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Dr. Navdeep Sharma
43470 Rockforest Ct
Sterling VA 20166

Reference: Subsurface Exploration & Geotechnical Evaluation report,
Proposed Animal Hospital/Office Building,
1009 Ruritan Circle, Sterling, Loudoun County, Virginia (Project No. 072216)

Dear Dr. Sharma:

As authorized we have completed the subsurface exploration of the above referenced project. The services were provided in accordance with our proposal. The property is located on the east-southeast side of Ruritan Circle and to the west of the existing W&O Asphalt Trail. More specifically the site is located at 1009 Raritan Circle. The approximate location of the property is shown on the Site Vicinity and Site Location Maps provided in the Appendix.

Introduction

We understand that this project consists of design and construction of a two-story Animal Hospital/Office building. The building will be located in the southern half of the site (general area of Borings 3, 4, 7, 7A, 8, and 8A). A copy of the Approximate Boring and Test Pit Location Plan is provided in the Appendix. Based on the information provided to us, the building will measure approximately 50 ft. by 64 ft. in plan dimensions and will have its first floor constructed at EL 299.50. Based on the existing grades provided to us, approximately between 2 to 10 feet of cut is required for establishing the proposed first floor elevation.

The structural loads were not available at the time of preparation of this report and based on our previous experience with similar projects, we have assumed that the maximum column and wall loads will not exceed 300 kips and 4 kips per linear ft., respectively. We have also assumed that the maximum total and differential settlements of 1 inch and 0.75 inches will be tolerable by the structure. Our office shall be informed if the actual loads exceed these assumed values.

This project also include parking area in the northern portion and a storm water management dry basin northeast of the parking area (near Ruritan Circle). The approximate location of the proposed building, parking areas, and the dry basin are shown on the Approximate Boring and Test Pit Location Plan provided in the Appendix.

Scope of Work

The scope of services consisted of site visits by our engineers, performing soil borings, conducting laboratory tests, engineering analysis, and preparing a report outlining our recommendations for the proposed structure. This report is intended for use with regard to the specific project as described herein. Changes in proposed construction, building location, etc., should be brought to our attention so that we may determine any effect on the recommendations given in this report.

Recommendations contained in this report are based on data obtained from the relatively limited number of borings performed at specific locations. This report does not reflect variations, which may occur between the borings. The nature and extent of variations between borings may not become evident until during the construction period. It is essential for successful completion of this project that on-site observations of subgrade conditions are performed during construction to determine if any re-evaluation or additional design recommendations are necessary. It is recommended that this report be made available to project bidders prior to submitting their budget and to the successful contractor and subcontractors for their information only. The opinions outlined in this report are those of the geotechnical engineer based on his/her interpretation of the subsurface conditions, as indicated by the field and laboratory test. The data presented by this report may not be sufficient for the contractor's purposes, and therefore, it may be necessary for the contractors to do their own investigations prior to submitting bids. This report has been prepared for the use of the design professionals for design purposes in accordance with generally accepted geotechnical engineering practices.

No warranties expressed, implied, or made as to the professional advice included in this report. As explained later in the report, inspection is considered necessary to confirm the observed subsurface conditions and to verify that the soils related construction phases are implemented properly. It shall also be noted that Environmental assessments were not requested as part of this service.

Field Exploration Procedures

In order to examine the nature of the subsurface conditions at the site, four (4) auger borings (7, 7A, 8, and 8A) were performed in accordance with ASTM D-1452, Soil Investigations by Auger Borings.

In evaluation of the subsurface condition of the site, we have utilized our previous subsurface exploration study for this site (report dated 8/30/2002). In addition, we also utilized the subsurface exploration report for the adjacent property to the west (report dated 11/7/2006). The boring and test pit information along with associated laboratory tests are included in the Appendix. In addition, the locations of the borings and test pit from the aforementioned two reports are shown on the Approximate Boring and Test Pit Location Plan.

The recent borings were advanced to approximate depths of 3 to 7 feet below the ground surface. The soil samples recovered were visually classified in the field in accordance with the Unified Soil Classification System (U.S.C.S.) and ASTM Specification D-2488. In addition, soil samples were collected in glass jars and subsequently transferred to the laboratory for confirmatory tests. The group symbols for each soil type are indicated in parenthesis following the soil description on the boring logs. It should be noted that soil transitions may be gradual. The Boring Location Plan and Logs are included in this report. The Approximate Boring and

Test Pit Location Plan indicates the approximate physical location of the borings and test pit performed at the site.

