County of Roanoke

FINANCE DEPARTMENT
PURCHASING DIVISION

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February 15, 2018

IFB # 2018-038
Hollins Fire and Rescue #5 Lot Repairs

ADDENDUM NO. 1

Pre-Bid Meeting Notes
Geotechnical Report

Due Date & Time:
February 23, 2:00 PM
(Local Prevailing Time)

IFB # 2018-038

Hollins Fire and Rescue #5 Lot Repairs

ADDENDUM NO. 1

1. **Extension of Due Date** – Please note that the due date and time for this IFB has been extended to: **Friday, February 23, 2018 at 2:00 PM**

2. **Pre-Bid Meeting Notes** – Please see the following notes, confirmations, and further specifications as discussed at the non-mandatory pre-bid meeting held at Hollins Fire and Rescue #5 on February 12, 2018. Bids submitted shall abide by any new information contained below in order to be considered compliant with IFB 2018-038.

   a) How long each access to the building can be closed? The County understands and anticipates that replacement of the concrete pad in the base bid will limit pull through bay access for a reasonable continuous construction period to allow for demo of the existing pad, new pad placement, and curing. Likewise, should asphalt or concrete work related to access to the bays from the front of the building facing Barrens Road be awarded in Alternate #1 or Alternate #2, work will be phased in such a manner as to allow for access to the bays from the rear lot.

   b) Are we sure the exact depth of existing concrete is 5 ½-6 inches? Soil borings were performed at select locations. The subsequent report provided by Froehling and Robertson, Inc. is attached to this addendum.

   c) How much does fire truck weigh? Fire truck=44,000lbs and Ladder Fire truck=72,000lbs

   d) Would BM-25 be allowed instead of IM-19A bituminous pavement for base course of asphalt? Yes

   e) The County leases the cellular tower location on the property accessed through the station lot. The Tenant and Sub-Tenants have 24-hour access to the equipment. The County will need to communicate access issues to the Tenant in advance of work that may impact.

   f) No markings on concrete pad and pavement markings are to be the same as before new pavement.

   g) Does the County want to consider distributed steel reinforcement as an additional bid alternate? No, please provide pricing for the existing base and alternate bid items.

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ATTACHMENT A TO
ADDENDUM NO. 1 TO IFB # 2018-038
Hollins Fire and Rescue #5 Lot Repairs
<u>Hollins Fire and Rescue Lot Geotechnical Report</u>

Report of Subsurface Exploration
and Pavement Commentary
Hollins Fire Station
Roanoke County, Virginia
F&R Project No. 62V0201

Prepared For:
Roanoke County
1216 Kessler Mill Road
Salem, Virginia 24153

Prepared By:
Froehling & Robertson, Inc.
1734 Seibel Drive, N.E.
Roanoke, Virginia 24012
Phone: 540.344.7939
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September 2017

FROEHLING & ROBERTSON, INC.
Engineering Stability Since 1881

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F&R Project No.: 62V0201 20 September 2017

Roanoke County
1216 Kessler Mill Road
Salem, Virginia 24153

Attention: Mr. Rob Light
Director - Roanoke County General Services

Subject: Report of Subsurface Exploration and Pavement Commentary
Hollins Fire Station
Roanoke County, Virginia

Mr. Light:

The purpose of this report is to present the results of the subsurface exploration program and pavement commentary undertaken by Froehling & Robertson, Inc. (F&R) in connection with the above referenced project. Our services were performed in general accordance with F&R Proposal No. 1662-00337 revised: 20 July 2017 as authorized by Roanoke County. The attached report presents our understanding of the project, reviews our exploration procedures, describes existing site and general subsurface conditions, and presents our pavement commentary.

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.

For

Erin K. Phillips, M.S., E.I.T. Stephen D. Hjelle, M.S., P.E.
Staff Engineer Geotechnical Department Manager

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

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1.1 INTRODUCTION

1.2 Project Information

Our understanding of the project is based on information provided by Mr. John Patten with Roanoke County in October 2015, additional updated information from Mr. Rob Light with Roanoke County in July 2017, and our experience with similar projects. We understand that the proposed project consists of the replacement of the concrete pavement at the fire station at Hollins in Roanoke County, Virginia (see Site Vicinity Map, Drawing No. 1).

The existing concrete pavement, as well as the existing asphalt pavement, has exhibited cracking and a full replacement of the existing concrete pavement is planned. Included in the provided information was a site aerial illustrating three requested boring locations.

1.3 Scope of Services

The purposes of our involvement on this project were to 1) provide general descriptions of the subsurface soil conditions at the locations explored and 2) comments regarding anticipated excavation conditions and pavement support capability as well as discussing geotechnical related aspects of the proposed construction. 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 and facility personnel.
3) Reviewed and summarized readily available geologic information relative to the project site.
4) Executed a subsurface exploration consisting of three borings drilled to depths ranging from 6.5 to 10 feet each. Boring B-1 was located on the existing concrete pavement at the north side of the fire station. Borings B-2 and B-3 were located on existing asphalt pavement at the east and west sides of the fire station, respectively.
5) Performed a laboratory testing program consisting of one California Bearing Ratio (CBR) with Standard Proctor and soil classification (Atterberg limits and wash #200) tests and three natural moisture content tests.
6) Prepared this written report summarizing our work on the project, providing descriptions of the subsurface conditions encountered and laboratory testing results, our impression regarding general pavement support conditions, and discussing geotechnical related aspects of the proposed construction. We understand that the results of the CBR testing will be used by others in development of an appropriate replacement pavement section for the project. Copies of the test boring logs and laboratory test results are included.

