Detailed Report of Geotechnical Exploration

MSV - Phase II
Site Improvements
Winchester, Virginia

Triad Project No. 07-17-0307

Prepared for:

Ms. Dana Hand Evans, Executive Director
Museum of The Shenandoah Valley
901 Amherst Street
Winchester, Virginia 22601

Prepared by:

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April 23, 2018

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Ms. Dana Hand Evans, Executive Director
Museum of the Shenandoah Valley
901 Amherst Street
Winchester, Virginia 22601

RE: Report of Geotechnical Exploration
MSV – Phase II Site Improvements
Winchester, Virginia
Triad Project No. 07-17-0307

Dear Ms. Hand Evans:

Triad Engineering, Inc. (Triad) has completed a detailed geotechnical exploration for the planned Phase II Site Improvements project at the Museum of the Shenandoah Valley (MSV) in Winchester, Virginia. The geotechnical scope of services was completed in substantial conformance with our proposal dated November 16, 20174 and executed by signing and returning the Professional Services Agreement on February 2, 2018. This report includes the results of the field exploration and laboratory testing, and it presents our recommendations related to the geotechnical design and construction elements of the project.

The subsurface exploration was performed to evaluate the subsurface conditions encountered at the planned MSV Phase II Site Improvements project for the limited purposes of preparing design and construction recommendations for geotechnical aspects of the project. It is emphasized that subsurface conditions may vary dramatically between the test pits, and Triad makes no representations as to subsurface conditions other than those encountered at the specific excavation locations.

This report has been prepared for the exclusive use of MSV for specific application to the design of the Phase II Site Improvements project in Winchester, Virginia. Triad's responsibilities and liabilities are limited to our Client and apply only to their use of our report for the purposes described above. To observe compliance with design concepts and specifications, and to facilitate design changes in the event that subsurface conditions differ from those anticipated prior to construction, it is recommended that Triad be retained to provide continuous engineering and testing services during the earthwork and foundation construction phases of the work.

We appreciate the opportunity to provide our services during the design phase of the project. If you should have any questions concerning this report, or if you require any additional information, please do not hesitate to contact us.

Sincerely,

TRIAD ENGINEERING, INC.

Raymond A. Strother II

Raymond A. Strother II, P.E.
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TABLE OF CONTENTS

REPORT OF GEOTECHNICAL EXPLORATION

MSV PHASE II SITE IMPROVEMENTS
WINCHESTER, VIRGINIA

TRIAD PROJECT NO. 07-17-0307

FOREWORD

This report has been prepared for the exclusive use of Museum of the Shenandoah Valley (MSV) for specific application to the design of the Phase II Site Improvements project located in Winchester, Virginia. The work has been performed in accordance with generally accepted geotechnical engineering practices. No other warranty, expressed or implied, is made.

This report should not be used for estimation of construction quantities and/or costs, and contractors should conduct their own exploration of site conditions for these purposes. Please note that Triad is not responsible for any claims, damages or liability associated with any other party's interpretation of the data or re-use of these data or engineering analyses without the express written authorization of Triad. Additionally, this report must be read in its entirety. Individual sections of this report may cause the reader to draw incorrect conclusions if considered in isolation from each other.

The conclusions and recommendations contained in this report are based, in part, upon our field observations and data obtained from the test pits at the site. The nature and extent of variations may not become evident until construction. If variations then appear evident, it may be necessary to re-evaluate the recommendations presented herein. Similarly, in the event that any changes in the nature, design, or location of the facilities are planned, the conclusions and recommendations contained herein shall not be considered valid unless the changes are reviewed and the conclusions are modified or verified in writing by Triad. If we are not afforded the privilege of making this review, we will not assume responsibility for misinterpretation of our recommendations, as our recommendations are strictly limited to conditions represented to Triad at the time this report was issued.

SITE DESCRIPTION

The site planned for the MSV Phase II Site Improvements project is located at 901 Amherst Street at the existing Museum of the Shenandoah Valley complex in Winchester, Virginia. The approximate site location is illustrated on Figure No. A-1 located in Appendix A.