During the field exploration, the soils were also investigated for strength utilizing a Dynamic Cone Penetrometer (DCP). The DCP consists of a 15 lb steel ring weight falling 20 inches on an E-Rod slide drive. The drive is tipped with a 45-degree cone point having a projected area of 1.8 inches square. The penetration test is made through an augered hole and the cone point is seated 2 inches into the undisturbed bottom of the hole. The cone point is further driven 1 3/4 inches using the weight hammer. The blows are counted and recorded for each 1 3/4 inch increment. The results are empirically converted to approximate SPT N-values using the Penetration Relationship Curves included in ASTM Publication No. 399.

The soil samples from the previous borings and test pit have been discarded. The soil samples from the recent borings will be retained in our office for a period of 30 days after which they will be discarded unless other instructions are received as to their disposition.

Site Description

As discussed earlier, the site is located at 1009 Ruritan Circle. There are existing scattered trees along the northern and western portion of the site, and the southern portion is relatively open. The ground surface is covered with brush and undergrowth and generally slopes downward in a northwesterly direction. According to the topographical plan provided to us, the existing grades at the site are between 296± near the northwest corner and EL 310± near the southwest corner. The plan indicates that the existing grades in the building area are between EL 308± and EL 302±.

Our review of the report dated 8/30/2002 and the observations made during the recent field work indicated that the house, sheds, and trailer referenced in the aforementioned report have been removed. In addition, it appears that some fill have been placed and the south-southwestern portion of the site have been graded. The topographical plan provided to us confirms the aforementioned findings. However, considering the proposed final grades, the existing fill/possible fill in the building area will be removed during excavation necessary for establishing the first floor elevation. As discussed later in this report special attention shall be given during excavation to ensure that all structural elements associated with the previous structures as well as any unsuitable materials are completely removed and natural soils/rock are exposed. Please note that observation and evaluation of the exposed subgrade condition by the geotechnical engineer of record shall be considered a necessity.

Area Geology and Soil Mapping

The site is located in the Triassic Lowland, Culpeper Basin, of the Piedmont Physiographic Province of Virginia. The soils in this region are residual materials that have developed from the in place weathering of the underlying bedrock. The Diabase rock intrusions are common in this vicinity, and were indicated in the borings. The diabase materials usually weather to an expansive clay near the surface, which transitions into a fine to coarse sand, and ultimately into unweathered bedrock. The boring results confirmed the presence of expansive clay.

In general, the natural soils which have resulted from the in place physical and chemical weathering of the bedrock are composed primarily of residual clayey or silty soils with minor amounts of fine sand. The granular nature of the residual soils generally increases with depth, as does the percentage of rock fragments. These layers are termed decomposed/weathered rock due to their rocklike structure but exhibit characteristics, which qualify them as soil. The decomposed rock strata often abruptly transitions into relatively unweathered rock.

According to Loudoun County Soil Map, the northern portion of the site is mapped as Kelly Sycoline Complex (62B) and the southern portion is mapped as Haymarket and Jackland Soils (68B). Both of this soil types are typically associated with high shrink swell clays, shallow rock and perched water conditions, which were observed in the borings. Copies of the USDA-NRCS unit descriptions for the 62B and 68B soils are included in the Appendix. We have also included engineering properties assigned to 62B and 68B Soils by USDA-NRCS in the Appendix.

The bedrock surface in this area is erratic in vertical depth over very short horizontal distances. This condition is due to various weathering forces and processes. Pinnacles and peaks of relatively unweathered bedrock are common in this area, making accurate determination of the bedrock surface difficult.

Boring Results

At the boring locations approximately 3 to 6 inches of topsoil was encountered. However, we recommend that an average topsoil thickness of approximately 12 inches be assumed for site stripping estimation purposes, due to the extra volume of soil and topsoil that will be encountered when tree root bulbs are removed.

The observed materials below the topsoil layer in the recent borings (7, 7A, 8, and 8A) were visually classified as existing fill materials based on their general appearances. The existing fill materials in borings 7A and 8A extended to approximate depths of 4 and 3 feet below the existing ground surface, respectively. In borings 7 and 8, the existing fill materials caused refusal at the boring termination depths, 3± and 2.5± feet, respectively. Copies of the logs are provided in the Appendix.

The observed existing fill materials in the borings generally consisted of Silt and Clay mixtures with some rock fragments and gravel. It shall be noted that existing fill at greater depths or at other site areas could be encountered. The materials below the existing fill in borings 7A and 8A were classified as possible fill/disturbed natural soils and extended to approximate depths of 6± and 5.5± feet, respectively.

Considering that the engineering characteristics of the existing fill materials, including (i.e. bearing capacity, settlement) could be highly variable, it is our professional opinion that the existing fill materials are not suitable for supporting any settlement sensitive structures (such the proposed building). However, considering the proposed final grades, the existing fill/possible fill will be removed during excavation necessary for establishing the first floor elevation of the building.

