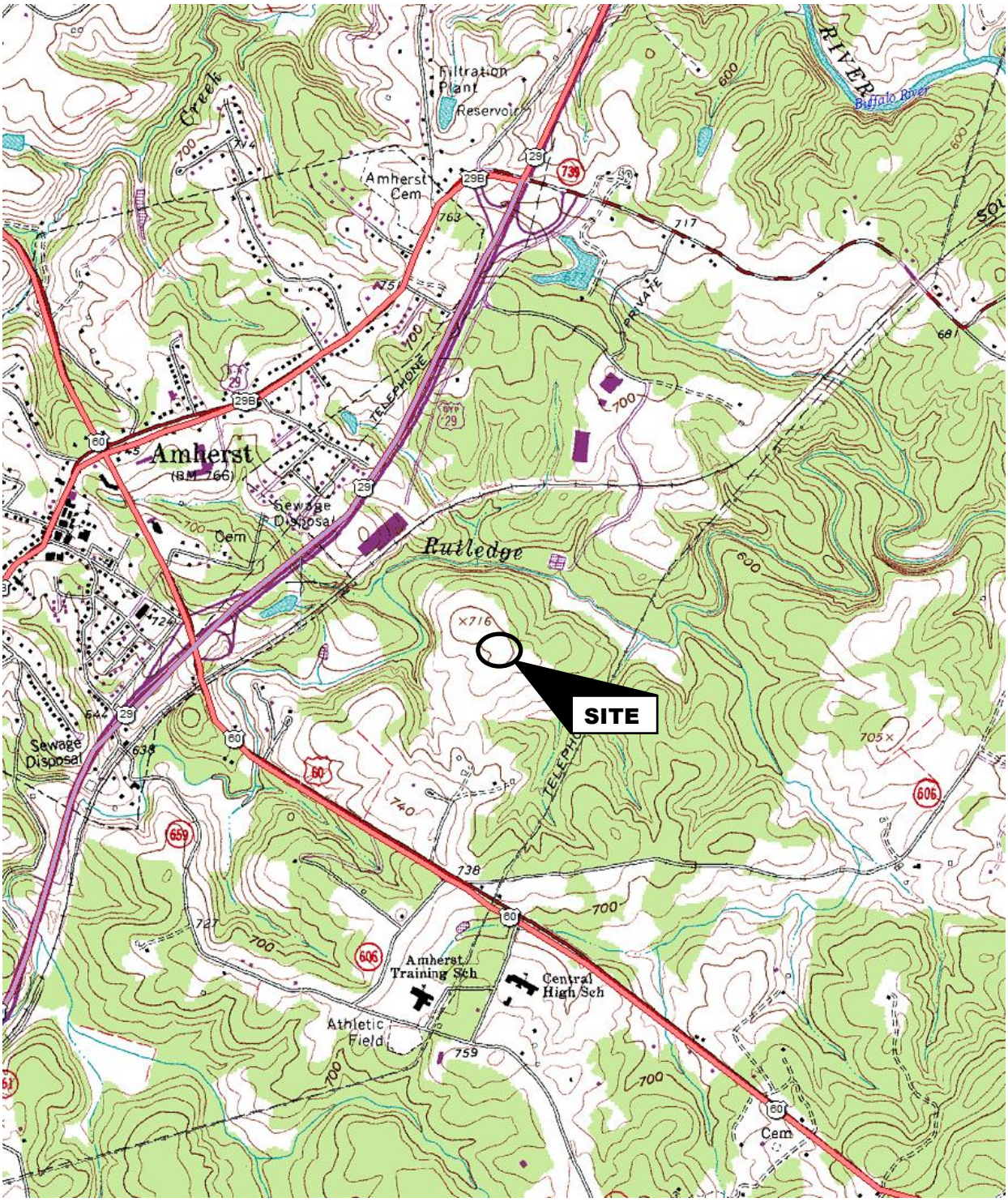
Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you ASFE-member geotechnical engineer for more information.

ASFE
The Best People on Earth

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

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

IIGER06045.0M



Adapted from the USGS 7.5 minute series topographic quadrangle: Amherst, VA (1991)

SINCE F&R 1881	**FROEHLING & ROBERTSON, INC.** *Engineering Stability Since 1881* 1734 Seibel Drive, NE Roanoke, Virginia 24012-5624 T 540.344.7939 I F 540.344.3657	**DATE:** May 2012 **SCALE:** 1 : 24,000 **DRAWN:** AKK	62P0009
The Hollingsworth Companies Brockman Lot 15 Development Amherst, Virginia	SITE VICINITY MAP	**DRAWING NO.** 1	

APPENDIX B

SINCE F&R 1881

CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES

ASTM Designation: D 2487

(Based on Unified Soil Classification System)

SOIL ENGINEERING

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests[A]				Soil Classification	
				Group Symbol	Group Name[B]
COARSE-GRAINED SOILS More than 50% retained on No. 200 sieve	Gravels More than 50% coarse fraction retaining on No. 4 sieve	Clean Gravels Less than 5% fines[C]	Cu≥ 4 and 1 ≤ Cc ≤ 3[E]	GW	Well graded gravel[F]
			Cu<4 and/or 1>Cc>3[E]	GP	Poorly graded gravel[F]
		Gravels with Fines More than 12% fines[C]	Fines classify as ML or MH	GM	Silty gravel[F,G,H]
			Fines classify as CL or CH	GC	Clayey gravel[F,G,H]
	Sands 50% or more of coarse fraction passes No. 4 sieve	Clean Sands Less than 5% fines[D]	Cu≥6 and 1 ≤ Cc ≤ 3[E]	SW	Well-graded sand[I]
			Cu<6 and/or 1>Cc>3[E]	SP	Poorly graded sand[I]
		Sands with Fines, More than 12% fines[D]	Fines classify as ML or MH	SM	Silty sand[G,H,I]
			Fines classify as CL or CH	SC	Clayey sand[G,H,I]
FINE-GRAINED SOILS 50% or more passes the No. 200 sieve	Silts and Clays Liquid Limit less than 50	Inorganic	PI>7 and plots on or above "A" line[J]	CL	Lean clay[K,L,M]
			PI<4 or plots below "A" line[J]	ML	Silt[K,L,M]
		Organic	$\frac{\text{Liquid limit-oven dried}}{\text{Liquid limit-not dried}} < 0.75$	OL	Organic clay[K,L,M,N]
					Organic silt[K,L,M,O]
	Silts and Clays Liquid limit 50 or more	Inorganic	PI plots on or above "A" line	CH	Fat clay[K,L,M]
			PI plots below "A" line	MH	Elastic silt[K,L,M]
		Organic	$\frac{\text{Liquid limit-oven dried}}{\text{Liquid limit-not dried}} < 0.75$	OH	Organic clay[K,L,M,P]
					Organic silt[K,L,M,Q]
HIGHLY ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor			PT	Peat

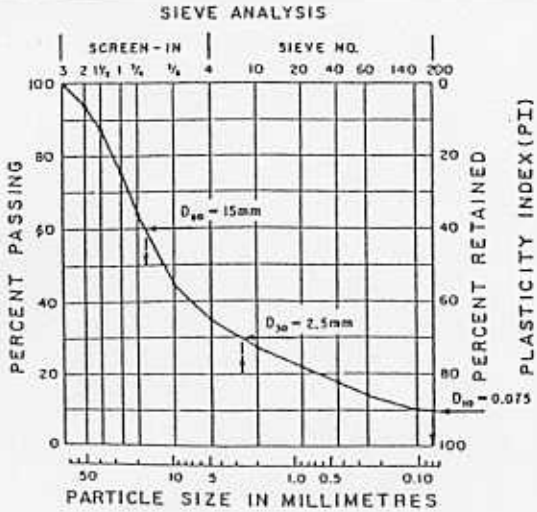
[A]Based on the material passing the 3-in. (75-mm) sieve
[B]If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.
[C]Gravels with 5 to 12% fines require dual symbols:
GW-GM well-graded gravel with silt
GW-GC well-graded gravel with clay
GP-GM poorly graded gravel with silt
GP-GC poorly graded gravel with clay
[D]Sands with 5 to 12% fines require dual symbols:
SW-SM well-graded sand with silt
SW-SC well-graded sand with clay
SP-SM poorly graded sand with silt
SP-SC poorly graded sand with clay

