Report of Preliminary Subsurface Exploration
and Geotechnical Engineering Evaluation
Brockman Business Park – Right Now Site (Site #1)
Amherst, Virginia
F&R Project No. E62-203G

Prepared For:
Dewberry & Davis, Inc.
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By:
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October 2003

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F&R Project No.: E62-203G

15 October 2003

Dewberry & Davis, Inc.
551 Piney Forest Road
Danville, Virginia 24540

Attention: Mr. Tim Reynolds

Subject: Brockman Business Park – Right Now Site (Site #1)
Amherst, Virginia

Dear Mr. Reynolds:

The purpose of this report is to present the results of the preliminary subsurface exploration program and geotechnical engineering analyses undertaken by Froehling & Robertson, Inc. (F&R) in connection with the above referenced project. Our services were performed in general accordance with our proposal dated 27 May 2003, as authorized by Dewberry & Davis, Inc. The attached report presents our understanding of the project, reviews our exploration procedures, describes existing site and general subsurface conditions, and presents our preliminary evaluations, conclusions, and recommendations.

We have enjoyed working with you on this project, and we are prepared to assist you with a final geotechnical evaluation upon determination of finished grades and structure locations, as well as quality control 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.

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1.0 INTRODUCTION

1.1 Project Information

Project information was provided in various correspondences with Mr. Tim Reynolds of Dewberry & Davis, Inc. Included in the provided information was a site map drawing that was provided by Dewberry & Davis, Inc. on 25 August 2003. We understand that Dewberry & Davis, Inc. is in the preliminary stages of development of a "Right-Now" site located within the Brockman Business Park in Amherst, Virginia (See Site Vicinity Map, Drawing No. 1, Appendix A).

We understand that the "Right-Now" site will be an approximate 450-feet by 800-feet pre-graded, pad-ready, building site for immediate light industrial use. The planned building pad is split into an initial 100,000 square-feet space, with two future expansion areas covering approximately 200,000 square feet. As requested, our preliminary subsurface exploration, which consisted of a total of six test borings, was performed near the four corners of the initial 100,000 square-feet space, in an planned area of cut, and in a select ingress/egress location.

Based on provided information, the existing topography of the project site slopes downward from a high elevation of about 708 feet near the central portion of the planned site pad to a low elevation of 660 feet in its northeastern corner. We anticipate that cuts and fills of up to 22 feet and 24 feet, respectively, will be required to develop the site pad finished elevation of 686 feet. No future anticipated structural loads were available at the time this report was written. However, based on our previous experience, we have assumed that the future building will have maximum column and continuous wall loads of 100 kips and 3 kips per linear foot, respectively.

We note that our preliminary subsurface exploration for the Brockman Business Park – Right Now Site (Site #1) was performed in conjunction with a preliminary exploration for an adjacent site (Site #2) located within the Brockman Business Park. Both projects were performed under the same F&R project number (Project No. E62-203G); however, a *Report of Preliminary Subsurface Exploration and Geotechnical Engineering Evaluation* for Site #2 has been submitted under a separate cover.

1.2 Scope of Services

The purpose of this preliminary subsurface exploration was to evaluate (with a limited number of borings) the subsurface soil conditions at the requested locations explored, primarily with respect to general subsurface characterization and excavation conditions. As requested, we performed limited laboratory testing on one bulk soil sampled from the site. As the limited test boring and laboratory data allowed, we have also commented on an available design bearing pressure range and preliminary pavement section design. Preliminary design parameters will require further review once definitive construction plans are developed. The additional review may require additional subsurface exploration as well as engineering analyses. In order to accomplish the preliminary exploration objectives, we undertook the following scope of services:

1) Visited the site to observe existing surface conditions and features and to mark boring locations.
2) Coordinated with Miss Utility services for utility clearance.
3) Reviewed readily available geologic and subsurface information relative to the project site.
4) Executed a preliminary subsurface exploration program consisting of six standard penetration test borings. Each test boring was drilled to the planned termination depth of 20 feet or auger refusal, whichever occurred first.
5) Collected bulk soil samples from two of boring locations and perform one laboratory soil classification, natural moisture, California Bearing Ratio (CBR), and standard Proctor moisture-density test.
6) Evaluated the findings of the test boring and laboratory test data relative general subsurface characterization and excavation conditions.
7) Prepared this written preliminary report summarizing our work on the project, providing general descriptions of the subsurface conditions encountered, and as the limited data allowed, providing preliminary foundation and pavement design recommendations, and discussing geotechnical related aspects of the proposed construction

Our scope of services did not include a survey of boring locations and elevations, rock coring, quantity estimates, preparation of plans or specifications, or the identification and evaluation of environmental aspects of the project site.

2.0 SUBSURFACE EXPLORATION PROCEDURES

Our subsurface exploration program consisted of six test borings (designated B-1 through B-6) performed in the general locations requested by Dewberry & Davis, Inc. We note that test boring B-3 was relocated approximately 75 feet to the southwest due to inaccessible conditions (dense woods) that existed at the requested location. The test borings were performed on 10 September 2003 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 by taping and/or otherwise estimating distances from existing features indicated on the provided topographic plan. In addition, ground surface elevations were interpolated from the contour information shown on the provided plan. No claim is made as to the accuracy of the information contained in the provided documents. In consideration of the methods used in their determination, the boring locations and elevations shown on the attached Boring Location Plan and boring logs should be considered approximate.

The test borings were performed in accordance with generally accepted practice using an All-Terrain Vehicle (ATV)-mounted CME-55 rotary drill rig. 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. Utilizing an automatic hammer, the split-spoon sampler was driven into the soil by freely dropping a weight of 140 pounds from a height of 30 inches. 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 10 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 10 blows, the N-value is reported as 10/0. Otherwise, the depth of penetration after 50 blows is reported in inches, i.e. 50/5, 50/2, etc.

The test borings were advanced through the soil overburden to a planned termination depth. Subsurface water level readings were taken in each of the borings immediately upon completion of the 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 evaluated 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 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 and bulk 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 situated 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 project site is an approximate 10-acre parcel that is generally grass-covered along its southern portion and heavily wooded along its northern portion. The topography of the site generally slopes downward in all directions from a high elevation of about 708 feet near the central portion of the planned site pad to a low elevation of 660 feet in its northeastern corner, resulting in a maximum change in elevation of about 48 feet. No surface water or existing structures were observed within the project site.