Our geotechnical scope of services did not include a survey of boring locations and elevations, quantity estimates, rock coring, pavement design, foundation design, preparation of plans or specifications, detention pond considerations, evaluations of earthquake motions, wetland delineation, or the evaluation of environmental aspects of the site.

2.0 SUBSURFACE EXPLORATION PROCEDURES

The subsurface exploration program consisted of three test borings (designated as B-1 through B-3) performed on 23 August 2017 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 site features shown on the provided site aerial image. In consideration of the methods used in their determination, the test boring locations shown on the attached Boring Location Plan should be considered approximate.

The SPT borings were performed in accordance with generally accepted practice using a track-mounted Dietrich D50 Turbo 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. 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.

An automatic hammer was used to perform the SPT boring on this project. Research has shown that the Standard Penetration Resistance (N-value) determined by an automatic hammer is different than the N-value determined by the safety hammer method. Most correlations that are published in the technical literature are based on the N-value determined by the safety hammer method. This is commonly termed N_{60} as the rope and cathead with a safety hammer delivers about 60 percent of the theoretical energy delivered by a 140-pound hammer falling 30 inches. Several researchers have proposed correction factors for the use of hammers other than the safety hammer to correct the values to be equivalent to the safety hammer SPT N_{60}-values. The correction is made using the following equation:

$$N_{60} = N_{field} \times C_E$$

N_{field} in the equation above is the SPT N-value as recorded with the equipment utilized in the field, and for our use of this equation, C_E a relative hammer efficiency ratio, i.e. our automatic hammer efficiency (specifically 86.2% for the track-mounted drill rig used on this project) divided by the theoretical N_{60} efficiency (60%). Accordingly, we recommend a correction factor (C_E) of approximately 1.44 for conversion of the recorded N_{field} values to normalized N_{60} values for the automatic hammers used on this project. We note that the N-values reported on the Boring Log included in this report are the actual, uncorrected, field derived N-values (N_{field}).

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. 50/1, etc.

Subsurface water level readings were taken in each of the test borings immediately upon completion of the drilling process. Upon completion of drilling, the boreholes were backfilled with auger cuttings (soil) and capped with a quick-setting cementitious grout. The surface was smoothed with a trowel. Periodic observation and maintenance of the boreholes should be performed to monitor for 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 based on visual observation and should be considered approximate. Copies of the boring logs are provided and classification procedures are further explained in the attachments to this letter.

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.

3.1 SITE AND SUBSURFACE CONDITIONS

3.2 Site Description

The proposed project site is the existing Hollins Volunteer Fire & Rescue (Fire Station) located at 7401 Barrens Road in Roanoke County, Virginia. The site is bound by an auto dealership to the north and east, Barrens Road to the south, and residential homes to the west. The ground cover is existing asphalt and concrete pavements. Based on observations of utility clearance efforts at the site, buried power, gas, communications, water and sewer lines are reportedly present in the project vicinity. Other undisclosed buried utilities may also be present.

3.3 Regional Geology

The site lies in the Valley and Ridge physiographic province of Virginia. Available geologic references (Geology Map of Virginia, 1993) report that the site is underlain by Ordovician-aged rocks of the Moccasin or Bays Formation through Blackford Formation. Locally, this formation is composed of dusky-red shale and mudstone; sandstone; gray limestone, in part cherty; and calcareous shale. Sometimes, these rocks weather to form a highly variable bedrock surface consisting of troughs and pinnacles which may greatly fluctuate in elevation within short lateral distances.

Limestone is composed predominantly of calcium carbonate. Impurities (i.e., silicates, sulfides, and other mineral groups) within these rock formations occur either as distinct beds of shale or siltstone, or may be widely dispersed throughout the rock. Carbonate rocks are susceptible to dissolution in the presence of acidic groundwater. The mineral residues remaining after the carbonates are eroded, and after shales and mudstones are altered by chemical weathering, are known as residual soils, and typically consist of medium to highly plastic silts and clays. Where the residual soils result from minerals that had been widely dispersed throughout the parent rock, the residual soils are likely to have a very low in-situ density and low shear strength, and are also likely to be highly compressible.

Continued subsurface dissolution of the carbonate bedrock may lead to development of a highly irregular rock profile that may include underground voids. Over time, the soils overlying a void may subside, in a continual process of subsurface chemical erosion of bedrock and infilling by overburden soils. The resulting ground surface depression is known as a sinkhole. Terrain characterized by sinkholes and other solutional features is known as karst.

There are numerous other variations on sinkhole development. Regardless of the mode of development, it is important to note that changes in soil stress and water regime can greatly accelerate sinkhole development. Natural geologic processes that might otherwise occur over thousands of years can occur within several years or even months. Construction activities such as site grading, building construction, and water impoundment have reportedly caused sinkholes to develop rapidly or to collapse suddenly. This site lies within a geologic formation known to contain solutional features; however, the potential for development of sinkholes, along with the rate at which a sinkhole will develop, are not easily determined or accurately predicted.

3.4 Subsurface Conditions

3.4.1 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. 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. Sometimes the relatively small volume of sample recovered does not allow for definitive origin classification. In these instances, the term “possible” is applied (i.e. possible residuum, etc.).