The site planned for construction consists of gently to steeply sloping, grass covered and densely wooded terrain. The southern and majority of the northern and western portions of the planned project consist of open pasture or manicured grass and parking areas.

The southern and eastern portions of the roadway and trail alignments include dense mature growth forest with thick to sparse underbrush. Exposed rock outcrops were observed throughout the planned project limits. Underground gas, sewer, water, electric and telephone lines are located throughout the project limits.

PROJECT DESCRIPTION

Based on our review of the provided plans, we understand that the site improvements will include over 5,000 lineal feet of new access roads, several parking areas, some of which will include permeable pavers, two (2) bio-retention basins and a new entrance located along the property boundary adjacent to James Wood Middle School. Several structures are also planned, and they will include a new ticket kiosk, bathrooms and open pavilions at the new entrance, a circular observatory, new amphitheater structure with new seating and grass viewing areas and a future office building near the existing metal storage building. We are not aware of any retaining or below grade walls for this project. However, there is potential for some small retaining walls in parts of the site. Based on the provided profiles, it appears that maximum cuts and fills may be on the order of 10 feet or less.

GEOLOGIC SETTING

General

The project is located within the Valley and Ridge Geologic Province. Based on review of the Geologic Map of Frederick County, Virginia, the project site is situated within sedimentary bedrock units belonging to the Conococheague Formation of Cambrian Age. This formation is generally described as algal limestone with interbedded aphanitic limestone and dolomite. Siliceous and dolomitic laminations are common, and minor sandy beds are present throughout the formation. The site is located just east of a thrust fault between the Conococheague Formation and the Elbrook Formation. Residual overburden soils weathered from the parent bedrock generally consist of medium to high plasticity silty clay with varying amounts of sand and rock fragments.

Development in Karst Areas

Karst terrain is characterized by caves, internal drainage, lack of surface streams, and topographic features such as sinkholes. These features are the result of the dissolution of soluble bedrock, such as limestone or dolomite, by groundwater. As groundwater enters fractures and bedding planes in soluble carbonate bedrock, it slowly dissolves the rock and enlarges the fractures. This results in the formation of solution channels or underground streams or ravines.

The carbonate rock at the site is moderately solution-prone, highly calcareous and weathers differentially to produce a pinnacled or "sawtooth" top of rock profile. The degree of weathering or dissolution within the limestone bedrock is controlled by joint orientation and frequency and to some degree bedrock structure. Where joints intersect or where rock is highly fractured, dissolution is intensified typically creating topographically low areas and weathered rock seams that are generally filled with residual clay soils. Conversely, topographically high areas generally represent more competent, slightly to non-weathered rock that is often coarse grained and only slightly solution prone.

Sinkholes are created by the subsidence of unconsolidated materials (soils) into underlying voids such as solution channels or caves. Usually, subsidence occurs slowly and steadily over geologic time. Many sinkholes, however, are caused by a sudden collapse of a solution cave when the roof of the cave becomes too thin to support the overburden materials. Sinkholes recently created by such a collapse can usually be identified by the presence of freshly broken rock outcrops around the rim or throat of the sinkhole.

Based on our limited site reconnaissance due to areas of heavy vegetative cover in portions of the site, and review of available topographic mapping of the project site, we did not observe any surface features suggesting the presence of any active or incipient sinkhole activity at this or other locations along the roadway alignment. It is important to note, however, that sinkholes and other types of solution features are very common in the immediate geographic area, and there are certain risks that an owner must accept when developing in these karst areas. These risks can include groundwater contamination, subsidence and flooding. In all these instances, water is the primary cause of the problem. The levels of these risks cannot be clearly or completely defined since they are partially controlled by nature. A geophysical study can be conducted to help better define the level of risk associated with potential future sinkhole activity in and around the closed depression or along the roadway alignment, if desired.