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 judgment. The transitions between different soil strata are usually less distinct than those shown on the boring logs and estimated subsurface profiles. 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 are shown on the attached boring logs in Appendix B.

Below the existing ground surface, the test borings generally encountered surficial soils underlain by residual soils, partially weathered rock, and auger refusal materials. These materials are generally discussed in the following paragraphs.

3.3.2 Surficial Soils

Surficial soils typically contain root mat and/or other fibrous organic matter and are generally unsuitable for engineering purposes. Surficial soils were encountered in each test boring to a depth ranging from 5 to 6 inches. Actual surficial soil depths may vary in unexplored areas of the site.

3.3.3 Residual Soils

Residual soils, formed by the in-place weathering of the parent rock, were encountered in each test boring. With the exception of boring B-6, residual soils were encountered to the termination depth of 20 feet below existing site grades. In boring B-6, residual soils were encountered to a depth of 8 feet. Sampled residual soils were generally described as clays (CL/CH), silts (ML/MH), and silty sands (SM). Standard penetration resistance in the sampled residuum ranged from 7 to 60 blows per foot (bpf); however, we note that the higher N-values (greater than 30 bpf) were generally associated with boring B-6 conditions. In test borings B-1 through B-5, standard penetration resistances within the residual soils typically ranged from 7 to 22 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 in test boring B-6 at a depth of approximately 8 feet below the existing ground surface. Sampled PWR was generally described as silty sand (SM). Standard penetration resistance within the PWR ranged from 50 blows per 3 inches of split-spoon penetration to 50 blows per no split-spoon penetration.

3.3.5 Auger Refusal

Auger refusal occurs when materials are encountered that cannot be penetrated by the soil auger and is normally indicative of a hard or very dense material, such as boulders, rock lenses, pinnacles, impenetrable debris within fill, or the upper surface of bedrock. Refusal was encountered in test boring B-6 at a depth of approximately 15.5 feet below the existing ground surface.

Auger refusal discussed herein is based on conditions impenetrable to our drilling equipment (CME 55). Auger refusal conditions with a CME 55 do not necessarily indicate conditions impenetrable to other equipment. Auger refusal conditions may exist intermediate of the boring locations or in unexplored areas of the site.

3.3.6 Subsurface Water

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

A bulk soil sample obtained from test boring B-4 was tested for moisture content (ASTM D 2216), Atterberg limits (ASTM D 4318), percent passing #200 sieve (ASTM D 1140), standard Proctor moisture-density relationship (ASTM D 698), and California Bearing Ratio (ASTM D 1883). The results of the laboratory tests are summarized in the following tables, and specific results of the standard Proctor and CBR tests are provided in Appendix C.

Soil Classification Test Summary

Boring No.	Sample Depth (ft)	Sample Type	% Retained on No. 4 Sieve	% Finer than No. 200 Sieve	Atterberg Limits			USCS Classification
					L.L.	P.L.	P.I.	
B-4	0 – 10	Bulk	0.0	82.0	66	34	32	orange brown SILT (MH) with sand

Note: Bulk = bulk sample

Standard Proctor and CBR Test Summary

Boring No.	Sample Depth (ft)	Sample Type	Optimum Moisture Content (%)	Maximum Dry Density (pcf)	CBR
B-4	0 – 10	Bulk	29.7	88.6	10.5

4.0 PRELIMINARY DESIGN RECOMMENDATIONS

4.1 General

The following evaluations and preliminary recommendations are based on our observations at the site, interpretation of the field and laboratory data obtained during this exploration, and our experience with similar subsurface conditions and projects. Soil penetration data have been used to estimate an allowable bearing pressure range using established correlations. Subsurface conditions in unexplored locations may vary from those encountered. When final structure type, loadings, and elevations are determined, we request that we be advised so that we may reevaluate our preliminary 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 Preliminary Foundation Design

Based on the limited subsurface and structural information, we envision that the proposed development can be supported on a shallow foundation system bearing on undisturbed residual soils or controlled compacted fill (see controlled fill recommendations). For an anticipated relatively light structure, we envision that an allowable design bearing pressure in the range of 2,500 to 4,000 pounds per square foot (psf) should be suitable for footings bearing on undisturbed residual soils or compacted fill materials. The actual appropriate design bearing pressure to be used should consider the final structure loads, location, and elevation. Generally, we anticipate that an appropriately selected design bearing pressure within this range would result in a total settlement of less than 1 inch. However, once structure location, loading, and elevations are determined, a specific design bearing pressure can be provided and settlement estimates can be evaluated.

To reduce the possibility of localized shear failures, spread and strip footings should be a minimum of 3 feet and 2 feet wide, respectively. We recommend that exterior footings be constructed at least 2 feet below adjacent grades in order to bear below normal frost depth.

4.3 Ground Floor Slabs

Ground floor slabs may be designed as a slab-on-grade supported by undisturbed residual soils or newly placed controlled fill. 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 minimize 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.

4.4 General Slope Stability

Our preliminary exploration did not include a detailed analysis of slope stability for any temporary or permanent condition. However, within building, pavement, and landscaped areas, we generally recommend temporary slopes no steeper than 1.5(H):1(V) and permanent slopes no steeper than 2(H):1(V) up to a maximum height of 20 feet for construction in undisturbed residual soils or newly compacted structural fill placed in accordance with our recommendations. In addition, in building and pavement areas, minimum top of slope setbacks of 10 feet and 5 feet are recommended, respectively.

During construction, temporary slopes should be regularly evaluated for signs of movement or unsafe conditions. Soil slopes should be covered for protection from rain, and surface runoff should be diverted away from the slopes. For erosion protection, a protective cover of grass or other vegetation should be established on permanent soil slopes as soon as possible.

These general slope recommendations are appropriate for slopes underlain by competent materials. However, the provided recommendations should not be used to deviate from OSHA regulations. Construction should be performed in accordance with applicable OSHA regulations.

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4.5 Pavement Design

The following preliminary pavement design recommendations were develop based on the *1993 AASHTO Guide for Design of Pavement Structures* and the following assumptions:

- a 20-Year design life
- a design CBR of 5
  (Our design CBR value was developed based on our experience with soils similar to those encountered at the project site, the laboratory determined CBR value of 10.5, and the laboratory CBR value of 4.4 determined in testing for the adjacent Site #2 project discussed in a separate *Report of Preliminary Subsurface Exploration and Geotechnical Engineering Evaluation*)
- assumed traffic loads consisting of passenger cars and light trucks for light-duty pavement design
- assumed traffic loads consisting of up to 40 trucks per day for heavy-duty pavement design
- subgrade soils supporting proposed pavements are evaluated and prepared in accordance with recommendations provided in this report

Based on the above assumptions, we recommend using the following asphalt pavement sections. A final evaluation of pavement design sections for light- and heavy-duty traffic conditions should be performed once definitive traffic loads and finished grades are determined. We note that additional testing may be required at that time.