Below the existing ground surface, the borings generally encountered asphalt or concrete pavement underlain by existing fill materials or residual/possible residual soils over partially weathered rock, and auger refusal materials. These materials are generally discussed in the following paragraphs.

3.4.2 Asphalt and Crushed Stone

Existing asphalt pavement sections were encountered in Test Borings B-2 and B-3. The encountered asphalt pavement sections generally consisted of approximately 3.5 to 4.5 inches of an asphalt surface layer underlain by a base layer consisting of approximately 5 to 6 inches of crushed stone. An existing concrete pavement section was encountered in Test Boring B-1 consisting of 5.5 inches of concrete underlain by approximately 6 inches of crushed stone. We note that the drilling process tends to disturb the pavement and base stone during penetration and removal of the augers. Therefore, the measured pavement section thicknesses should be considered approximate. Actual depths of asphalt, concrete, and crushed stone may vary in unexplored areas of the site.

3.4.3 Existing Fill Materials

Existing fill materials include those materials deposited by man. Materials identified as existing fill were encountered in Test Borings B-1 and B-2 extending to approximately 6 feet below the existing ground surface. The fill soils generally consisted of clays (CL) and silts (ML). Standard penetration resistance in the sampled fill ranged from 4 to 12 blows per foot (bpf).

3.4.4 Residual/Possible Residual Soils

Residual soils, formed by the in-place weathering of the parent rock, were encountered below the existing pavement sections and/or fill materials in each of the test borings. Sampled residual/possible residual soils were generally described as clays (CL) and silts (ML). Standard penetration resistances within the sampled residuum ranged from 9 to 26 bpf.

3.4.5 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. Partially weathered rock is defined, for engineering purposes, as residual material with penetration resistances in excess of 100 blows per foot. PWR was encountered at the bottom of the residual soil profile in Boring B-3. Sampled PWR was comprised of mostly shale rock fragments and exhibited a penetration resistance of 50 blows per 1 inches of split-spoon penetration (50/1).

3.4.6 Auger Refusal Materials

Test Boring B-3 encountered auger refusal (AR) at a depth of 6.5 feet. AR occurs when materials are encountered that cannot be penetrated by the soil auger and is normally indicative of a very hard or very dense material, such as boulders, rock lenses, rock pinnacles, or the upper surface of rock.

Auger refusal discussed herein is based on conditions impenetrable to our drilling equipment (Dietrich D50 Turbo rotary drill rig). Auger refusal conditions with a Dietrich D50 do not necessarily indicate conditions impenetrable to other equipment. Auger refusal conditions may exist intermediate of the borings or in unexplored areas of the site.

3.4.7 Subsurface Water

Subsurface water for the purposes of this report is defined as water encountered below the existing ground surface. Measurable subsurface water 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

The following laboratory tests were performed on a selected split spoon sample in general accordance with ASTM International (ASTM) test methods: percent passing #200 sieve (ASTM D 1140), Atterberg limits (ASTM D 4318), CBR (ASTM D 1883) with Standard Proctor (ASTM D 698), and moisture content (ASTM D 2216). The results of the laboratory tests are summarized in the following table.

Soil Classification Test Summary

Bore No.	Sample Depth (ft)	Sample Type	% Natural Moisture Content	% Retained on No. 4 Sieve	% Finer than No. 200 Sieve	Atterberg Limits			USCS Classification
						L.L.	P.L.	P.I.	
B-1	1 - 8	Bulk	21.5	6	72	37	20	17	Brown CLAY (CL) with sand
B-2	2 – 4	Jar	27.2	0	53	39	24	15	Gray sandy CLAY (CL)

Standard Proctor and CBR Test Summary

Boring No.	Sample Depth (ft)	Natural Moisture Content (%)	Optimum Moisture Content (%)	Maximum Dry Density (pcf)	CBR
B-1	1 - 8	21.5	17.5	111.2	7.5

***Rock corrected values**

Natural Moisture Content Test Summary

Boring No.	Sample Depth (ft)	Natural Moisture Content (%)
B-1	0.5 – 2	20.0
B-2	0.5 – 2	28.5
B-3	0.5 – 2	21.4

4.1 PAVEMENT SUPPORT COMMENTARY

4.2 General

As discussed previously, although not widespread, areas of pavement distress were observed in both the concrete and asphalt pavements. The soil test borings were generally located in areas of observed pavement distress.

The following comments are based on interpretation of the field and laboratory data obtained during this exploration and our experience with similar subsurface conditions and projects. Subsurface conditions in unexplored locations may vary from those encountered. If pavement replacement locations, are changed, we should be notified and requested to confirm and, if necessary, re-evaluate our comments.

In general, method of construction greatly affects the soils intended for pavement support, consideration must be given to the implementation of suitable methods of site preparation, fill compaction, and other aspects of construction.

4.3 Support on Existing Fills

As previously noted, existing fill materials were encountered in Test Borings B-1 and B-2 to a depth of 6 feet below the existing ground surface. In general, construction on existing fill material requires a building owner to accept some risks due to unforeseen conditions within the material. Associated risks may be additional support related cost (i.e. undercutting, etc.) and excessive settlement. In order to eliminate the risks associated with structural support on existing fill materials, the existing materials could be completely removed and replaced with new controlled structural fill. However, based on the boring data obtained during our subsurface exploration, it appears that controlled structural fill placement as well as pavement support on the existing fill materials is possible with a reduced risk to the owner, provided the recommended engineering evaluations provided in this report are performed during construction and with the understanding that some undercutting and/or in-place stabilization may be recommended as a result of those time of construction evaluations.