It is important to note that alterations in the ground surface, particularly in cut areas, during construction can impact the natural drainage within the site, and it is common to have some solutioning features develop in these areas as a result of construction. Based on the planned cuts and fills and results of the test pits, we anticipate that blasting will be necessary to achieve design grades for this project. Any blasting required to remove hard rock can create micro-fractures within the bedrock that will allow greater surface water infiltration into areas that may normally not receive water and, in turn, disturb old solutioning features and/or possibly create new features. These features can develop during and/or after construction and they will result in some minor construction delays and unanticipated costs for repairs. Certain design and construction measures can and should be implemented to help reduce potential risks associated with future sinkhole development within the site. All of these suggested measures are associated with implementing proper site drainage, minimizing water infiltration, and reducing groundwater fluctuation during and after construction. These additional measures include the following:

- Positive slopes should be maintained away from pavement areas during and subsequent to construction.

- All storm water should be collected into inlets and ditches and routed to discharge points away from the roadway and into stormwater management (SWM) areas.

- Maintain positive slopes around excavations and the roadway footprints prior to and after construction.

- Due to significant increased infiltration into new SWM areas, it is common to have new solutioning features develop within these areas during and/or after construction. We recommend that any SWM areas planned for the project be lined with a synthetic geomembrane. The use of the geomembrane will effectively arrest infiltration within the basin. As such, the risk of potential development of sinkholes within the basin would be very low. If this measure is deemed cost-prohibitive, we recommend that any SWM areas be lined with a minimum one (1) foot thick layer of low-permeability clay. Although this measure will slow the infiltration rate of water in the basin, it will not be eliminated. Consequently, use of a low permeable clay layer may result in continual maintenance issues associated with remediation of sinkholes, if they develop. Additionally, we recommend that serious consideration be given to conducting electrical resistivity within any planned SMW areas to evaluate them for underlying karst related features that may impact the planned construction or groundwater.

FIELD EXPLORATION

The field exploration for the project included excavating fifty-three (53) test pits along the planned roadway alignment and at the planned structure locations. The approximate test pit locations are shown on Figure Nos. A-2 and A-3 in Appendix A. The test pit locations were selected by Triad and Painter-Lewis and were established in the field by Painter-Lewis, P.L.C. Due to thick vegetation in the southern and eastern parts of the project, Triad mechanically cleared along strategic areas with the backhoe to gain access into the densely wooded parts of the site.

The test pits included Dynamic Cone Penetrometer (DCP) testing at select intervals to in the majority of the test pit locations. Hard rock, as indicated by bucket refusal on hard rock was encountered in the majority of the test pits at depths ranging from 1 to 9 feet below planned grades. Test pits TP-7, TP-10 to TP-13, TP-16, TP-21, TP-23 to TP-27, TP-29, TP-30, TP-33, TP-34, TP-35, TP-39, TP-46, TP-48 to TP-51 and TP-53 were extended to the planned termination depths ranging from 3 to 10 feet below existing grades without encountering hard rock. Please note that visible rock outcrops were observed throughout the entire project limits during the field exploration. A geotechnical engineer from our office was present full time during the excavation work to direct the operator, log all recovered soil samples and observe groundwater and rock conditions. The recovered soil samples were transported to our laboratory for further testing. Detailed descriptions of materials encountered in the test pits are contained on the Test Pit Summary Sheets in Appendix B. Figure No. 1 in Appendix B contains a description of the classification system and terminology utilized.

SUBSURFACE CONDITIONS

The materials encountered in the test pits are generally described below. Stratification lines indicated on the Test Pit Summary Sheets represent the approximate boundaries between material types, and the actual transitions may be gradual.

Surface Materials: Approximately 3 to 24 inches of topsoil was encountered at the surface of the majority of the test pits. The topsoil generally consisted of tan-brown to red-brown organic silt with some clay and an appreciable surface root mat. The majority of the topsoil ranged in thickness from 4 to 12 inches. However, thicknesses may be greater in lower lying areas and adjacent to large root balls in the forested areas. Approximately 10 to 12 inches of surface gravel associated with the existing parking areas was encountered at test pitsTP-44 and TP-45.