PAVEMENT SECTION		STANDARD	HEAVY
LAYER	VDOT SPECIFICATION	THICKNESS (INCHES)	THICKNESS (INCHES)
Surface Course	Asphalt Concrete (SMA 9.5)	2.0	2.0
Base Course	Asphalt Concrete (BM-25)	--	3.0
Subbase Course	Type I Crushed Aggregate (No. 21A or No. 21B)	8.0	8.0

Asphalt paved parking lots are typical for the region of this project and are anticipated. However, it is recommended that the approaches, loading and unloading areas, main turnaround areas, and other areas subjected to excessive starting and stopping motion, be supported with concrete pavement constructed in general accordance with ACI 330R-92. For pavements restricted to light-duty traffic and where excessive starting and stopping motions are anticipated, we recommend the pavement be constructed of 4-inch thick concrete. For pavements subject to heavy-duty traffic with excessive starting and stopping motions, we recommend that the pavement be constructed of 6.5-inch thick concrete.

Our pavement recommendations are based on pavements being supported on soils similar to the soils encountered during our subsurface exploration. Fill materials underlying pavements should be placed in accordance with the controlled fill and pavement subgrade recommendations contained in this report. In addition, all pavement subgrades should be evaluated by a geotechnical engineer prior to base stone placement. If excessive subgrade movement is observed, appropriate improvements such as undercutting and/or in-place stabilization will be required at that time.

The aggregate subbase course should be placed, compacted, and tested in general accordance with the requirements of Section 309 of *Virginia Department of Transportation Road and Bridge Specifications,* January 1994 (VDOT Specifications).

We recommend that the asphalt concrete base course and surface course be placed and compacted in general accordance with the requirements of VDOT Specifications Section 315. In addition, acceptable compaction should be defined as a test section density within the range of 98% to 102% of the maximum density determined on a density control strip constructed by an approved roller at the start of paving operations for the course mix. The size of test sections should be determined based on field observations made by experienced testing personnel. A minimum of five density tests should be performed in each test section and the results averaged. In addition to the average required compaction recommended above, no one test should be below 95% compaction.

5.0 PRELIMINARY CONSTRUCTION RECOMMENDATIONS

5.1 Site Preparation

Before proceeding with construction, any existing surficial soils, existing utilities, and 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 foundations, pavements, floor slabs, and new fill 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. Any unsuitable materials observed during the evaluation and proofrolling operations should be undercut and replaced with compacted fill and/or stabilized in-place.

The proofrolling observation is an opportunity for the geotechnical engineer to locate inconsistencies intermediate of our boring locations in the existing subgrade. Any unsuitable materials observed during the evaluation and proofrolling operations should be undercut and replaced with compacted fill or stabilized in-place. The possible need for, and extent of, undercutting and/or in-place stabilization required could 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 will 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 residual soils encountered on-site soils or an off-site borrow having a classification of CL, ML, or SM as defined by the Unified Soil Classification System. In addition, excavated partially weathered rock should also be acceptable for use as fill material provided that the placement and compactive process adequately pulverizes the material. 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.

We typically recommend a minimum standard Proctor (ASTM D 698) maximum dry density of approximately 90 pounds per cubic feet (pcf) for fill materials. However, based on the laboratory testing, the available on-site materials have a slightly lower maximum dry density of about 88.6 pcf. We do not anticipate this to be detrimental to the project; however, due to the lower laboratory-determined maximum dry density, we recommend using a higher degree of compaction to compensate.

Fill materials should be placed in horizontal lifts with a 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. Due to the lighter weight characteristics of the planned cut soils, we recommend that structural fill be compacted to at least 100 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.

We note that the sandy silts (ML/MH) and silt sands (SM) encountered in the exploration will tend to be more moisture sensitive than other typical piedmont residual soils. These materials will often exhibit near-surface shearing as open subgrades. The soil's silt and mica content provides a somewhat "slick" component resulting in an apparent low shear strength, especially in an unconfined state.

As a result, open subgrades for pavement or slab support will likely exhibit surface shearing under wheel loads and will not hold up well to construction activities. A layer of crushed stone quickly placed after subgrade preparation and after subgrade verification by a geotechnical engineer will help confine the subgrade soils and reduce imminent disturbance from construction activities. Conversely, in a confined state such as a small footing subgrade not subject to on-going construction traffic, the subgrade should perform adequately.

We know from our previous experience with similar soils that this type of material will be powdery when 1 to 2 percent dry of its optimum moisture content (per ASTM D-698, standard Proctor) and saturated to the point of pumping when 1 to 2 percent wet. Within its workable moisture range (perhaps +/- 1 percent), this material can be compacted to meet project specifications. However, it should be noted that due to its moisture sensitivity and surface shearing characteristics, earthwork and open at-grade subgrade preparation could be more problematic than typical.

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 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. Field density tests to verify fill compaction should be performed for every 5,000 square feet (approximately 70 feet square) of fill area, with a minimum of two tests per lift. In confined areas, a greater frequency may be required.

5.4 Excavation Conditions

Based on the preliminary grading information available at the time this report was written, we do not anticipate difficult excavation conditions will be encountered within the planned site pad footprint. However, we do note that dense residual soils, partially weathered rock, and auger refusal material was encountered in test boring B-6 at a depth of about 3 feet below the existing site grade, respectively. Therefore, difficult excavation techniques should be anticipated in the vicinity of the boring B-6 location.

In mass excavations for general sitework, partially weathered rock and dense or hard soils (soils with standard penetration resistances of 30 or more blows per foot) can usually be removed by ripping with a single-tooth ripper attached to a large crawler tractor or by breaking it out with a tracked excavator or large front-end loader. However, we note that while ripping and/or breaking out with large tracked equipment might be possible, it may be time prohibitive for deep mass excavations. Blasting can be performed to facilitate the excavation effort where time is a controlling factor. In confined excavations such as foundations, utility trenches, elevator pits, etc., removal or partially weathered rock typically requires use of large backhoes, pneumatic spades, or light blasting.