4.4 Pavement Subgrade Support

Based on the boring and laboratory testing data it is our opinion that most of the existing site soils are suitable for pavement support. This would include approved existing fill and residual/possible residual soil subgrades. We note that lower consistency ($N \leq 5$ bpf) existing fill materials were encountered in Boring B-1 near the anticipated upper subgrade pavement bearing elevation. Given the presence of these existing, lower consistency, fill materials, while not anticipated to be extensive, the potential need for some undercutting at the time of construction should be understood.

5.1 CONSTRUCTION RECOMMENDATIONS

5.2 Site Preparation

Before proceeding with construction, any pavement sections, 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 and pavements 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 and provides an opportunity for the geotechnical engineer to locate inconsistencies intermediate of our boring locations.

Particular attention should be given to existing utility trenches within the proposed construction area. For obvious reasons, existing underground utility trenches were avoided in our drilling program. Our experience is that utility trenches are sometimes backfilled with very little compactive effort. Where utility lines are removed, the trench subgrade should be verified by an F&R representative prior to backfilling in accordance with the controlled structural fill recommendations provided in this report. If in-place abandonment is preferred, open conduits, pipes, or culverts should be grouted full and the overlying in-place backfill evaluated prior to at-grade construction.

As previously discussed, existing fill materials were encountered in Test Borings B-1 and B-2 to a depth of approximately 6 feet below the existing ground surface. In addition, lower consistency ($N \leq 5$ bpf) existing fill soils were encountered in Boring B-1 that are near the anticipated bearing level. Depending on how these materials respond during the subgrade evaluation and proofrolling operations, some in-place densification, undercutting, or in-place stabilization may be required. The actual extent of densification, undercutting and/or in-place stabilization required can best be determined by a representative of the geotechnical engineer at the time of construction. Once the site has been properly prepared, at-grade construction may proceed.

5.3 Controlled Structural Fill

Based on the boring data, controlled structural fill may be constructed using the non-organic on-site soils. However, we do not anticipate extensive grading efforts onsite, therefore an offsite borrow source may be required to balance the site. 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 to confirm that the recommended degree of compaction is attained.

In general, we recommend that the moisture content of fill soils be maintained within three percentage points of the optimum moisture content as determined from the Standard Proctor 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.

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 the season.

6.0 CONTINUATION OF SERVICES

We recommend that we be given the opportunity to review final site plans, 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 Roanoke County or their agent, for specific application to the Hollins Fire Station Pavement Replacement project in Roanoke County, 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 program, 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 and pavement 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 improvements, 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 13 pages of text and the attached appendices.

APPENDIX A

Important Information about This

Geotechnical-Engineering Report

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

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

The Geoprofessional Business Association (GBA) has prepared this advisory to help you – assumedly a client representative – interpret and apply this geotechnical-engineering report as effectively as possible. In that way, clients can benefit from a lowered exposure to the subsurface problems that, for decades, have been a principal cause of construction delays, cost overruns, claims, and disputes. If you have questions or want more information about any of the issues discussed below, contact your GBA-member geotechnical engineer. Active involvement in the Geoprofessional Business Association exposes geotechnical engineers to a wide array of risk-confrontation techniques that can be of genuine benefit for everyone involved with a construction project.

Geotechnical-Engineering 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 given civil engineer will not likely meet the needs of a civil-works constructor or even a different civil engineer. Because each geotechnical-engineering study is unique, each geotechnical-engineering report is unique, prepared *solely* for the client. *Those who rely on a geotechnical-engineering report prepared for a different client can be seriously misled.* No one except authorized client representatives should rely on this geotechnical-engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you – should apply this report for any purpose or project except the one originally contemplated.*

Read this Report in Full

Costly problems have occurred because those relying on a geotechnical-engineering report did not read it *in its entirety*. Do not rely on an executive summary. Do not read selected elements only. *Read this report in full*.

You Need to Inform Your Geotechnical Engineer about Change

Your geotechnical engineer considered unique, project-specific factors when designing the study behind this report and developing the confirmation-dependent recommendations the report conveys. A few typical factors include:

- the client's goals, objectives, budget, schedule, and risk-management preferences;
- the general nature of the structure involved, its size, configuration, and performance criteria;
- the structure's location and orientation on the site; and
- other planned or existing site improvements, such as retaining walls, access roads, parking lots, and underground utilities.

Typical changes that could erode the reliability of this report include those that affect:

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

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *The geotechnical engineer who prepared this report cannot accept responsibility or liability for problems that arise because the geotechnical engineer was not informed about developments the engineer otherwise would have considered*.

This Report May Not Be Reliable

Do not rely on this report if your geotechnical engineer prepared it:

- for a different client;
- for a different project;
- for a different site (that may or may not include all or a portion of the original site); or
- before important events occurred at the site or adjacent to it; e.g., man-made events like construction or environmental remediation, or natural events like floods, droughts, earthquakes, or groundwater fluctuations.