$E\ Cu = D_{60}/D_{10},\quad Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$

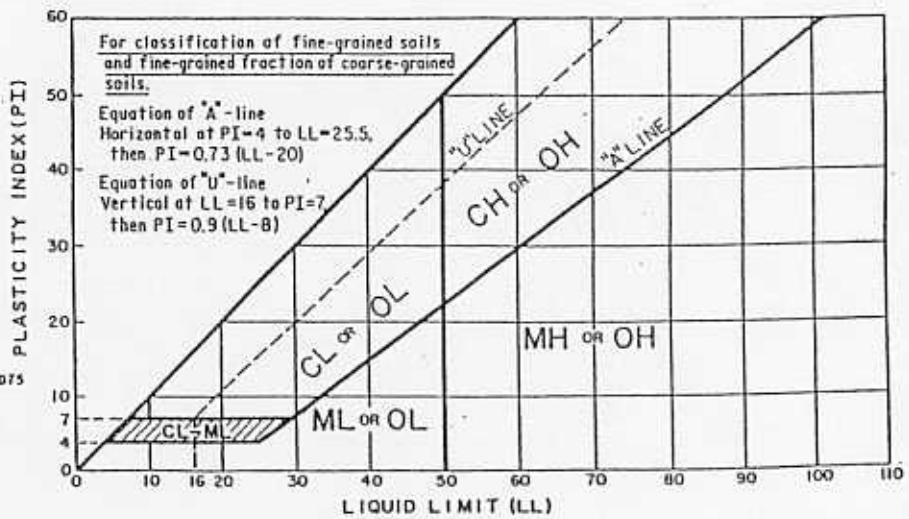
[F]If soil contains ≥ 15% sand, add "with sand" to group name.
[G]If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.
[H]If fines are organic, add "with organic fines" to group name.
[I]If soil contains ≥ 15% gravel, add "with gravel" to group name.

[J]If Atterberg limits plot in hatched area, soil is a CL-ML, silty clay.
[K]If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.
[L]If soil contains ≥ 30% plus No. 200, predominantly sand, add "sandy" to group name.
[M]If soil contains ≥ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.
[N]PI ≥ 4 and plots on or above "A" line.
[O]PI < 4 or plots below "A" line.
[P]PI plots on or above "A" line
[Q]PI plots below "A" line.

SIEVE ANALYSIS



$Cu = \frac{D_{60}}{D_{10}} = \frac{15}{0.075} = 200 \qquad Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}} = \frac{(2.5)^2}{0.075 \times 15} = 5.6$



KEY TO BORING LOG SOIL CLASSIFICATION

Particle Size and Proportion

Visual descriptions are assigned to each soil sample or stratum based on estimates of the particle size of each component of the soil and the percentage of each component of the soil.

Particle Size Descriptive Terms		Proportion Descriptive Terms		
Soil Component	Particle Size	Component	Term	Percentage
Boulder	> 12 inch	Major	Uppercase Letters (e.g., SAND, CLAY)	> 50%
Cobble	3 - 12 inch			
Gravel-Coarse	3/4 - 3 inch			
-Fine	#4 - 3/4 inch	Secondary	Adjective (e.g., sandy, clayey)	25% - 50%
Sand-Coarse	#10 - #4			
-Medium	#40 - #10			
-Fine	#200 - #40	Minor	Some	15% - 25%
Silt (non-cohesive)	< #200		Little	5% - 15%
Clay (cohesive)	< #200		Trace	0% - 5%

Notes:
1. Particle size is designated by U.S. Standard Sieve Sizes
2. Because of the small size of the split-spoon sampler relative to the size of gravel, the true percentage of gravel may not be accurately estimated.

Density or Consistency

The standard penetration resistance values (N-values) are used to describe the density of coarse-grained soils (GRAVEL, SAND) or the consistency of fine-grained soils (SILT, CLAY). Sandy silts of very low plasticity may be assigned a density instead of a consistency.

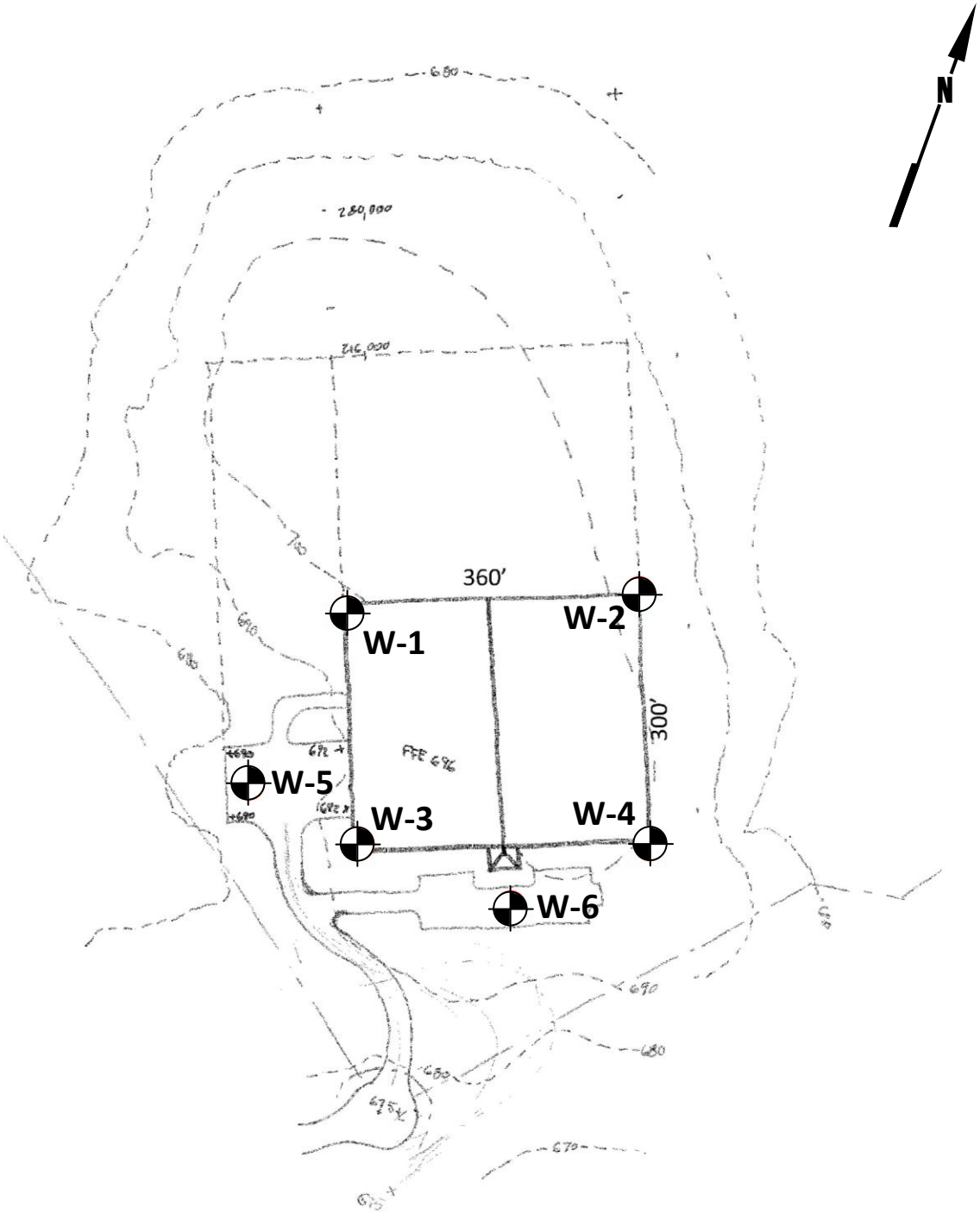
DENSITY		CONSISTENCY	
Term	N-Value	Term	N-Value
Very Loose	0 - 4	Very Soft	0 - 1
Loose	5 - 10	Soft	2 - 4
Medium Dense	11 - 30	Medium Stiff	5 - 8
Dense	31 - 50	Stiff	9 - 15
Very Dense	> 50	Very Stiff	16 - 30
		Hard	> 30