Refusal materials will normally require blasting for removal in all types of excavations. Any blasting in footing excavations must be done carefully to prevent damage to the bearing materials. Blasting should be performed by an experienced and licensed specialty contractor familiar with local practice and regulations. The gradation of the material removed by ripping or blasting will probably be erratic. Excavated rock is generally unsuitable for use as structural fill and should be wasted. It is sometimes feasible to use rock material in the deeper parts of architectural or driveway and parking lot fills. Rock placed in non-structural areas should be well choked with soil fill and compacted. Any soil/rock fill should be capped with a minimum of 5 feet of clean soil fill.

The definition of rock can be a source of conflict during construction. The following definitions have been incorporated into specifications on other projects and are provided for your general guidance:

GENERAL EXCAVATION:

Rip Rock - Any material that cannot be removed by scrapers, loaders, pans, dozers, or graders; and requires the use of a single-tooth ripper mounted on a crawler tractor having a minimum draw bar pull rated at not less than 56,000 pounds.

Blast Rock - Any material which cannot be excavated with a single-tooth ripper mounted on a crawler tractor having a minimum draw bar pull rated at not less than 56,000 pounds (Caterpillar D-8K or equivalent) or by a Caterpillar 977 front-end loader or equivalent; and occupying an original volume of at least one (1) cubic yard.

TRENCH EXCAVATION:

Blast Rock - Any material which cannot be excavated with a backhoe having a bucket curling force rated at not less than 25,700 pounds (Caterpillar Model 225 or equivalent), and occupying an original volume of at least one-half (1/2) cubic yard.

5.5 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 do not generally anticipate that subsurface water will 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

Once definitive information with respect to structure type, location, loading, and elevations are determined, additional subsurface information may be required to provide final geotechnical design parameters and recommendations. Upon completion of a final geotechnical report and subsequent project design, 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. These services are not part of the currently authorized scope of work.

7.0 LIMITATIONS

This preliminary report has been prepared for the exclusive use of Dewberry & Davis, Inc. or their agent, for specific application to the Brockman Business Park – Right Now Site (Site #1) project located in Amherst, Virginia, in accordance with generally accepted soil and foundation engineering practices. No other warranty, express or implied, is made. Our preliminary conclusions and recommendations are based on the limited design information furnished to us, the data obtained from the previously described subsurface exploration program, and generally accepted geotechnical engineering practice. The preliminary 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.

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 preliminary 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 18 pages of text and the attached appendices.

APPENDIX A

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL ENGINEERING REPORT

As the client of a consulting geotechnical engineer, you should know that site subsurface conditions cause more construction problems than any other factor. ASFE/The Association of Engineering Firms Practicing in the Geosciences offers the following suggestions and observations to help you manage your risks.

A GEOTECHNICAL ENGINEERING REPORT IS BASED ON A UNIQUE SET OF PROJECT-SPECIFIC FACTORS

Your geotechnical engineering report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. These factors typically include: the general nature of the structure involved, its size, and configuration; the location of the structure on the site; other improvements, such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask your geotechnical engineer to evaluate how factors that change subsequent to the date of the report may affect the report's recommendations.

Unless your geotechnical engineer indicates otherwise, do not use your geotechnical engineering report:

- when the nature of the proposed structure is changed, for example, if an office building will be erected instead of a parking garage, or a refrigerated warehouse will be built instead of an unrefrigerated one;
- when the size, elevation, or configuration of the proposed structure is altered;
- when the location or orientation of the proposed structure is modified;
- when there is a change of ownership; or
- for application to an adjacent site.

Geotechnical engineers cannot accept responsibility for problems that may occur if they are not consulted after factors considered in their report's development have changed.

SUBSURFACE CONDITIONS CAN CHANGE

A geotechnical engineering report is based on conditions that existed at the time of subsurface exploration. Do not base construction decisions on a geotechnical engineering report whose adequacy may have been affected by time. Speak with your geotechnical consultant to learn if additional tests are advisable before construction starts.Note, too, that additional tests may be required when subsurface conditions are affected by construction operations at or adjacent to the site, or by natural events such as floods, earthquakes, or ground water fluctuations. Keep your geotechnical consultant apprised of any such events.

MOST GEOTECHNICAL FINDINGS ARE PROFESSIONAL JUDGMENTS

Site exploration identifies actual subsurface conditions only at those points where samples are taken. The data were extrapolated by your geotechnical engineer who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your geotechnical engineer can work together to help minimize their impact. Retaining your geotechnical engineer to observe construction can be particularly beneficial in this respect.

A REPORT'S RECOMMENDATIONS CAN ONLY BE PRELIMINARY

The construction recommendations included in your geotechnical engineer's report are preliminary, because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Because actual subsurface conditions can be discerned only during earthwork, you should retain your geotechnical engineer to observe actual conditions and to finalize recommendations. Only the geotechnical engineer who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations are valid and whether or not the contractor is abiding by applicable recommendations. The geotechnical engineer who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

GEOTECHNICAL SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND PERSONS

Consulting geotechnical engineers prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your geotechnical engineer prepared your report expressly for you and expressly for purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the geotechnical engineer. No party should apply this report for any purpose other than that originally contemplated without first conferring with the geotechnical engineer.

GEOENVIRONMENTAL CONCERNS ARE NOT AT ISSUE

Your geotechnical engineering report is not likely to relate any findings, conclusions, or recommendations about the potential for hazardous materials existing at the site. The equipment, techniques, and personnel used to perform a geoenvironmental exploration differ substantially from those applied in geotechnical engineering. Contamination can create major risks. If you have no information about the potential for your site being contaminated, you are advised to speak with your geotechnical consultant for information relating to geoenvironmental issues.

A GEOTECHNICAL ENGINEERING REPORT IS SUBJECT TO MISINTERPRETATION

Costly problems can occur when other design professionals develop their plans based on misinterpretations of a geotechnical engineering report. To help avoid misinterpretations, retain your geotechnical engineer to work with other project design professionals who are affected by the geotechnical report. Have your geotechnical engineer explain report implications to design professionals affected by them, and then review those design professionals' plans and specifications to see how they have incorporated geotechnical factors. Although certain other design professionals may be familiar with geotechnical concerns, none knows as much about them as a competent geotechnical engineer.