Note, too, that it could be unwise to rely on a geotechnical-engineering report whose reliability may have been affected by the passage of time, because of factors like changed subsurface conditions; new or modified codes, standards, or regulations; or new techniques or tools. *If your geotechnical engineer has not indicated an "apply-by" date on the report, ask what it should be*, and, in general, *if you are the least bit uncertain* about the continued reliability of this report, contact your geotechnical engineer before applying it. A minor amount of additional testing or analysis – if any is required at all – could prevent major problems.

Most of the "Findings" Related in This Report Are Professional Opinions

Before construction begins, geotechnical engineers explore a site's subsurface through various sampling and testing procedures. *Geotechnical engineers can observe actual subsurface conditions only at those specific locations where sampling and testing were performed*. The data derived from that sampling and testing were reviewed by your geotechnical engineer, who then applied professional judgment to form opinions about subsurface conditions throughout the site. Actual sitewide-subsurface conditions may differ – maybe significantly – from those indicated in this report. Confront that risk by retaining your geotechnical engineer to serve on the design team from project start to project finish, so the individual can provide informed guidance quickly, whenever needed.

This Report's Recommendations Are Confirmation-Dependent

The recommendations included in this report – including any options or alternatives – are confirmation-dependent. In other words, *they are not final*, because the geotechnical engineer who developed them relied heavily on judgment and opinion to do so. Your geotechnical engineer can finalize the recommendations *only after observing actual subsurface conditions* revealed during construction. If through observation your geotechnical engineer confirms that the conditions assumed to exist actually do exist, the recommendations can be relied upon, assuming no other changes have occurred. *The geotechnical engineer who prepared this report cannot assume responsibility or liability for confirmation-dependent recommendations if you fail to retain that engineer to perform construction observation*.

This Report Could Be Misinterpreted

Other design professionals' misinterpretation of geotechnical-engineering reports has resulted in costly problems. Confront that risk by having your geotechnical engineer serve as a full-time member of the design team, to:

- confer with other design-team members,
- help develop specifications,
- review pertinent elements of other design professionals' plans and specifications, and
- be on hand quickly whenever geotechnical-engineering guidance is needed.

You should also confront the risk of constructors misinterpreting this report. Do so by retaining your geotechnical engineer to participate in prebid and preconstruction conferences and to perform construction observation.

Give Constructors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can shift unanticipated-subsurface-conditions liability to constructors by limiting the information they provide for bid preparation. To help prevent the costly, contentious problems this practice has caused, include the complete geotechnical-engineering report, along with any attachments or appendices, with your contract documents, *but be certain to note conspicuously that you've included the material for informational purposes only*. To avoid misunderstanding, you may also want to note that "informational purposes" means constructors have no right to rely on the interpretations, opinions, conclusions, or recommendations in the report, but they may rely on the factual data relative to the specific times, locations, and depths/elevations referenced. Be certain that constructors know they may learn about specific project requirements, including options selected from the report, *only* from the design drawings and specifications. Remind constructors that they may perform their own studies if they want to, and *be sure to allow enough time* to permit them to do so. Only then might you be in a position to give constructors the information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions. Conducting prebid and preconstruction conferences can also be valuable in this respect.

Read Responsibility Provisions Closely

Some client representatives, design professionals, and constructors do not realize that geotechnical engineering is far less exact than other engineering disciplines. That lack of understanding has nurtured unrealistic expectations that have resulted in disappointments, delays, cost overruns, claims, and disputes. To confront that risk, geotechnical engineers commonly include 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 personnel, equipment, and techniques used to perform an environmental study - e.g., a "phase-one" or "phase-two" environmental site assessment – differ significantly from those used to perform a geotechnical-engineering study. For that reason, a geotechnical-engineering report does not usually relate any environmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated subsurface environmental problems have led to project failures*. If you have not yet obtained your own environmental information, ask your geotechnical consultant for risk-management guidance. As a general rule, *do not rely on an environmental report prepared for a different client, site, or project, or that is more than six months old.*

Obtain Professional Assistance to Deal with Moisture Infiltration and Mold

While your geotechnical engineer may have addressed groundwater, water infiltration, or similar issues in this report, none of the engineer's services were designed, conducted, or intended to prevent uncontrolled migration of moisture – including water vapor – from the soil through building slabs and walls and into the building interior, where it can cause mold growth and material-performance deficiencies. Accordingly, ***proper implementation of the geotechnical engineer's recommendations will not of itself be sufficient to prevent moisture infiltration.*** Confront the risk of moisture infiltration by including building-envelope or mold specialists on the design team. ***Geotechnical engineers are not building-envelope or mold specialists.***

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e-mail: info@geoprofessional.org www.geoprofessional.org

N

SITE

Adapted from the USGS 7.5 minute series topographic quadrangle:
Roanoke, VA (1984)

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:	September 2017	
SCALE:	As Shown	
DRAWN:	EKP	62V0201

Roanoke County
Hollins Fire Station
Roanoke County, Virginia

SITE
VICINITY
MAP

DRAWING NO.
1

APPENDIX B

CLASSIFICATION OF SOILS FOR ENGINEERING PURPOSES
ASTM Designation: D 2487
(Based on the Unified Soil Classification System)