Notes:
1. The N-value is the number of blows of a 140 lb. Hammer freely falling 30 inches required to drive a standard split-spoon sampler (2.0 in. O.D., 1-3/8 in. I.D.) 12 inches into the soil after properly seating the sampler 6 inches.
2. When encountered, gravel may increase the N-value of the standard penetration test and may not accurately represent the in-situ density or consistency of the soil sampled.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		(LITTLE OR NO FINES)		GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)		GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		(LITTLE OR NO FINES)		SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES		SM	SILTY SANDS, SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)		SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	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 SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
EXISTING FILL				FILL	EXISTING FILL MATERIALS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

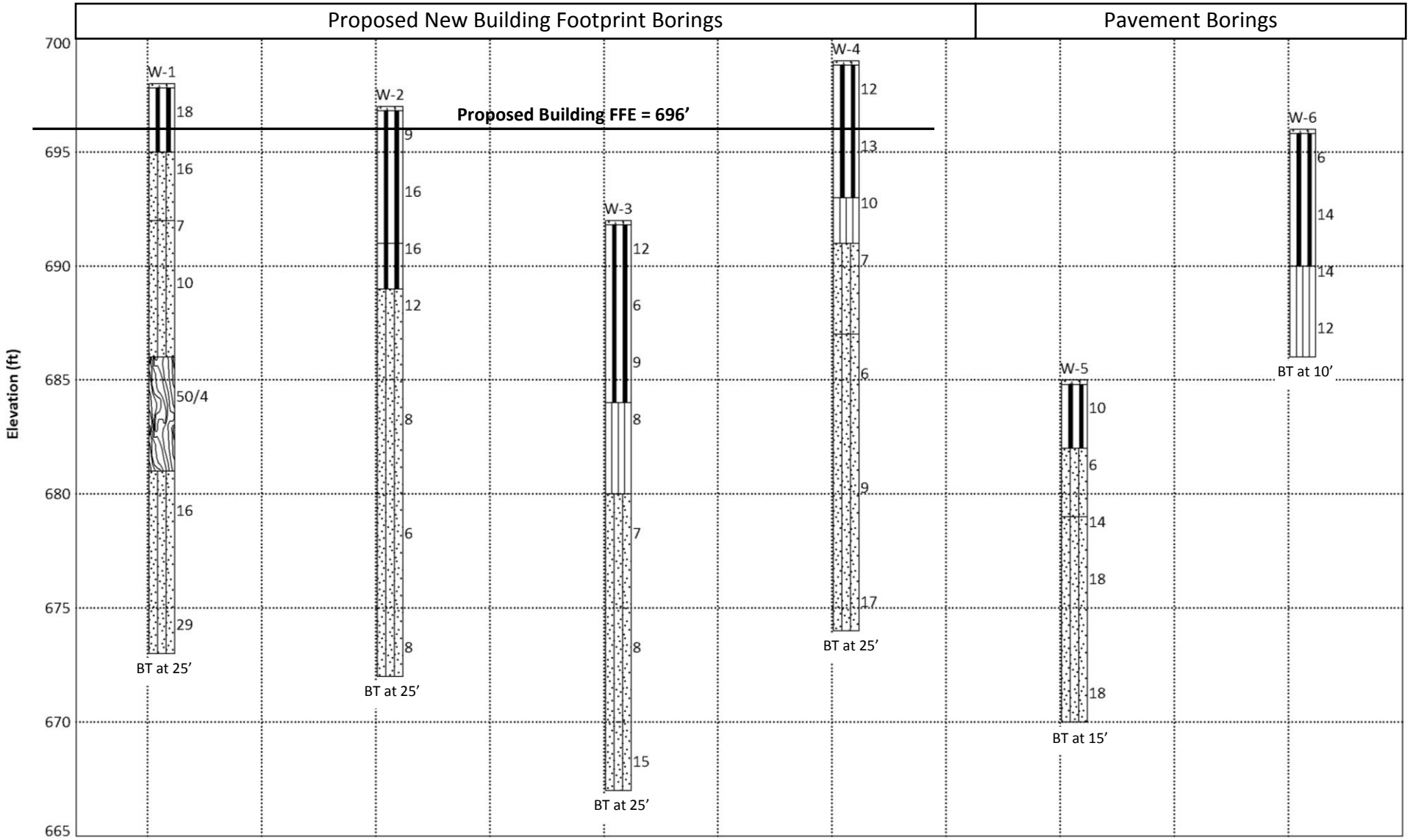


Adapted from the drawing (filename: *Boring Location Sketch.pdf)*
provided by The Hollingsworth Companies on 3/28/12.

FROEHLING & ROBERTSON, INC.
Engineering Stability Since 1881
1734 Seibel Drive, NE
Roanoke, Virginia 24012-5624
T 540.344.7939 | F 540.344.3657

DATE:	May 2012	
SCALE:	1" = 200' Assumed	
DRAWN:	AKK	62P-0009

The Hollingsworth Companies Brockman Lot 15 Development Amherst, Virginia	BORING LOCATION PLAN	**DRAWING NO.** 2



Legend

BT = Boring Terminated

FROEHLING & ROBERTSON, INC.
Engineering Stability Since 1881
1734 Seibel Drive, NE
Roanoke, Virginia 24012-5624 I USA

DATE:	May 2012	
SCALE:	Not to Scale	
DRAWN:	BWS	62P0009

The Hollingsworth Companies
Brockman Lot 15 Development
Amherst, Virginia

COMPOSITE SUBSURFACE PROFILE

DRAWING NO.
3

FROEHLING & ROBERTSON, INC.

BORING LOG

Boring: W-1 (1 of 1)

Project No: 62P0009
Client: The Hollingsworth Companies
Project: Brockman Lot 15 Development
City/State: Amherst, Virginia

Elevation: 698
Total Depth: 25.0'
Boring Location: See Boring Location Plan

Drilling Method: 3.25" ID HSA
Hammer Type: Automatic
Date Drilled: 4/13/12
Driller: W. Wilson

Elevation	Depth	Description of Materials (Classification)	* Sample Blows	Sample Depth (feet)	N-Value (blows/ft)	Remarks
697.8	0.2	Surficial Soils				Subsurface water was not encountered immediately upon completion of drilling.
		RESIDUUM: Very stiff, red brown, moist, SILT (MH) with little fine sand	9-10-8	1.0	18	
				2.5		
695.0	3.0	Medium dense, dark brown, brown, and red brown, moist, silty fine to coarse SAND (SM)	3-8-8	3.5	16	
				5.0		
692.0	6.0	Loose, red brown and brown, moist, silty fine to coarse SAND (SM)	3-3-4	6.0	7	
				7.5		
			3-3-7	8.5	10	
				10.0		
686.0	12.0	**PARTIALLY WEATHERED ROCK:** Sampled as very dense, brown and dark gray, moist, fine to coarse SAND (SM) with some silt and some fine gravel	32-50/4	13.5	100+	
				15.0		
681.0	17.0	**RESIDUUM:** Medium dense, brown, tan, orange brown, and black, moist, silty fine to coarse SAND (SM)	24-8-8	18.5	16	
				20.0		
		-with some fine gravel from 22 to 25 feet	18-11-18	23.5	29	
673.0	25.0	Boring terminated at 25 feet		25.0		

*Number of blows required for a 140 lb hammer dropping 30" to drive 2" O.D., 1.375" I.D. sampler a total of 18 inches in three 6" increments. The sum of the second and third increments of penetration is termed the standard penetration resistance, N-Value.