Old Fill: Old fill was encountered below the surficial layers in test pits TP-34, TP-35 and TP-44 through TP-49, and it was present to approximate depths of 2 feet and 7.5 feet, respectively. In general, the fill consisted of tan-brown and brown shot rock and clay with varying amounts of sand, gravel and cobbles. DCP values obtained in the fill, where applicable, ranged from 2 to 14 blows per increment indicating soft consistencies to loose relative densities. The majority of the blow counts within the fill exhibited medium stiff to stiff consistencies. DCP testing was not performed in some locations due to the high rock content of the fill.

Residual Soils: Residual soil was encountered below the surface layer or old fill in all the excavations. The residuum was present to the termination depths or bucket refusal on hard rock. In general, the residual soils consisted of orange-brown to tan-brown, low to medium plasticity silty clays or clayey silts with varying amounts of sand and rock fragments. DCP values obtained in the residuum ranged from 2 to greater than 50 blows per foot indicating a very soft consistency to very dense relative density. The DCP values in excess of 50 blows per foot were associated with samples obtained at the top of hard rock. The majority of the DCP values within the residuum exhibited medium stiff to stiff consistencies. The DCP values of less than 3 blows per increment were generally obtained near the ground surface or at the initial test increment which is located within the material disturbed during the excavation process.

Groundwater Observations: The test pits were checked for the presence of groundwater both during and upon completion of the excavations. We did not detect an apparent static groundwater level in any of the excavations. It is important to note that fluctuations in perched water and groundwater levels may occur due to variations in environmental conditions, surface drainage and other factors which may not have been evident at the time measurements were made and reported herein.

LABORATORY TESTING

Laboratory tests were performed to supplement the field classifications, assess potential volume change characteristics and establish geotechnical design criteria. All laboratory tests were completed in accordance with appropriate ASTM and AASHTO standard test methods. Detailed results of the laboratory tests are contained in Appendix C. A summary of the test results is presented below.

TEST TYPE	TEST RESULTS
Natural Moisture Contents	7.2 to 32.6 %
Atterberg Limits: Liquid Limit Plasticity Index	35 to 54 7 to 32
Percent Passing No. 200 Sieve	46 to 94 %
USCS Soil Classification	CH, GC-GM, CL (7 Samples)
Standard Proctor Maximum Dry Density Optimum Moisture Content	 95.5 to 110.5 pcf 15.0 to 23.5 %
California Bearing Ratio	5.7, 5.9, 6.5 and 8.3 %

DISCUSSION

General

The results of the exploration indicated that the site is generally underlain by residual soil. However, old fill was encountered in test pits TP-34, TP-35 and TP-44 to TP-49. Based on previous experience at the site and results of the recent excavations, the area in and around test pit TP-34 which is east of the maintenance facility contains old fill with a high percentage of large boulders and debris. The soil types and consistencies of the residual soils are generally within typical ranges for limestone derived soils within the general geographic area. These soils typically do not exhibit adverse shrink-swell characteristics provided that compaction criteria and proper construction drainage measures, grading and sloping are maintained both during and after construction. We anticipate that some of the soils obtained from cut areas will require drying on the order of 5 to 15% to attain an appropriate moisture level for compaction. Weathering of the parent bedrock has produced a very irregular or pinnacled top of rock profile as evidenced by the visible rock outcrops throughout the property and irregular bucket refusal depths observed during the exploration.

Preventative construction measures which can be employed to help reduce risks of subsidence as a result of solutioning activity were discussed previously in this report. It is emphasized that sinkholes and/or solutioning features can develop as a result of blasting and alterations of surface drainage patterns, particularly in cut areas. Specific corrective measures for repair of any such solutioning features which may develop during construction can be better addressed at the time of construction based on the specific condition of the feature. Therefore, contingency costs should be included in the construction budget to repair sinkholes which may develop during construction.

Utility trenches tend to serve as conduits to accumulate subsurface water and allow flow along the trench area, particularly through the bedding stone. This condition increases the risk of potential future solutioning activity beneath and/or around the utility lines especially if blasting is performed to achieve invert elevations. All underground water lines should be pressure tested prior to backfilling to verify that no leakage is present. Additionally, we recommend using VDOT 21B for pipe bedding in lieu of ASTM #57 stone to reduce the potential of accumulation of subsurface water within the bedding layer of the utility trenches.