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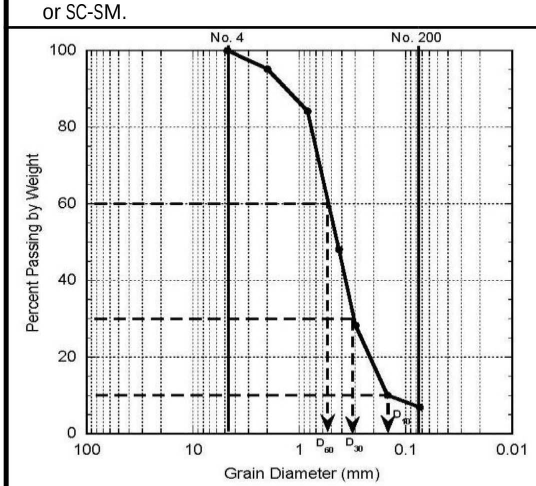
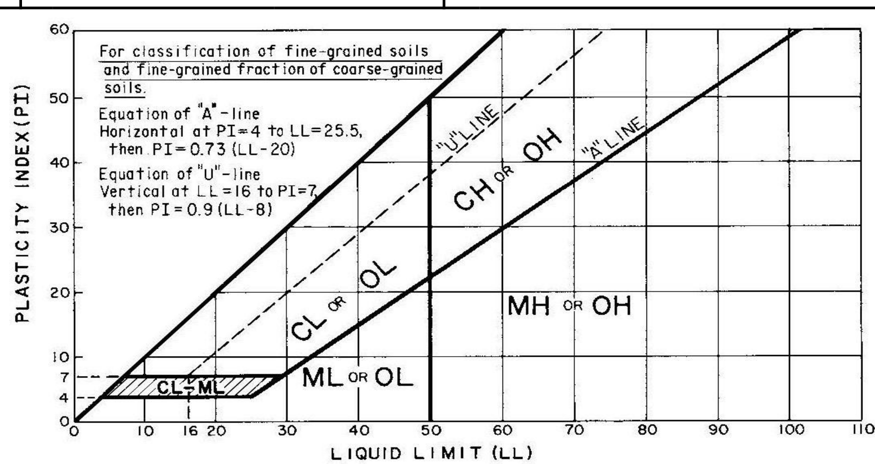
Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests[A]				Soil Classification: Group Symbol	Soil Classification: Group Name[B]
COARSE-GRAINED SOILS More than 50% retained on the No. 200 sieve	Gravels (More than 50% of coarse fraction retained on No. 4 sieve)	Clean gravels (Less than 5% fines[C])	$Cu \geq 4$ and $1 \leq Cc \leq 3$[D]	GW	Well-graded gravel[E]
			$Cu < 4$ and/or $[Cc < 1$ or $Cc > 3]$[D]	GP	Poorly graded gravel[E]
		Gravels with fines (More than 12% fines[C])	Fines classify as ML or MH	GM	Silty gravel[E,F,G]
			Fines classify as CL or CH	GC	Clayey gravel[E,F,G]
	Sands (50% or more of coarse fraction passes No. 4 sieve)	Clean Sands (Less than 5% fines[H])	$Cu \geq 6$ and $1 \leq Cc \leq 3$[D]	SW	Well-graded sand[I]
			$Cu < 6$ and/or $[Cc < 1$ or $Cc > 3]$[D]	SP	Poorly graded sand[I]
		Sands with fines (More than 12% fines[H])	Fines classify as ML or MH	SM	Silty sand[F,G,I]
			Fines classify as CL or CH	SC	Clayey sand[F,G,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	$\frac{\text{Liquid limit} - \text{oven dried}}{\text{Liquid limit} - \text{not dried}} < 0.75$	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	$\frac{\text{Liquid limit} - \text{oven dried}}{\text{Liquid limit} - \text{not dried}} < 0.75$	OH	Organic clay[K,L,M,P]
					Organic silt[K,L,M,Q]
HIGHLY ORGANIC SOILS	Primarily orgainic matter, dark in color, and organic in 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] [illegible]
[E] If soil contains ≥ 15 % sand, add "with sand" to group name.
[F] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.
[G] If fines are organic, add "with organic fines" to group name.
[H] 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
[I] If soil contains ≥ 15 % gravel, add "with gravel" to group name.
[J] If Atterburg limits plot in hatched area, soil is a CL-ML, silty clay.
[K] If soil contains 15 to < 30 % 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.





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.

	Boulder	Cobble	Gravel		Sand			Silt	Clay
			Coarse	Fine	Coarse	Medium	Fine		
Pass		12 in.	3 in.	3/4 in.	#4 M	#10 M	#40 M	#200 M	#200 M
Retained	12 in.	3 in.	3/4 in.	#4 M	#10 M	#40 M	#200 M		

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.

< 50% Fines (-200 Mesh) Descriptive Terms			> 50% Fines (-200 Mesh) Descriptive Terms		
Comp.	**Term**	**Percentage**	**Comp.**	**Term**	**Percentage**
Major	Uppercase Letters (GRAVEL, SAND)	% Gravel > % Sand	Major	Uppercase Letters (CLAY, SILT)	% Clay > % Silt
Secondary	With sand/gravel Adjective (Clayey, Silty)	≥ 15% Sand/Gravel ≥ 15% Fines	Secondary	Adjective (Sandy, Gravely) With gravel/sand	≥ 30% Coarse % Sand > % Gravel Rem. Coarse > 15%
Minor	With clay/silt Do Not Note	10% Fines ≤ 5% Fines	Minor	With gravel/sand Do Not Note	15% -25% Coarse <15% Coarse

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	Firm	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.