BORING LOGS SHOULD NOT BE SEPARATED FROM THE REPORT

Geotechnical engineers develop final boring logs based upon their interpretation of the field logs (assembled by site personnel) and laboratory evaluation of field samples. Geotechnical engineers customarily include only final boring logs in their reports. Final boring logs should not under any circumstances be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process. Although photographic reproduction eliminates this problem, it does nothing to minimize the possibility of contractors misinterpreting the logs during bid preparation. When this occurs, delays, disputes, and unanticipated costs are the all-too-frequent result.

To minimize the likelihood of boring log misinterpretation, give contractors ready access to the complete geotechnical engineering report prepared or authorized for their use. (If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared and that developing construction cost estimates was not one of the specific purposes for which it was prepared. In other words, while a contractor may gain important knowledge from a report prepared for another party, the contractor would be well-advised to discuss the report with your geotechnical engineer and to perform the additional or alternative work that the contractor believes may be needed to obtain the data specifically appropriate for construction cost estimating purposes.) Some clients believe that it is unwise or unnecessary to give contractors access to their geotechnical engineering reports because they hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems. It also helps reduce the adversarial attitudes that can aggravate problems to disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY

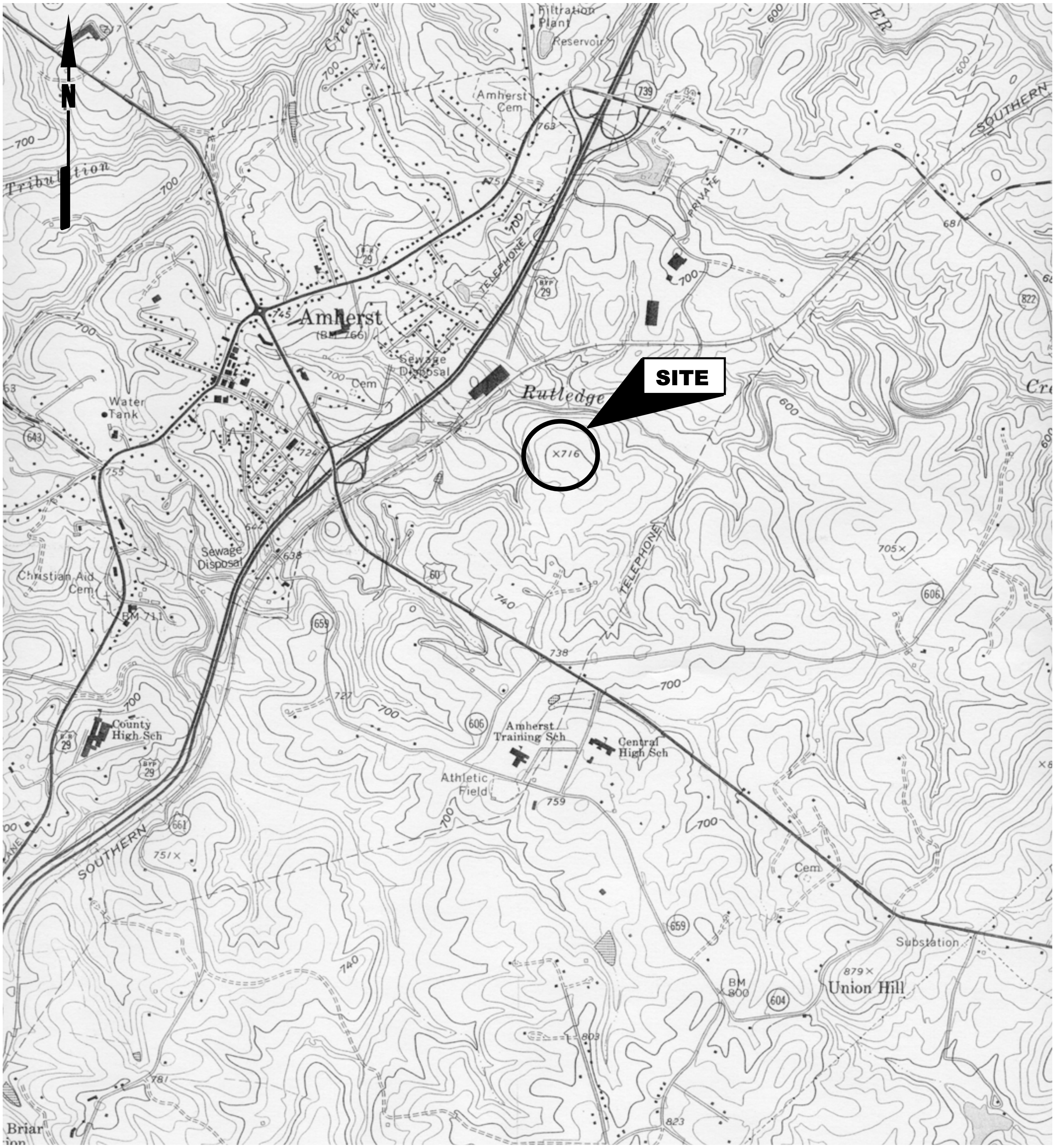
Because geotechnical engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against geotechnical engineers. To help prevent this problem, geotechnical engineers have developed a number of clauses for use in their contracts, reports, and other documents. Responsibility clauses are not exculpatory clauses designed to transfer geotechnical engineers' liabilities to other parties. Instead, they are definitive clauses that identify where geotechnical engineers' responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your geotechnical engineering report. Read them closely. Your geotechnical engineer will be pleased to give full and frank answers to any questions.

RELY ON THE GEOTECHNICAL ENGINEER FOR ADDITIONAL ASSISTANCE

Most ASFE-member consulting geotechnical engineering firms are familiar with a variety of techniques and approaches that can be used to help reduce risks for all parties to a construction project, from design through construction. Speak with your geotechnical engineer not only about geotechnical issues, but others as well, to learn about approaches that may be of genuine benefit. You may also wish to obtain certain ASFE publications. Contact a member of ASFE of ASFE for a complimentary directory of ASFE publications.