We anticipate that the material from the on-site excavations will be utilized as a borrow source for the project. Based on the variable top of rock profile at the site, we also anticipate that some of the new fill material may consist of a "shot" (blasted) rock and soil mixture. For construction of utility trenches through any fill zones containing the "shot" rock and soil mixture, it is important to note that some of the shot rock may not be suitable for re-use as backfill due to the size of the rock and the type of compaction equipment (light) typically utilized.

Rock removal techniques will be required for effective removal of hard rock such as large boulders, ledges or massive bedrock. In addition, excavated rock may require additional down-sizing with a hoe-ram in order to achieve a satisfactory size for incorporation into the fill. Hard rock was encountered at shallow depths in the planned underground SWM storage basin. Therefore, blasting will be required to achieve final design grades in this area. Additionally, infiltration of stored water should not be utilized in the design calculations of the underground stormwater management area.

Old Fill

Although the old fill was found to be generally in a firm and stable condition, there will remain the possibility that some of the old fill may not be suitable for support of the new construction. Therefore, we recommend that all areas planned for new construction be heavily proofrolled with approved construction equipment such as a fully loaded, tandem axle dump truck prior to fill placement or construction. If unsuitable fill is present within or extending 5 feet beyond the pavement areas, it should be removed and replaced with new controlled fill. The final depth and extent to which the fill should be removed should be based on conditions observed during construction. Detailed recommendations for design and construction are presented in subsequent sections of this report.

RECOMMENDATIONS FOR DESIGN

Foundations

Based on the test pit and DCP results and our understanding of the project, we believe that the proposed structures can be supported on conventional spread foundations bearing on new controlled fill or in-situ soils at shallow depths. Based on the data obtained from the test pits and the available floor elevations, we recommend that a maximum allowable bearing pressure of 2,000 psf be utilized for design of spread footings for the new ticket kiosk, restroom structures, and maintenance facility. We recommend a maximum allowable bearing pressure of 3,000 psf for the planned tower, 2,500 psf for the western pedestrian bridge abutment at test pit TP-53 and 3,500 psf for the eastern pedestrian bridge abutment at test pit TP-54. Minimum dimensions of 2 feet and 3 feet should be observed for continuous and isolated footings, respectively. We anticipate that hard rock will be encountered within some foundation excavations (possible tower foundation and eastern pedestrian bridge near test pit TP-54). Foundations should not bear on a combination of rock and soil conditions to reduce the risk of intolerable differential settlements. As such, any rock encountered at or above the planned foundation bearing level should be over-excavated a minimum of 12 inches below the footing bearing level and should be replaced with new controlled fill. Exterior foundations should bear at least 30 inches below the final outside grade for frost protection. Footings within permanently heated areas can bear at minimum depths below the finished floor.

Based on the above design recommendations and the various types and consistencies of bearing materials, it is estimated that total settlements for foundations bearing on new controlled fill or approved residuum will be on the order of one (1) inch or less. Differential settlements are anticipated to be half of the total settlements or in this case one-half (½) inch. Differential settlements along continuous wall footings are not expected to exceed an angular distortion of 0.0015 inch/inch.

The project site is located in Winchester, Virginia which is considered to be a low seismic risk region. We recommend that site class "D" be utilized for seismic design of foundations. This recommendation is for the designer utilizing the International Building Code (IBC) 2015 guidelines. Liquefaction potential of the on-site soils is considered to be negligible.

Floor Slabs

The structures will include concrete slabs supported on grade and bearing on new controlled fill and/or approved in-situ materials. We recommend that a modulus of subgrade reaction, "k," of 100 pci be adopted for analysis and design of the slabs-on-grade. Slabs should be underlain by a minimum 4-inch thick layer of open-graded aggregate such as ASTM designation No. 57 stone. The slabs should also be underlain by a conventional polyethylene vapor barrier.