FROEHLING & ROBERTSON, INC.

BORING LOG

Boring: W-2 (1 of 1)

Project No: 62P0009
Client: The Hollingsworth Companies
Project: Brockman Lot 15 Development
City/State: Amherst, Virginia

Elevation: 697
Total Depth: 25.0'
Boring Location: See Boring Location Plan

Drilling Method: 3.25" ID HSA
Hammer Type: Automatic
Date Drilled: 4/13/12
Driller: W. Wilson

Elevation	Depth	Description of Materials (Classification)	* Sample Blows	Sample Depth (feet)	N-Value (blows/ft)	Remarks
696.8	0.2	Surficial Soils				Subsurface water was not encountered immediately upon completion of drilling.
		RESIDUUM: Stiff to very stiff, red brown, moist, SILT (MH)	3-4-5	1.0	9	
				2.5		
			2-7-9	3.5	16	
				5.0		
691.0	6.0	Very stiff, red brown, moist, SILT (MH) with little fine sand	5-8-8	6.0	16	
				7.5		
689.0	8.0	Medium dense to loose, red brown, moist, silty fine to coarse SAND (SM)	4-6-6	8.5	12	
				10.0		
			2-3-5	13.5	8	
				15.0		
		- brown and tan from 17 to 25 feet	3-3-3	18.5	6	
				20.0		
			3-4-4	23.5	8	
672.0	25.0	Boring terminated at 25 feet		25.0		

*Number of blows required for a 140 lb hammer dropping 30" to drive 2" O.D., 1.375" I.D. sampler a total of 18 inches in three 6" increments. The sum of the second and third increments of penetration is termed the standard penetration resistance, N-Value.

FROEHLING & ROBERTSON, INC.

BORING LOG

Boring: W-3 (1 of 1)

Project No: 62P0009
Client: The Hollingsworth Companies
Project: Brockman Lot 15 Development
City/State: Amherst, Virginia

Elevation: 692
Total Depth: 25.0'
Boring Location: See Boring Location Plan

Drilling Method: 3.25" ID HSA
Hammer Type: Automatic
Date Drilled: 4/13/12
Driller: W. Wilson

Elevation	Depth	Description of Materials (Classification)	* Sample Blows	Sample Depth (feet)	N-Value (blows/ft)	Remarks
691.8	0.2	Surficial Soils				Subsurface water was not encountered immediately upon completion of drilling.
		RESIDUUM: Stiff to medium stiff, red brown, moist, SILT (MH) with trace to some fine sand	18-5-7	1.0	12	
				2.5		
			2-2-4	3.5	6	
				5.0		
			3-3-6	6.0	9	
				7.5		
684.0	8.0	Medium stiff, red brown, moist, SILT (ML) with little fine sand and trace mica	2-3-5	8.5	8	
				10.0		
680.0	12.0	Loose to medium dense, orange brown, moist, silty fine SAND (SM) with trace mica				
			2-3-4	13.5	7	
				15.0		
			2-4-4	18.5	8	
				20.0		
			4-7-8	23.5	15	
667.0	25.0	Boring terminated at 25 feet		25.0		

*Number of blows required for a 140 lb hammer dropping 30" to drive 2" O.D., 1.375" I.D. sampler a total of 18 inches in three 6" increments. The sum of the second and third increments of penetration is termed the standard penetration resistance, N-Value.

FROEHLING & ROBERTSON, INC.

BORING LOG

Boring: W-4 (1 of 1)

Project No: 62P0009
Client: The Hollingsworth Companies
Project: Brockman Lot 15 Development
City/State: Amherst, Virginia

Elevation: 699
Total Depth: 25.0'
Boring Location: See Boring Location Plan

Drilling Method: 3.25" ID HSA
Hammer Type: Automatic
Date Drilled: 4/13/12
Driller: W. Wilson

Elevation	Depth	Description of Materials (Classification)	* Sample Blows	Sample Depth (feet)	N-Value (blows/ft)	Remarks
698.8	0.2	Surficial Soils				Subsurface water was not encountered immediately upon completion of drilling.
		RESIDUUM: Stiff, red brown, moist, SILT (MH)	6-4-8	1.0	12	
				2.5		
696.0	3.0	Stiff, red brown, moist, SILT (MH) with some fine sand	3-5-8	3.5	13	
				5.0		
693.0	6.0	Stiff, red brown, moist, fine sandy SILT (ML) with trace mica	3-4-6	6.0	10	
				7.5		
691.0	8.0	Loose, tan and red brown, moist, silty fine SAND (SM)	2-3-4	8.5	7	
				10.0		
687.0	12.0	Loose to medium dense, tan and brown, moist, silty fine SAND (SM)	3-2-4	13.5	6	
				15.0		
			3-4-5	18.5	9	
				20.0		
			6-7-10	23.5	17	
674.0	25.0	Boring terminated at 25 feet		25.0		

*Number of blows required for a 140 lb hammer dropping 30" to drive 2" O.D., 1.375" I.D. sampler a total of 18 inches in three 6" increments. The sum of the second and third increments of penetration is termed the standard penetration resistance, N-Value.

FROEHLING & ROBERTSON, INC.

BORING LOG

Boring: W-5 (1 of 1)

Project No: 62P0009
Client: The Hollingsworth Companies
Project: Brockman Lot 15 Development
City/State: Amherst, Virginia

Elevation: 685
Total Depth: 15.0'
Boring Location: See Boring Location Plan

Drilling Method: 3.25" ID HSA
Hammer Type: Automatic
Date Drilled: 4/13/12
Driller: W. Wilson

Elevation	Depth	Description of Materials (Classification)	* Sample Blows	Sample Depth (feet)	N-Value (blows/ft)	Remarks
684.8	0.2	Surficial Soils				Subsurface water was not encountered immediately upon completion of drilling.
		RESIDUUM: Stiff, red brown, moist, fine sandy SILT (MH)	7-5-5	1.0	10	
				2.5		
682.0	3.0	Loose, orange brown, moist, silty fine to coarse SAND (SM)	2-3-3	3.5	6	
				5.0		
679.0	6.0	Medium dense, brown, tan, and light gray, moist, silty fine to coarse SAND (SM) with some fine gravel	2-3-11	6.0	14	
				7.5		
			4-8-10	8.5	18	
				10.0		
			14-10-8	13.5	18	
670.0	15.0	Boring terminated at 15 feet		15.0		

*Number of blows required for a 140 lb hammer dropping 30" to drive 2" O.D., 1.375" I.D. sampler a total of 18 inches in three 6" increments. The sum of the second and third increments of penetration is termed the standard penetration resistance, N-Value.

FROEHLING & ROBERTSON, INC.