ASFE THE ASSOCIATION OF ENGINEERING FIRMS PRACTICING IN THE GEOSCIENCES
8811 COLESVILLE ROAD/SUITE G106/SILVER SPRING, MD 20910
TELEPHONE: 301/565-2733 FACSIMILE: 301/589-2017

BPC0592A/3.5M



Note: Adapted from the USGS 7.5 minute series topographic map: Amherst, Virginia, Quadrangle, 1963 (photorevised 1978)

FROEHLING & ROBERTSON, INC. GEOTECHNICAL • ENVIRONMENTAL • MATERIALS ENGINEERS • LABORATORIES *"OVER ONE HUNDRED YEARS OF SERVICE"*	**DATE:** October 2003 **SCALE:** 1 : 24,000	
	DRAWN: JTM	E62-203G
Dewberry & Davis, Inc. Brockman Business Park – Right Now Site (Site #1) Amherst, Virginia	SITE VICINITY MAP	**DRAWING NO.** 1

APPENDIX B

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	Soil Classification: 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]	$Cu \geq 4$ and $1 \leq Cc \leq 3$[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]	$Cu \geq 6$ and $1 \leq Cc \leq 3$[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

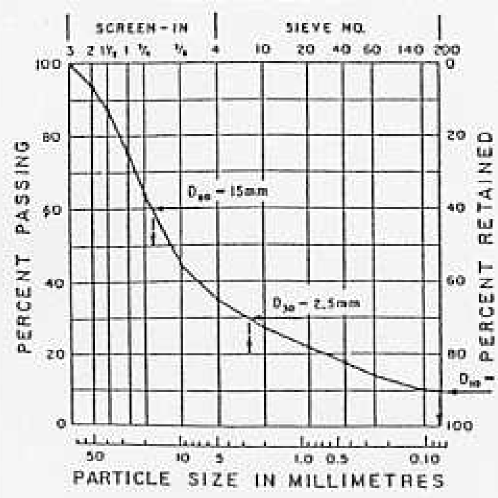
[A]Based on the material passing the 3-in. (75-mm) sieve
[B]If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.
[C]Gravels with 5 to 12% fines require dual symbols:
GW-GM well-graded gravel with silt
GW-GC well-graded gravel with clay
GP-GM poorly 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] $Cu = D_{60}/D_{10}$, $Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$

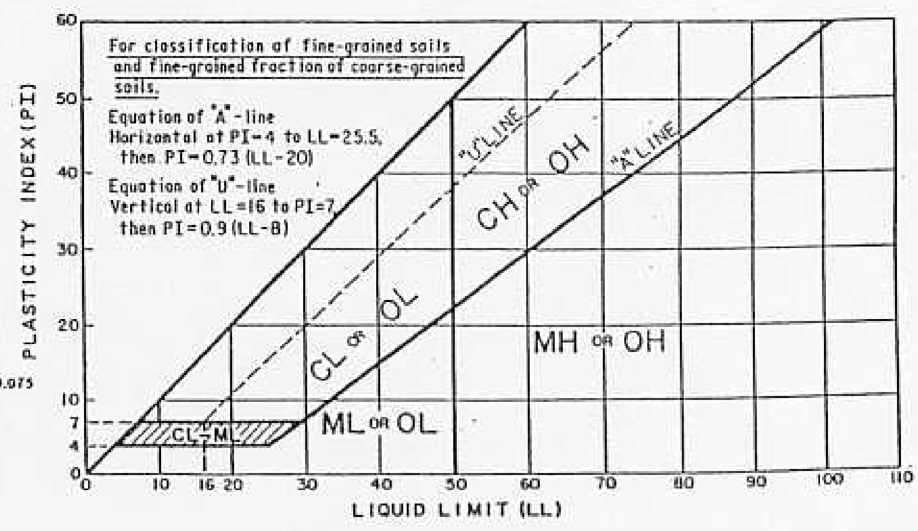
[F]If soil contains ≥ 15% sand, add "with sand" to group name.
[G]If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.
[H]If fines are organic, add "with organic fines" to group name.
[I]If soil contains ≥ 15% gravel, add "with gravel" to group name.

[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



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



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	> 50%
Cobble	3 - 12 inch		(e.g., SAND, CLAY)	
Gravel-Coarse	3/4 - 3 inch			
-Fine	#4 - 3/4 inch	Secondary	Adjective	20% - 50%
Sand-Coarse	#10 - #4		(e.g., sandy, clayey)	
-Medium	#40 - #10			
-Fine	#200 - #40	Minor	Some	15% - 25%
Silt (non-cohesive)	< #200		Little	5% - 15%
Clay (cohesive)	< #200		Trace	0% - 5%

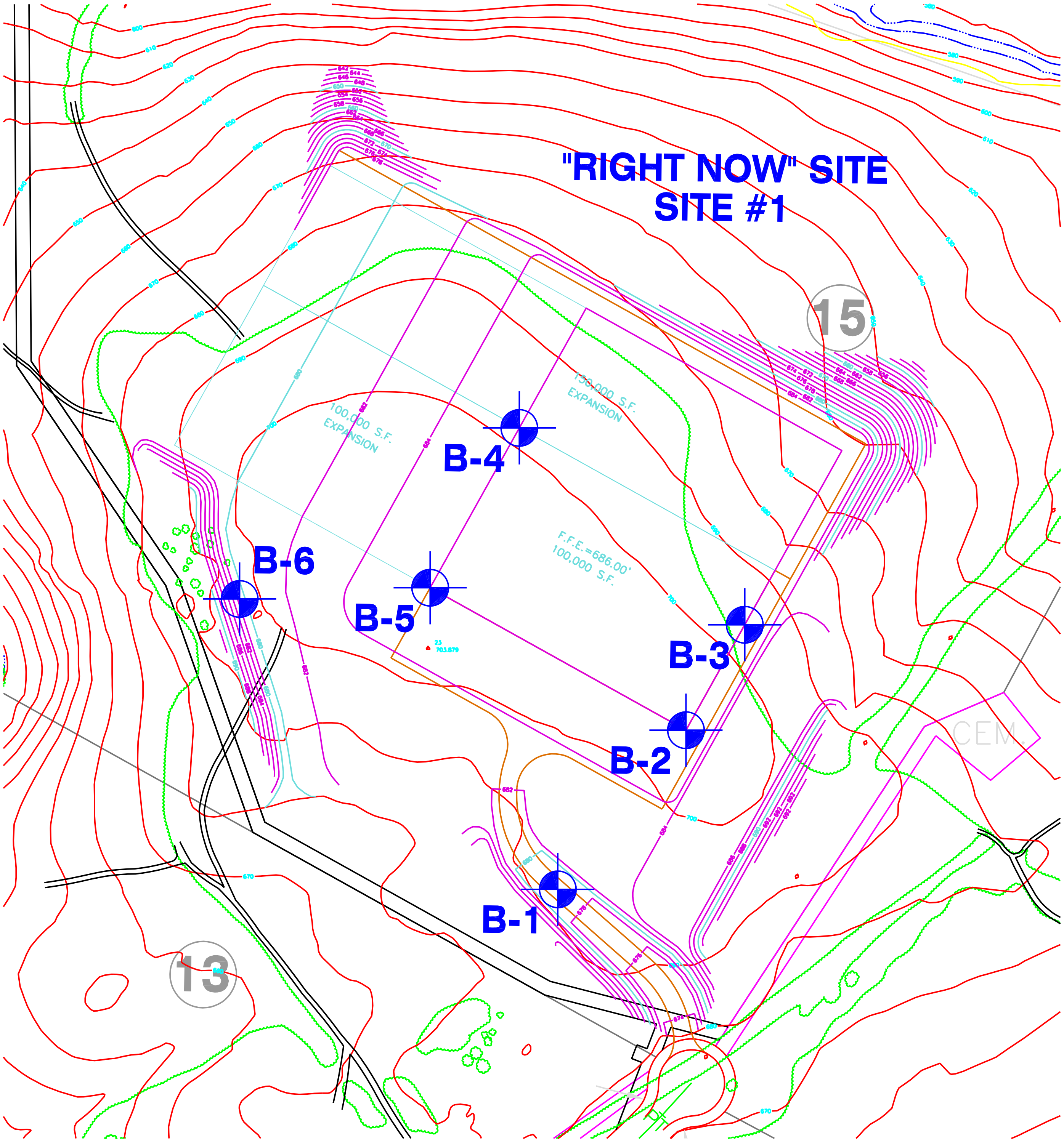
Notes:
1. Particle size is designated by U.S. Standard Sieve Sizes
2. Because of the small size of the split-spoon sampler relative to the size of gravel, the true percentage of gravel may not be accurately estimated.