Pavement Design

California Bearing Ratio (CBR) tests were performed in accordance with ASTM D 1883, for bulk samples S-1 through S-4. All of the CBR samples were obtained within five (5) feet of the ground surface. Sample S-1 was composited from test pits TP-2 through TP-4, sample S-2 was composited from test pits TP-14 through TP-16, sample S-3 was composited from test pits TP-30 through TP-32 and sample S-4 was composited from test pits TP-46 through TP-51. Based on the laboratory testing of the samples, CBR values of 5.7, 5.9, 6.5 and 8.3 percent were obtained for the specimens compacted to nearly 100 percent of the maximum dry density based on the Standard Proctor Test method (ASTM D 698).

For design and/or evaluation purposes, we recommend that an average CBR value of 4.5 percent be utilized for evaluation of flexible pavement sections. In the event that a higher CBR design value is desired, we recommend that consideration be given to use of a granular off-site borrow source for the upper 2 feet of the pavement subgrade. Alternatively, you may wish to consider using stabilization fabric or geogrids to reduce the required pavement section, if desired. The soil subgrade should be crowned or properly sloped to provide drainage of the base course aggregate. All structural fill placed as pavement subgrade shall conform to the requirements listed in the Controlled Fill section. The pavement design and all asphaltic concrete should be in accordance with the most current VDOT Road and Bridge Specifications.

Drainage ditches, underdrains (UD-1), edge drains (UD-4) and/or inlets should be designed for the pavement areas to maintain drainage and divert runoff away from the pavement subgrade at all times. The drains should either be routed to appropriate stormwater management areas or day-lighted away from the roadway subgrade. Failure to provide adequate drainage could result in a shorter pavement life and failure.

CONSTRUCTION RECOMMENDATIONS

Site Preparation

Initial site clearing and grubbing should include removal of the topsoil, trees, brush and any other deleterious materials within the planned pavement areas and extending five (5) feet beyond their perimeters. After removal of the unsuitable surface materials, the subgrade soils should be heavily proof-rolled with approved construction equipment to locate isolated soft spots or areas of excessive "pumping" which are too wet to accommodate compacted fill or building construction. The exposed subgrade should be examined and verified by a representative from our office prior to placement of compacted fill. Areas of unstable/wet subgrade soils should be either scarified, air-dried to a sufficient moisture content and re-compacted prior to fill placement or excavated to the level of stable soils. If stable soils are not encountered within 2 feet of the planned bottom of the aggregate sub-base layer, we recommend that the soil be over-excavated a maximum of 2 feet and stabilized with a 1-foot thick layer of open graded base (VDOT OGB). The stabilization layer should be compacted until firm, stable conditions are achieved. We recommend that VDOT 21-B dense graded aggregate be utilized to "choke-off" the OGB and return the over-excavated area back to the bottom of the planned sub-base layer. If desired, alternate recommendations such as utilizing geogrid or stabilization fabric to address unstable conditions can be provided at the time of construction.

Excavation Areas

The carbonate bedrock present beneath the site generally weathers differentially to produce a very irregular top of rock profile. Consequently, it is quite impossible to predict where rock will be encountered at locations between specific exploration points. In general, the in-situ soils present can be excavated with conventional earth moving equipment such as backhoes, pans and tracked loaders. Decomposed rock encountered can possibly be removed to a very limited extent with a ripper. This layer, however, is typically thin and the transition from soil to hard rock is somewhat abrupt. Hard bedrock or large boulders will require blasting, hoe-ram chipping or hydraulic splitting for effective removal. Due to the close proximity of the existing structures and utilities, we suggest that blasting be prohibited in portions of the project. In areas where blasting is deemed necessary, we recommend the work be performed by a blasting contractor with at least 5 years of experience, and pre- and post-blast surveys of all existing structures in close proximity of blasting work should be mandatory. Based on the presence of existing rock outcrops observed during the exploration and the underlying karst terrain, hard rock will likely be encountered during site earthwork activities. Accordingly, excavations should be completed on an unclassified basis, where the contractor will assume the risk of determining the percentage of rock in the overall excavations. Contractors should be required to submit unit prices for unanticipated unsuitable materials, should they be encountered during the project.