BORING LOG

Boring: W-6 (1 of 1)

Project No: 62P0009
Client: The Hollingsworth Companies
Project: Brockman Lot 15 Development
City/State: Amherst, Virginia

Elevation: 696
Total Depth: 10.0'
Boring Location: See Boring Location Plan

Drilling Method: 3.25" ID HSA
Hammer Type: Automatic
Date Drilled: 4/13/12
Driller: W. Wilson

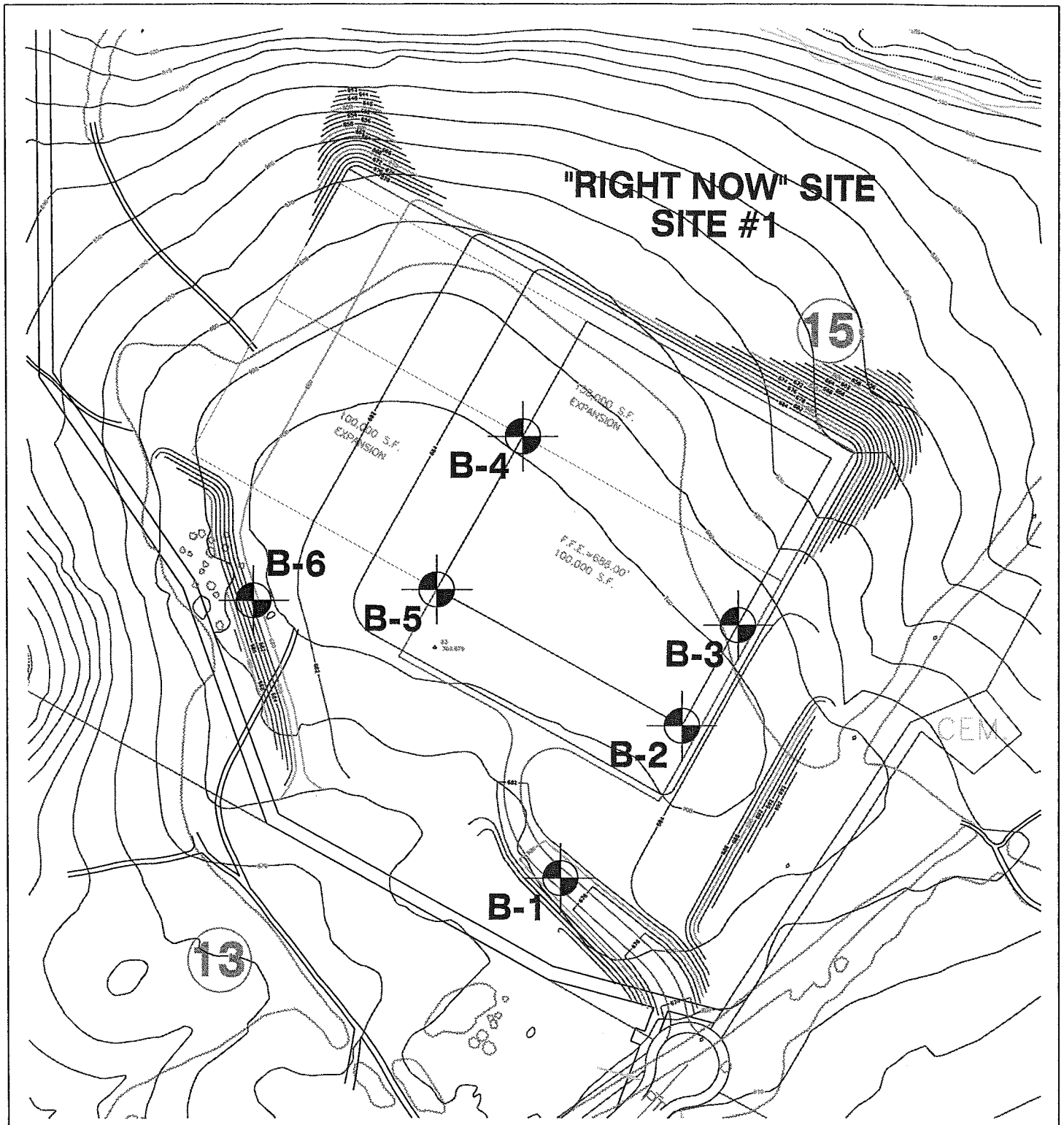
Elevation	Depth	Description of Materials (Classification)	* Sample Blows	Sample Depth (feet)	N-Value (blows/ft)	Remarks
695.8	0.2	Surficial Soils				Subsurface water was not encountered immediately upon completion of drilling.
		RESIDUUM: Medium stiff to stiff, red brown, moist, SILT (MH)	2-2-4	1.0	6	
				2.5		
			4-6-8	3.5	14	
				5.0		
690.0	6.0	Stiff, red brown, moist, fine sandy SILT (ML) with trace mica	3-6-8	6.0	14	
				7.5		
			3-5-7	8.5	12	
686.0	10.0	Boring terminated at 10 feet		10.0		

*Number of blows required for a 140 lb hammer dropping 30" to drive 2" O.D., 1.375" I.D. sampler a total of 18 inches in three 6" increments. The sum of the second and third increments of penetration is termed the standard penetration resistance, N-Value.

BORING_LOG 62P-0009 (WITH OLD DATA).GPJ F&R.GDT 5/1/12

APPENDIX C

PREVIOUS EXPLORATION BORING DATA



Note: Adapted from an overall map drawing provided by Dewberry & Davis, Inc. on 25 August 2003.

SINCE F&R 1881	**FROEHLING & ROBERTSON, INC.** GEOTECHNICAL • ENVIRONMENTAL • MATERIALS ENGINEERS • LABORATORIES *"OVER ONE HUNDRED YEARS OF SERVICE"*	**DATE:**	October 2003
		SCALE:	1" = 200'
		DRAWN: JTM	E62-203G

Dewberry & Davis, Inc. Brockman Business Park - Right Now Site (Site #1) Amherst, Virginia	BORING LOCATION PLAN	**DRAWING NO.** 2

BORING LOG

SINCE 1881

FROEHLING & ROBERTSON, INC.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS
ENGINEERS • LABORATORIES
"OVER ONE HUNDRED YEARS OF SERVICE"

Report No.: **E62-203G** Date: **October 2003**

Client: **Dewberry & Davis, Inc.**

Project: **Brockman Business Park - Right Now Site (Site #1), Amherst, Virginia**

Boring No.: **B-1** **(1 of 1)** Total Depth **20.0'** Elev: **692.0 ±** Location: **See boring location plan**

Type of Boring: **2.25" ID HSA CME 55** Started: **9/10/03** Completed: **9/10/03** Driller: **B. Maxson**

Elevation	Depth	DESCRIPTION OF MATERIALS (Classification)	* Sample Blows	Sample Depth (feet)	N Value (blows/ft)	REMARKS
691.5	0.5	SURFICIAL SOIL				
		RESIDUUM: Stiff, red brown and brown, moist, fine sandy SILT (ML) with trace mica	6-6-7	1.0	13	Subsurface water was not encountered immediately upon completion of drilling.
				2.5		
689.0	3.0	Loose, red brown, moist, silty fine SAND (SM) with trace mica	3-4-4	3.5	8	
				5.0		
			3-4-5	6.0	9	
				7.5		
			3-4-5	8.5	9	
682.0	10.0	Medium dense, brown, moist, silty fine SAND (SM) with trace mica		10.0		
			4-5-7	13.5	12	
				15.0		
675.0	17.0	Stiff, brown, moist, fine sandy SILT (ML) with trace mica				
			2-4-6	18.5	10	
672.0	20.0	Boring terminated at 20 feet		20.0		

*Number of blows required for a 140 lb hammer dropping 30" to drive 2" O.D., 1.375" I.D. sampler a total of 18 inches in three 6" increments. The sum of the second and third increments of penetration is termed the standard penetration resistance, N.