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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 (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				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

N

B-3

B-1

B-2

Adapted from *Google Maps* image. No claim is made as to the accuracy of the indicated boring locations other than for conceptual purposes to illustrate the exploration locations relative to intersection corners, etc. In consideration of the methods used in their determination, as well as the base map's accuracy, the test boring locations shown should be considered approximate.

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Engineering Stability Since 1881

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T 540.344.7939 I F 540.344.3657

DATE:	September 2017
SCALE:	As Shown (approx.)
DRAWN:	EKP — 62V0201

Roanoke County
Hollins Fire Station
Roanoke County, Virginia

CONCEPTUAL BORING LOCATION PLAN

DRAWING NO. 1

Froehling & Robertson, Inc.

BORING LOG

Boring: B-1 (1 of 1)

Project No: 62V0201
Client: County of Roanoke

Elevation:
Total Depth: 10.0'

Drilling Method: 2.25" ID HSA
Hammer Type: Automatic

Project: Hollins Fire Station Concrete Pavement Rep.
City/State: Roanoke, VA

Boring Location: See Boring Location Plan

Date Drilled: 8/23/17
Driller: B. Maxson

Elevation	Depth	Description of Materials (Classification)	* Sample Blows	Sample Depth (feet)	N-Value (blows/ft)	Remarks
		5.5" Concrete				
	0.5	6" Base Stone	5-2-2	0.5		Subsurface water was not encountered immediately upon completion of drilling
	1.0	**FILL:** Sampled as soft, gray-brwon, moist, CLAY (CL) with sand and occasional small rock fragments			4	
	2.0	Sampled as firm to stiff, light brown, moist, CLAY (CL) with sand	2-3-5-5	2.0	8	
			2-6-6-5	4.0	12	
	6.0	**POSSIBLE RESIDUUM:** Very stiff, orange-brown, moist, CLAY (CL)	5-12-12-16	6.0	24	
			4-7-10-13	8.0	17	
	10.0	Boring terminated at 10'		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.

Froehling & Robertson, Inc.

BORING LOG

Boring: B-2

Project No: 62V0201
Client: County of Roanoke

Elevation:
Total Depth: 10.0'

Drilling Method: 2.25" ID HSA
Hammer Type: Automatic

Project: Hollins Fire Station Concrete Pavement Rep.
Boring Location: See Boring Location Plan
City/State: Roanoke, VA

Date Drilled: 8/23/17
Driller: B. Maxson

Elevation	Depth	Description of Materials (Classification)	* Sample Blows	Sample Depth (feet)	N-Value (blows/ft)	Remarks
		3.5" Asphalt				
	0.3	6" Asphalt	12-3-5	0.5		Subsurface water was not encountered immediately upon completion of drilling
	0.8	**FILL:** Sampled as firm, mottled orange to light brown, moist, CLAY (CL) with sand and occasional rock fragments			8	
	2.0	Sampled as firm, gray, moist, sandy CLAY (CL)	2-3-3-4	2.0	6	
	4.0	Sampled as stiff, mottled gray-brown and orange, moist, CLAY (CL) with sand and occasional rock fragments	3-6-3-4	4.0	9	
	6.0	**POSSIBLE RESIDUUM:** Stiff, mottled light brown and orange, moist, CLAY (CL) with rock fragments	4-5-8-11	6.0	13	
	8.0	**RESIDUUM:** Stiff, dark gray, dry, SILT (ML) with shale rock fragments	3-4-8-17	8.0	12	
	10.0	Boring terminated at 10'		10.0		

BORING_LOG 62V-0201.GPJ F&R.GDT 9/

*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: B-3 (1 of 1)

Project No: 62V0201
Client: County of Roanoke

Elevation:
Total Depth: 6.5'