During excavation operations, dry conditions should be maintained within the cut areas at all times in order to reduce the need for additional undercutting or aeration of soils. The contractor should be prepared to implement, if necessary, temporary de-watering measures in these areas during construction. These measures can include sloping the cut areas to appropriate sump pit(s) and pumping accumulated surface runoff from precipitation. All cut areas should be sealed at the end of each day, to the extent which construction practicality will permit, to help prevent infiltration of precipitation and subsequent unsuitable soil conditions.

Controlled Fill

Satisfactory Soils

On-site materials excavated from cut areas can generally be used for fill provided that compaction criteria are strictly maintained. Based on results of the moisture testing, we anticipate that the on-site materials may have to be dried or wetted on the order of 10 to 15% to attain a moisture content that is within a satisfactory range to obtain proper compaction. ***Due to the fine-grained soils at the site, construction during the winter and spring months is often futile. This will be very dependent upon seasonal conditions at the time of earthwork construction and the amount of care exercised by the contractor. Also, the low to medium plasticity silty clays are relatively sensitive to moisture fluctuations and typically can be effectively placed and compacted only during drier seasons.***

Fill materials should not contain any debris, waste, or frozen materials and they should contain less than two (2) percent vegetation-organic materials by weight. Also, materials classified as OL, OH, or Pt are not suitable for use as structural fill. The on-site clayey soils are generally suitable for re-use as controlled fill provided that proper drainage, grading, and sloping away from the pavement areas are maintained both during and after construction. Excavated limestone rock can be utilized for fill provided that certain construction procedures are observed. These procedures include maintaining the maximum particle size of the rock, prohibiting nesting of boulders, and mixing sufficient amounts of soil fines with the rock to fill in open voids between the rock particles.

Placement and Compaction

In general, all sloping areas upon which fill is to be placed should be benched or "notched" so that a smooth interface between existing ground and new fill will not be present. Each layer of fill should be benched into the existing ground in accordance with the most recent VDOT Road and Bridge Specifications. Controlled fill slopes and embankments should be constructed at 2.5H:1V slopes or flatter.

All fill material should be placed and compacted in accordance with the most recent VDOT Road and Bridge Specifications. Field density tests should be performed in accordance with ASTM D 6938. In areas where larger rock is utilized, the size of the loose lift will be dependent on the quantity of rock within the overall fill matrix, and this can be evaluated at the time of actual placement.

Prior to compaction, the moisture content of each layer should be adjusted, as necessary, to obtain the required moisture content to achieve the specified compaction level. Each layer should be compacted to the required percentage of maximum dry density. Fill should not be placed on surfaces that are muddy or frozen, or have not been approved by testing and/or proof-rolling. Free water should be prevented from appearing on the surface during or subsequent to compaction operations. Soil material which is removed because it is too wet to permit proper compaction can be spread and allowed to dry. Drying can be facilitated by discing or harrowing until the moisture content is reduced to an acceptable level. When the soil is too dry, water should be applied uniformly to the subgrade surface or to the layer to be compacted.

Foundation Construction

We anticipate that conventional earth excavation equipment such as a backhoe or trackhoe can be utilized to excavate the controlled fill for foundation construction. Based on the results of the test pits and the assumed grades, we anticipate that hard rock will be encountered during foundation construction. Any hard rock encountered will require hoe-ram chipping or blasting for effective removal. Due to the close proximity of the existing structures and utilities, we recommend that blasting be prohibited at the site. We recommend that any loose materials present at the bottom of footing excavations as a result of excavation work be re-compacted or completely removed by hand in order to reduce differential settlements.

As mentioned previously, any hard rock that is encountered should be over-excavated to at least 12 inches below bearing levels and should be replaced with on-site soil fill placed in accordance with the recommendations provided herein. This measure typically reduces the magnitude of differential settlement resulting from partial rock and partial soil bearing. Additionally, isolated column footings should be constructed to bear either entirely on rock or entirely on soil, but not on a combination of soil and rock. Areas which contain combined soil and rock bearing should either be cleaned of all soil and backfilled with "lean mix" concrete to achieve uniform rock bearing or should be over-excavated a minimum of 12 inches and backfilled with compacted soil to achieve complete soil bearing.