BORING LOG

SINCE 1881

FROEHLING & ROBERTSON, INC.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS
ENGINEERS • LABORATORIES
"OVER ONE HUNDRED YEARS OF SERVICE"

Report No.: **E62-203G** Date: **October 2003**

Client: **Dewberry & Davis, Inc.**

Project: **Brockman Business Park - Right Now Site (Site #1), Amherst, Virginia**

Boring No.: **B-2** **(1 of 1)** Total Depth **20.0'** Elev: **705.0 ±** Location: **See boring location plan**

Type of Boring: **2.25" ID HSA CME 55** Started: **9/10/03** Completed: **9/10/03** Driller: **B. Maxson**

Elevation	Depth	DESCRIPTION OF MATERIALS (Classification)	* Sample Blows	Sample Depth (feet)	N Value (blows/ft)	REMARKS
704.6	0.4	SURFICIAL SOIL				
		RESIDUUM: Stiff, red brown, moist, CLAY (CL/CH)	4-6-9	1.0	15	Subsurface water was not encountered immediately upon completion of drilling.
				2.5		
702.0	3.0	Stiff, red brown, moist, SILT (ML/MH) with little fine sand and trace mica	5-6-7	3.5	13	
				5.0		
699.5	5.5	Medium stiff, red brown and brown, moist, fine sandy SILT (ML) with trace mica	3-4-4	6.0	8	
				7.5		
697.0	8.0	Loose to medium dense, brown, moist, silty fine SAND (SM) with trace mica	2-3-4	8.5	7	
				10.0		
			3-3-5	13.5	8	
				15.0		
			4-5-6	18.5	11	
685.0	20.0	Boring terminated at 20 feet		20.0		

*Number of blows required for a 140 lb hammer dropping 30" to drive 2" O.D., 1.375" I.D. sampler a total of 18 inches in three 6" increments. The sum of the second and third increments of penetration is termed the standard penetration resistance, N.

BORING LOG

SINCE 1881

FROEHLING & ROBERTSON, INC.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS
ENGINEERS • LABORATORIES
"OVER ONE HUNDRED YEARS OF SERVICE"

Report No.: **E62-203G** Date: **October 2003**

Client: **Dewberry & Davis, Inc.**

Project: **Brockman Business Park - Right Now Site (Site #1), Amherst, Virginia**

Boring No.: **B-3** **(1 of 1)** Total Depth **20.0'** Elev: **696.0 ±** Location: **See boring location plan**

Type of Boring: **2.25" ID HSA CME 55** Started: **9/10/03** Completed: **9/10/03** Driller: **B. Maxson**

Elevation	Depth	DESCRIPTION OF MATERIALS (Classification)	* Sample Blows	Sample Depth (feet)	N Value (blows/ft)	REMARKS
695.6	0.4	SURFICIAL SOIL				
		RESIDUUM: Stiff, red brown, moist, CLAY (CL/CH)	3-4-5	1.0	9	Subsurface water was not encountered immediately upon completion of drilling.
				2.5		
693.0	3.0	Very stiff, red brown, moist, SILT (ML/MH) with little fine sand and trace mica	6-8-10	3.5	18	
				5.0		
690.5	5.5	Very stiff, red brown, moist, fine sandy SILT (ML/MH) with trace mica	5-6-9	6.0	15	
				7.5		
688.0	8.0	Loose to medium dense, orange brown, moist, silty fine SAND (SM) with trace mica	3-4-4	8.5	8	
				10.0		
		- brown and tan from 12 to 20 feet				
			4-4-4	13.5	8	
				15.0		
			5-6-8	18.5	14	
676.0	20.0	Boring terminated at 20 feet		20.0		

*Number of blows required for a 140 lb hammer dropping 30" to drive 2" O.D., 1.375" I.D. sampler a total of 18 inches in three 6" increments. The sum of the second and third increments of penetration is termed the standard penetration resistance, N.

BORING LOG

SINCE 1881

FROEHLING & ROBERTSON, INC.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS
ENGINEERS • LABORATORIES
"OVER ONE HUNDRED YEARS OF SERVICE"

Report No.: **E62-203G** Date: **October 2003**

Client: **Dewberry & Davis, Inc.**

Project: **Brockman Business Park - Right Now Site (Site #1), Amherst, Virginia**

Boring No.: **B-4** **(1 of 1)** Total Depth **20.0'** Elev: **698.0 ±** Location: **See boring location plan**

Type of Boring: **2.25" ID HSA CME 55** Started: **9/10/03** Completed: **9/10/03** Driller: **B. Maxson**

Elevation	Depth	DESCRIPTION OF MATERIALS (Classification)	* Sample Blows	Sample Depth (feet)	N Value (blows/ft)	REMARKS
697.5	0.5	SURFICIAL SOIL				
		RESIDUUM: Stiff and very stiff, red brown, moist, CLAY (CL/CH) with trace fine sand	2-5-6	1.0	11	Subsurface water was not encountered immediately upon completion of drilling.
				2.5		
			5-10-12	3.5	22	
				5.0		
			5-6-8	6.0	14	
				7.5		
690.0	8.0	Very stiff, orange brown, moist, SILT (ML/MH) with some fine sand and trace mica	6-8-9	8.5	17	
				10.0		
686.0	12.0	Medium dense, brown, moist, silty fine SAND (SM) with trace mica				
			6-8-8	13.5	16	
				15.0		
			6-7-8	18.5	15	
678.0	20.0	Boring terminated at 20 feet		20.0		

*Number of blows required for a 140 lb hammer dropping 30" to drive 2" O.D., 1.375" I.D. sampler a total of 18 inches in three 6" increments. The sum of the second and third increments of penetration is termed the standard penetration resistance, N.

BORING LOG

SINCE 1881

FROEHLING & ROBERTSON, INC.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS
ENGINEERS • LABORATORIES
"OVER ONE HUNDRED YEARS OF SERVICE"

Report No.: **E62-203G** Date: **October 2003**

Client: **Dewberry & Davis, Inc.**

Project: **Brockman Business Park - Right Now Site (Site #1), Amherst, Virginia**

Boring No.: **B-5** **(1 of 1)** Total Depth **20.0'** Elev: **705.0 ±** Location: **See boring location plan**

Type of Boring: **2.25" ID HSA CME 55** Started: **9/10/03** Completed: **9/10/03** Driller: **B. Maxson**

Elevation	Depth	DESCRIPTION OF MATERIALS (Classification)	* Sample Blows	Sample Depth (feet)	N Value (blows/ft)	REMARKS
704.6	0.4	SURFICIAL SOIL				
		RESIDUUM: Very stiff, red brown, moist, CLAY (CL/CH) with little fine sand and trace mica	4-9-11	1.0	20	Subsurface water was not encountered immediately upon completion of drilling.
				2.5		
			4-7-10	3.5	17	
				5.0		
699.5	5.5	Stiff, red brown, moist, fine sandy SILT (ML/MH) with trace mica	5-6-7	6.0	13	
				7.5		
			4-6-6	8.5	12	
				10.0		
693.0	12.0	Medium dense, brown, moist, silty fine SAND (SM) with trace mica				
			3-7-5	13.5	12	
				15.0		
688.0	17.0	Very stiff, red brown, moist, SILT (ML) with some fine sand and trace mica				
			4-7-11	18.5	18	
685.0	20.0	Boring terminated at 20 feet		20.0		

*Number of blows required for a 140 lb hammer dropping 30" to drive 2" O.D., 1.375" I.D. sampler a total of 18 inches in three 6" increments. The sum of the second and third increments of penetration is termed the standard penetration resistance, N.