Drilling Method: 2.25" ID HSA
Hammer Type: Automatic

Project: Hollins Fire Station Concrete Pavement Rep.
City/State: Roanoke, VA

Boring Location: See Boring Location Plan

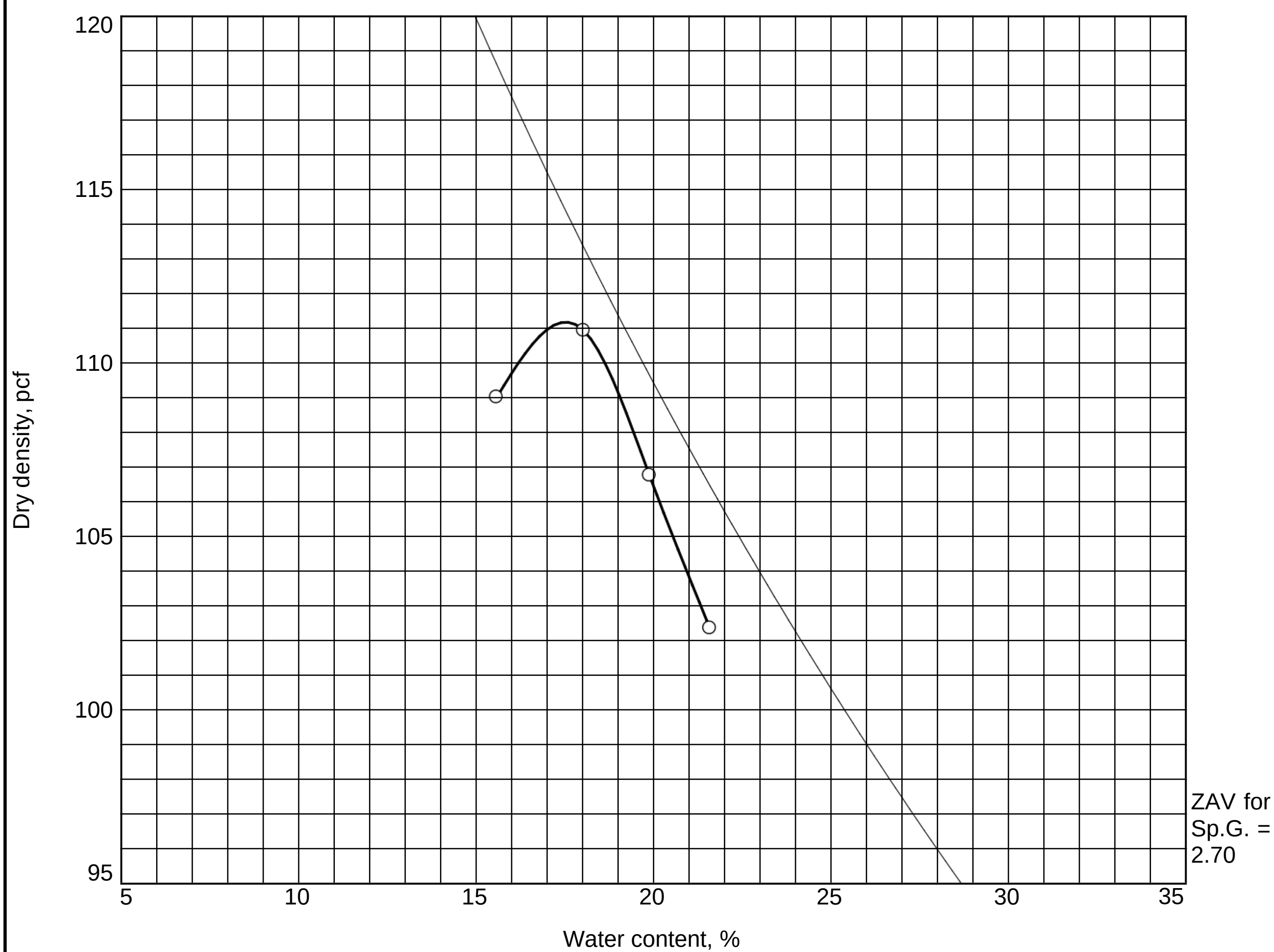
Date Drilled: 8/23/17
Driller: B. Maxson

Elevation	Depth	Description of Materials (Classification)	* Sample Blows	Sample Depth (feet)	N-Value (blows/ft)	Remarks
		4.5" Asphalt				
	0.4	5" Base Stone	2-3-6	0.5	9	Subsurface water was not encountered immediately upon completion of drilling
	0.8	**POSSIBLE RESIDUUM:** Firm to very stiff, mottled orange and light brown to brown, moist, CLAY (CL)	2-6-10-13	2.0	16	
	4.0	Very stiff, gray to brown to dark brown, moist, CLAY (CL) with shale rock fragments	3-7-19-49	4.0	26	
	6.0	**PARTIALLY WEATHERED ROCK:** Shale rock fragments	50/1	6.0	100+	
	6.5	Auger refusal at 6.5'				

*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.

APPENDIX C

MOISTURE-DENSITY RELATIONSHIP



Test specification: ASTM D 698-12 Method A Standard
ASTM D4718-15 Oversize Corr. Applied to Each Test Point

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > #4	% < No.200
	USCS	AASHTO						
1'-8'	CL	--	21.5	--	37	17	6	72

ROCK CORRECTED TEST RESULTS	UNCORRECTED	MATERIAL DESCRIPTION
Maximum dry density = 111.2 pcf	108.8 pcf	Brown lean CLAY with sand
Optimum moisture = 17.5 %	18.6 %	

Project No. 62V-0200 **Client:** County of Roanoke
Project: Hollins Fire Station Concrete Pavement Replacement

○**Source of Sample:** Boring B-1 **Sample Number:** 126200

Remarks:

September 6, 2017

Assumed sp. gr. of +No.4: 2.7

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Sp. gr. for ZAV is an assumed value.

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California Bearing Ratio (ASTM-D1883)

Record No.: 62V-0201

Test Date: 11-Sep-17

Client: County of Roanoke

Project: Hollins Fire Station Concrete Pavement Replacement

Compaction method: ASTM D698

____X____ **Soaked CBR**

Unsoaked CBR

Stress on Piston (psi)

CBR: penetration @ 0.2 in.	7.5
Swell (%):	0.3%
Dry Density Before Soaking (pcf):	111.7
Dry Density as Percentage of Maximum Dry Density:	100.4%
Percentage of +No. 4 in sample	6
Surcharge Weight (lb):	10

Maximum Dry Density (pcf):	111.2
Optimum Moisture Content (%):	17.5

Visual Description:
Brown CLAY with sand
F&R Lab No.: 126200

Source: Boring B-1, 1'-8'

Moisture Content Before Soaking (%):	17.1%
After Testing, Top Inch	19.2
After Testing, Average	18.1

FROEHLING & ROBERTSON, INC.

By:

SINCE

1881

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