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.



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

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	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

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SINCE 1881

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: **692ft ±** 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 2.5	13	Subsurface water was not encountered immediately upon completion of drilling.
689.0	3.0	Loose, red brown, moist, silty fine SAND (SM) with trace mica	3-4-4	3.5 5.0	8	
			3-4-5	6.0 7.5	9	
			3-4-5	8.5 10.0	9	
682.0	10.0	Medium dense, brown, moist, silty fine SAND (SM) with trace mica	4-5-7	13.5 15.0	12	
675.0	17.0	Stiff, brown, moist, fine sandy SILT (ML) with trace mica	2-4-6	18.5 20.0	10	
672.0	20.0	Boring terminated at 20 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.

BORING LOG

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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: **705ft ±** 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 - 2.5	15	Subsurface water was not encountered immediately upon completion of drilling.
702.0	3.0	Stiff, red brown, moist, SILT (ML/MH) with little fine sand and trace mica	5-6-7	3.5 - 5.0	13	
699.5	5.5	Medium stiff, red brown and brown, moist, fine sandy SILT (ML) with trace mica	3-4-4	6.0 - 7.5	8	
697.0	8.0	Loose to medium dense, brown, moist, silty fine SAND (SM) with trace mica	2-3-4	8.5 - 10.0	7	
			3-3-5	13.5 - 15.0	8	
			4-5-6	18.5 - 20.0	11	
685.0	20.0	Boring terminated at 20 feet				

BORING_LOG E62-203G.GPJ F&R.GDT 7/7/04

*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.

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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: **696ft ±** 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 - 2.5	9	Subsurface water was not encountered immediately upon completion of drilling.
693.0	3.0	Very stiff, red brown, moist, SILT (ML/MH) with little fine sand and trace mica	6-8-10	3.5 - 5.0	18	
690.5	5.5	Very stiff, red brown, moist, fine sandy SILT (ML/MH) with trace mica	5-6-9	6.0 - 7.5	15	
688.0	8.0	Loose to medium dense, orange brown, moist, silty fine SAND (SM) with trace mica	3-4-4	8.5 - 10.0	8	
		- brown and tan from 12 to 20 feet				
			4-4-4	13.5 - 15.0	8	
			5-6-8	18.5 - 20.0	14	
676.0	20.0	Boring terminated at 20 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.

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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: **698ft ±** 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 - 2.5	11	Subsurface water was not encountered immediately upon completion of drilling.
			5-10-12	3.5 - 5.0	22	
			5-6-8	6.0 - 7.5	14	
690.0	8.0	Very stiff, orange brown, moist, SILT (ML/MH) with some fine sand and trace mica	6-8-9	8.5 - 10.0	17	
686.0	12.0	Medium dense, brown, moist, silty fine SAND (SM) with trace mica	6-8-8	13.5 - 15.0	16	
			6-7-8	18.5 - 20.0	15	
678.0	20.0	Boring terminated at 20 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.

BORING_LOG E62-203G.GPJ F&R.GDT 7/7/04

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SINCE 1881

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: **705ft ±** 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

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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: **695ft ±** Location: **See boring location plan**