BORING LOG

SINCE 1881

FROEHLING & ROBERTSON, INC.
GEOTECHNICAL • ENVIRONMENTAL • MATERIALS
ENGINEERS • LABORATORIES
"OVER ONE HUNDRED YEARS OF SERVICE"

Report No.: **E62-203G** Date: **October 2003**

Client: **Dewberry & Davis, Inc.**

Project: **Brockman Business Park - Right Now Site (Site #1), Amherst, Virginia**

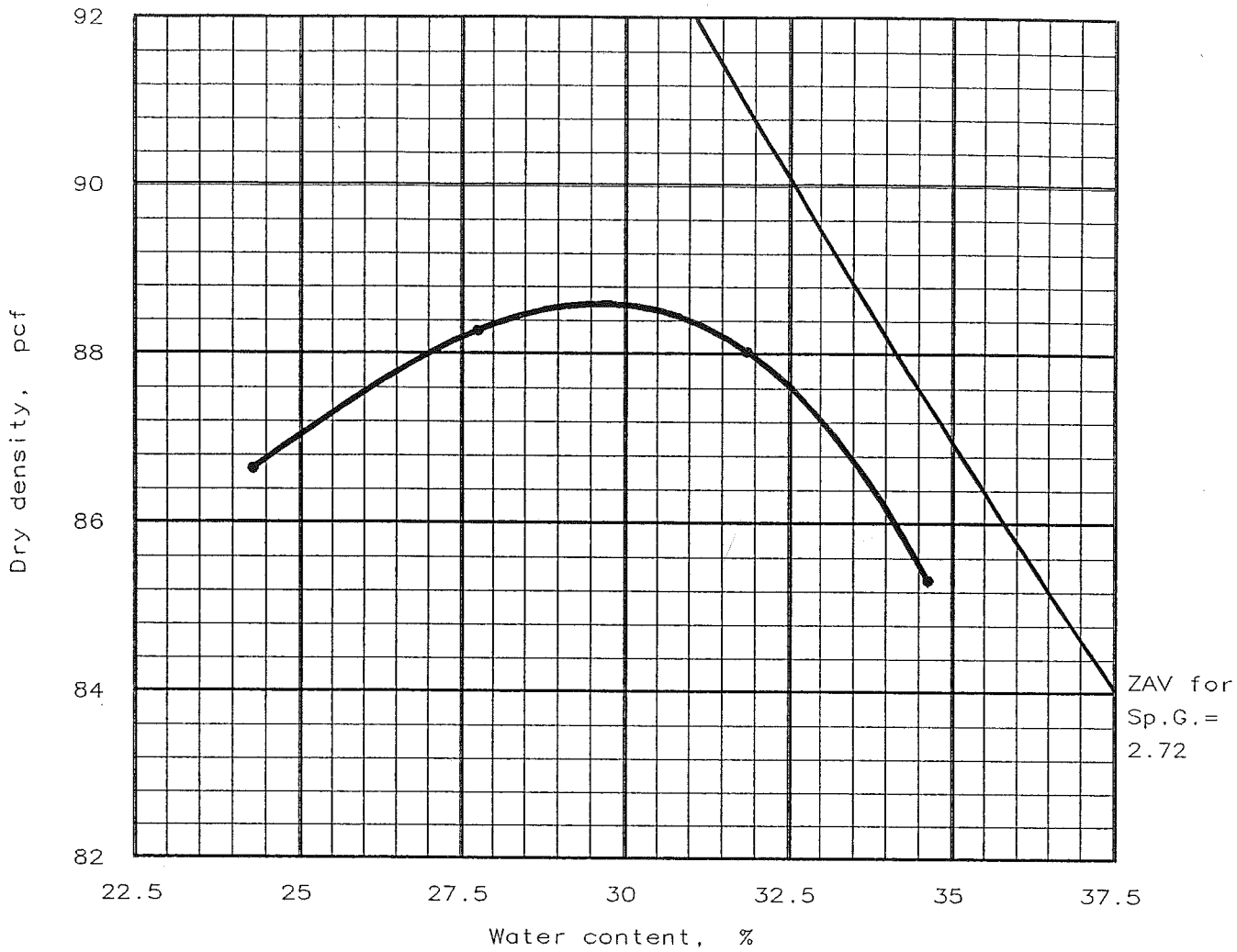
Boring No.: **B-6** **(1 of 1)** Total Depth **15.5'** Elev: **695.0 ±** Location: **See boring location plan**

Type of Boring: **2.25" ID HSA CME 55** Started: **9/10/03** Completed: **9/10/03** Driller: **B. Maxson**

Elevation	Depth	DESCRIPTION OF MATERIALS (Classification)	* Sample Blows	Sample Depth (feet)	N Value (blows/ft)	REMARKS
694.6	0.4	SURFICIAL SOIL				
		RESIDUUM: Stiff, red brown, moist, CLAY (CL) with little fine sand	2-4-5	1.0	9	Subsurface water was not encountered immediately upon completion of drilling.
				2.5		
692.0	3.0	Dense to very dense, dark gray brown, moist, silty fine to medium SAND (SM)	22-23-26	3.5	49	
				5.0		
			14-26-34	6.0	60	
				7.5		
687.0	8.0	**PARTIALLY WEATHERED ROCK:** Sampled as very dense, dark gray brown, moist, silty fine to coarse SAND (SM) with little fine angular gravel	9-50/0	8.5		
			34-50/3	13.5		
				14.3		
679.5	15.5	Auger refusal at 15.5 feet				

*Number of blows required for a 140 lb hammer dropping 30" to drive 2" O.D., 1.375" I.D. sampler a total of 18 inches in three 6" increments. The sum of the second and third increments of penetration is termed the standard penetration resistance, N.

MOISTURE-DENSITY RELATIONSHIP TEST



Test specification: ASTM D 698-91 Method A, Standard

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	USCS	AASHTO						
0'-10'	MH	--	30.8 %	--	66	32	0 %	82.0 %

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 88.6 pcf Optimum moisture = 29.7 %	Orange brown elastic SILT with sand

Project No.: E62-203G

Project: Dewberry & Davis, Inc.

Location: Brockman Business Park-Right Now Site
Amherst, Virginia

Date: 09-29-03

Remarks:

Lab No. 87888

Boring B-4

Right Now Site (Site #1)

MOISTURE-DENSITY RELATIONSHIP TEST

FROEHLING & ROBERTSON, INC.