Foundation concrete should be placed the same day that excavations are completed to reduce the potential for softening due to precipitation and/or runoff. In areas where backfill adjacent to wall construction has not been placed prior to precipitation events, any ponded water that accumulates in these areas should be pumped out immediately to help prevent softening and deterioration of the surrounding soils. In addition, all rough grades around the structure should be sloped away from the structure both during and after construction such that water from precipitation does not build up or pond adjacent to the perimeters.

Any underground utilities which are located below or adjacent to new foundations should be backfilled with approved, well-graded crushed aggregate, lean mix concrete or flowable fill grout to grades which are at or above the design bearing levels. In addition, minimal thicknesses of bedding stone should be utilized beneath the utility lines in order to help prevent significant accumulation of water from precipitation developing within the utility trench area.

Floor Slab Construction

Prior to placement of crushed stone for the floor slabs, the subgrade soil within the limits of the structure should be proof-rolled in order to detect any soft, wet "pumping" areas. Any unsuitable areas should be either scarified, aerated to an approved moisture content, and re-compacted or undercut and replaced with controlled fill. The subgrade should be properly sloped to allow water from precipitation to drain from the stone prior to slab placement. Water should not be allowed to pond within the stone prior to placement of concrete. We recommend that the sequence and timing of floor construction be coordinated such that slab concrete will be placed within a very short time, i.e., within a few days or less, after placement and compaction of the aggregate base course.

Utility Construction

Locations and invert elevations for proposed utilities have not been provided at this time. In general, we anticipate that conventional excavation equipment such as a backhoe or trackhoe can be used for utility excavations in the residual soils and controlled fill. Depending on the final invert elevations, hard rock may be encountered in the utility alignments. Any excavations which encounter hard rock will likely require heavy ripping or hoe-ram chipping with a trackhoe or hydraulic splitting to attain scheduled invert elevations. We recommend that blasting at the site be strictly prohibited to prevent damage to the existing structures and utilities.

In areas where excavated rock and soil fill has been placed during mass earthwork construction, an acceptable substitute backfill material should be used for new utility trench backfill. This is recommended because of the inherent difficulty in re-compacting larger excavated rock materials in trenches using small, hand-operated equipment. The substitute material should comply with the maximum particle size restrictions specified for the particular utility. Trenches below structure and pavement areas should be backfilled in accordance with the Controlled Fill section of this report.

Pavement Construction

Any wet and/or unstable soils present at the subgrade level during grading operations should be either scarified, aerated and re-compacted or should be removed and replaced with suitable fill material. Any unsuitable subgrade soils should be corrected immediately prior to placement of base stone and pavement material. **It will be very important that the final soil subgrade be properly sloped or crowned to help remove surface water from precipitation events from the subgrade area. Also, it will be very important that adequate ditches, underdrains and edge drains be constructed along cut sections to reduce the risk of surface runoff entering the base stone layer.**

It is very important that both the base stone and pavement section be placed immediately after acceptable subgrade conditions have been achieved due to the potential for subgrade softening from adverse weather conditions. In addition, heavy construction traffic should be limited from traveling across approved final subgrade areas that have been exposed to precipitation in order to help maintain a stable subgrade prior to pavement construction. If hard rock is encountered above the design subgrade level in the pavement area, it should be over-excavated to at least the level of the bottom of the pavement section (i.e. the bottom of the aggregate base material). All base stone and asphaltic concrete placement and testing should be performed in accordance with current VDOT specifications.

Construction Observations

We recommend that the geotechnical engineering firm of record, Triad, be retained to observe the construction activities to verify that the field conditions are consistent with the findings of our exploration. Construction observation services should be performed on a full-time and/or intermittent basis, as required, to:

- observe removal of all deleterious materials and observe proof-rolling of original subgrade material prior to initial fill placement.

- observe and test controlled fill construction in accordance with VDOT standards. Field density tests should be performed in accordance with ASTM D 6938 (nuclear method).

- observe and test asphalt placement in accordance with the guidelines provided in the most current VDOT Road and Bridge Specifications. Field density tests should be performed in accordance with ASTM 6938 (nuclear method).