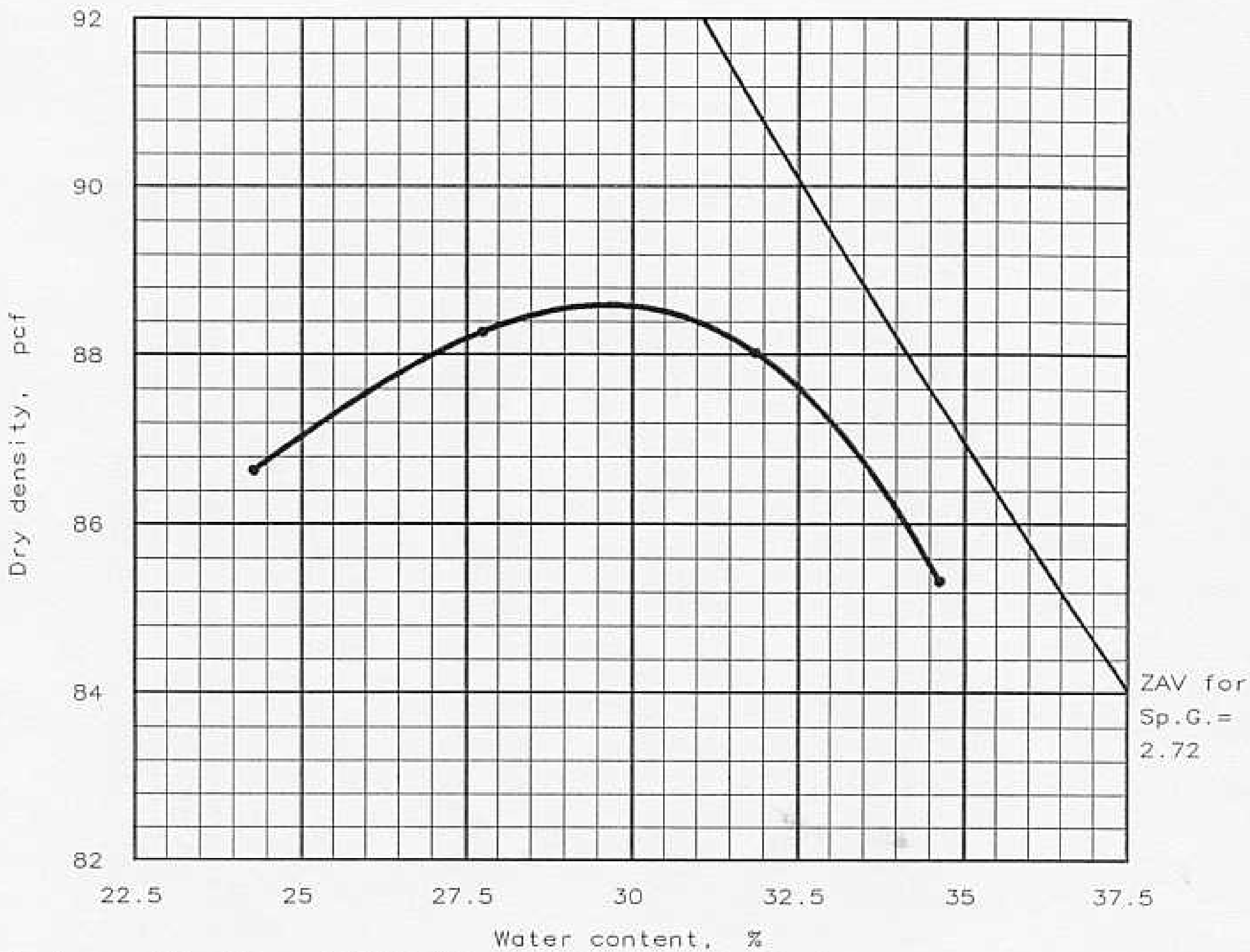
Type of Boring: **2.25" ID HSA CME 55** Started: **9/10/03** Completed: **9/10/03** Driller: **B. Maxson**

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 2.5	9	Subsurface water was not encountered immediately upon completion of drilling.
692.0	3.0	Dense to very dense, dark gray brown, moist, silty fine to medium SAND (SM)	22-23-26	3.5 5.0	49	
			14-26-34	6.0 7.5	60	
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	100+	
			34-50/3	13.5 14.3	100+	
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.

APPENDIX C

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

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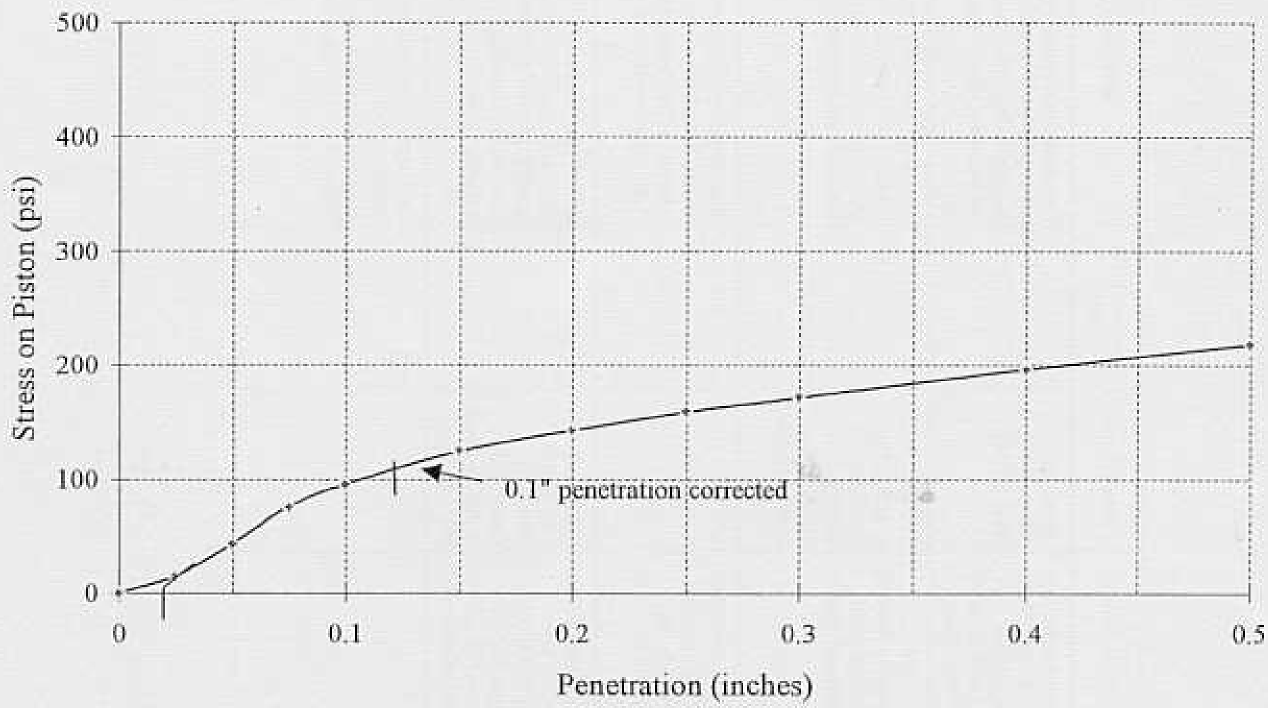
1734 Seibel Drive, N.E., Roanoke, Virginia 24012
Telephone: (540) 344-7939 Fax: (540) 344-3657

California Bearing Ratio (ASTM D 1883)

Record No.: E62-203G

Test Date: 07-Oct-03
Client: Dewberry & Davis, Inc.
Tested By: M R Henry
Project: Brockman Business Park - Right Now Site (Site #1) Amherst, Virginia
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: ______________________