SINCE 1881

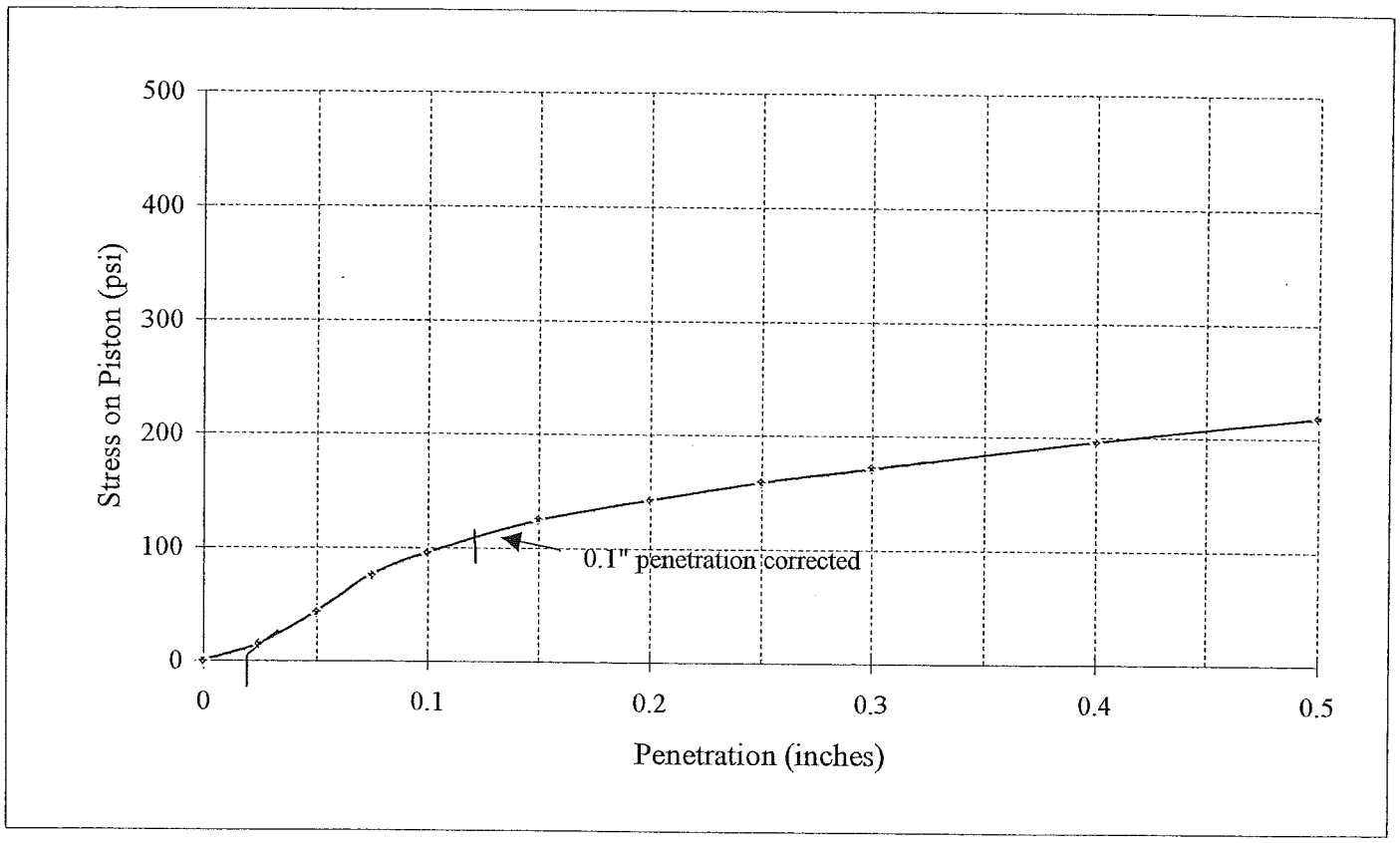
FROEHLING & ROBERTSON, INC.
GEOTECHNICAL - ENVIRONMENTAL - MATERIALS
ENGINEERS - LABORATORIES
"Over One Hundred Years of Service"
1734 Seibel Drive, N.E., Roanoke, Virginia 24012
Telephone: (540) 344-7939 Fax: (540) 344-3657

California Bearing Ratio (ASTM D 1883)

Record No.: E62-203G

Client: Dewberry & Davis, Inc.
Project: Brockman Business Park - Right Now Site
(Site #1)
Amherst, Virginia

Test Date: 07-Oct-03
Tested By: M R Henry
Compaction method: ASTM D 698
X Soaked CBR
____ Unsoaked CBR



CBR: penetration @ 0.1 in. (corrected)	10.5
Swell (%):	0.5
Dry Density Before Soaking (pcf):	93.7
Dry Density After Soaking (pcf):	93.9
Retained on 3/4 inch sieve (%):	0.0
Surcharge Weight (pounds):	10.0
Moisture Content Before Soaking (%):	29.5
Moisture Content After Soak, Top in. (%):	33.4
Moisture Content After Soak, Avg. (%):	30.1

Maximum Dry Density (pcf):	88.6
Optimum Moisture Content (%):	29.7

Visual Description:
Orange brown elastic SILT with sand

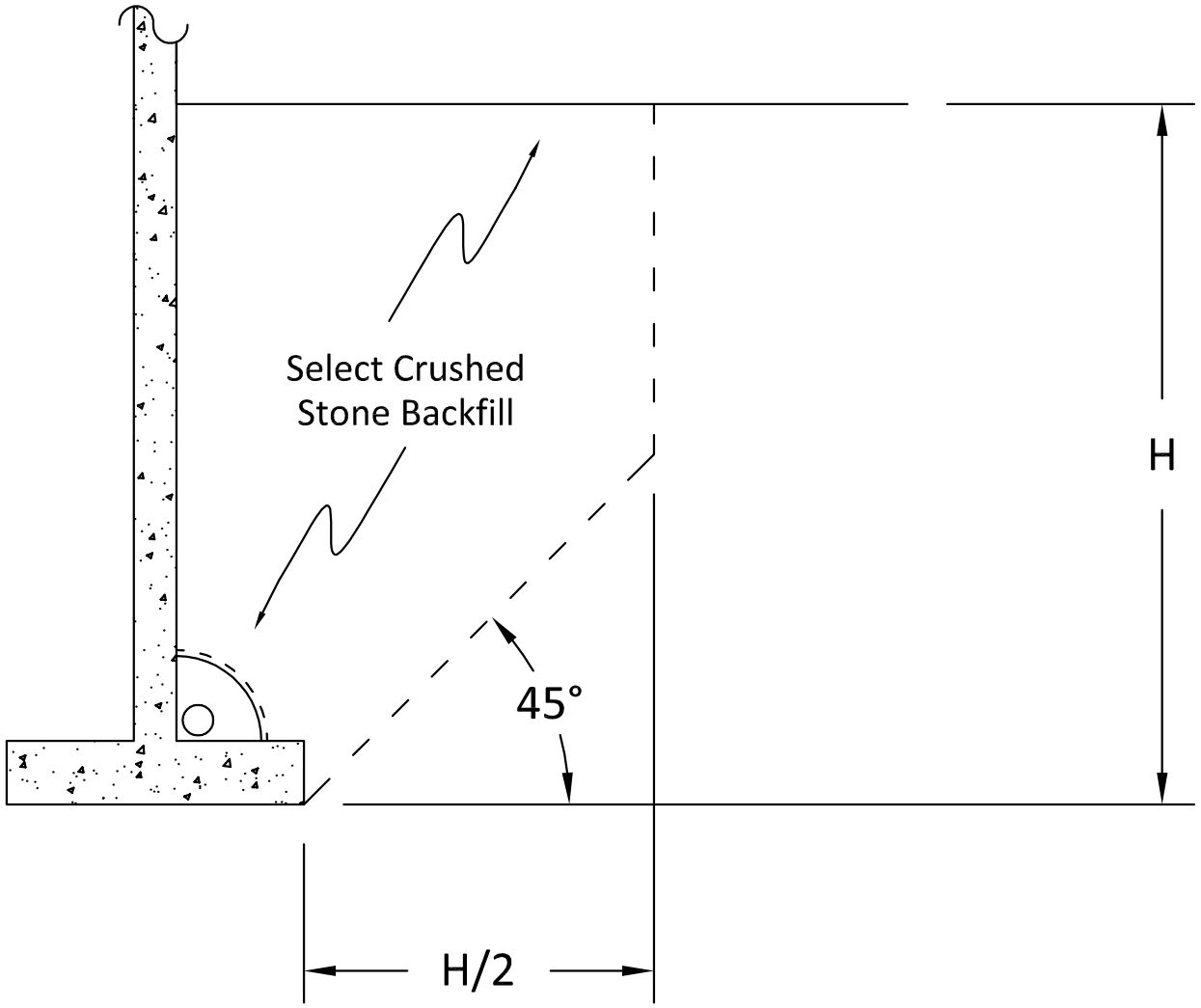
F&R Lab No.: 87888

Source: Boring B-4, 0'-10'
Right Now Site (Site #1)

FROEHLING & ROBERTSON, INC.

By: _____

APPENDIX D



FROEHLING & ROBERTSON, INC. GEOTECHNICAL • ENVIRONMENTAL • MATERIALS ENGINEERS • LABORATORIES *"OVER ONE HUNDRED YEARS OF SERVICE"*	**DATE:**	May 2012	
	SCALE:	Not to Scale	
	DRAWN:	BWS	62P0009

The Hollingsworth Companies Brockman Lot 15 Development Amherst, Virginia	EXTENT OF SELECT BACKFILL FOR RETAINING WALLS	**DRAWING NO.** 4

SINCE

1881