Based on the results of our subsurface investigation, the soils below the existing fill/possible fill materials appear to be generally consistent with the geologic data for the region. The presented stratification of the different layers is based on our visual observations, laboratory analysis, and geologic origin. The onsite natural soils can be generally summarized as follows based on the results of the recent borings, as well as the 2002 and 2006 borings and test pit:

1. *Medium to High plasticity CLAY*- This stratum consists of brown and gray Clay (CL-CH) with varying amounts of silt and fine sand. This stratum was encountered below the existing fill/possible fill, and extended to depths ranging between 1.5 and 7 feet below the ground surface.

2. *Clayey Sand/Clayey Gravel*- This stratum underlies the CL-CH stratum (as explained above), and generally consists of brown clayey Sand (SC) or Clayey Gravel (GC) with varying amounts of weathered rock fragments. This stratum extended from the bottom of the CL-CH stratum with approximate thicknesses of 6 to 18 inches. This stratum was not identified in the recent borings.

3. *Weathered Rock*- This stratum was encountered below the SC/GC stratum (as explained above), and caused refusal at approximate depths of 1.5 to 7 feet. The weathered rock, which was encountered, consisted of weathered diabase rock. As a result of the underlying rock, refusal was encountered in all test borings. Therefore, some excavation difficulties should be expected during earthwork operations, and/or site utility work, especially where the inverts lie below the top of the fresh/hard bedrock surface. Depending on the invert elevation, the use of rock excavation equipment (such as hoe ram) may be required during excavation and/or installation of deep utility line.

As discussed earlier, the soils in this area are underlain by weathered and ultimately unweathered bedrock. The bedrock surface in this area is erratic in vertical depth over very short horizontal distances. This condition is due to various weathering forces and processes. Pinnacles and peaks of relatively unweathered bedrock are common in this area, making accurate determination of the bedrock surface difficult. In addition, large floating boulders are also possible in this geology. Therefore construction inspections by an experienced geotechnical engineer shall be considered a necessity.

Laboratory Results

Several representative soil samples were tested in the laboratory for index property analysis and moisture content determination. The recovered soil samples were visually classified by one of our geotechnical engineers. Limits, gradation, and moisture content tests were also performed for selected soil samples. The laboratory test results are provided in the Appendix.

Groundwater

Observations for groundwater were made during sampling and upon completion of the augering operations at each location. Groundwater was not observed in the borings. It should be noted however, that groundwater is significantly influenced by surface water runoff and rainfall, especially during high precipitation seasons.

Variations in the location of the long-term water table may occur as a result of changes in precipitation, evaporation, surface water runoff, and other factors not immediately apparent at the time of this exploration.

It should be noted that perched water conditions are possible and excavations performed at this site, may encounter water. Usually, the groundwater flow can be controlled through trenching operations. The trenching operations should be aggressively undertaken to actively intercept the water flowing on the uphill side of the property. Therefore, this flow can usually be channeled down to existing drainage swales to minimize groundwater problems.

Where it is not possible to dewater individual excavations using trench techniques, some pit and pumping operations will be required. These pumping operations should be aggressively undertaken in order to remove as much water from the excavation as possible, and to minimize standing water. Therefore, significant reduction of construction difficulties can be achieved through aggressive dewatering of the site, and through the minimization of standing water.

It shall be noted that rainfall that enters the site begins to percolate through the moderate permeable, surficial soils. Once the infiltrated water reaches the interface between the soil layer and the very dense decomposed rock, which is virtually impermeable, it begins to flow at the intersection of the rock and the soil. This groundwater flow continues downhill where it will ultimately intersect with the phreatic watertable; however, it occasionally surfaces to form as wet springs. Groundwater conditions associated with this site are typically related to rainfall. Therefore, the groundwater conditions at this site are expected to be significantly influenced by surface water runoff and rainfall, especially during high precipitation seasons. Excavations performed at this site, may encounter water at the interface of the rock and the soil. Usually, the groundwater flow can be controlled through trenching operations.

Engineering Analysis & Recommendations

Based on the boring results and provided that the recommendations as outlined in this report are followed, the proposed structure may be supported on shallow spread footings supported on firm natural soils. Based on the anticipated structural loading and the boring results, we recommend a net allowable soil bearing pressure of 3,000 psf be used to size the footings bearing on suitable firm natural soils and rock. The footings may also be supported on engineered fill constructed over firm natural soils. Such footings shall be sized with a soil bearing pressure of 3,000 psf. Under no circumstances should the footings be supported on or over any existing fill materials.

The floor slab may also be ground supported on firm natural soils or on engineered fill constructed over firm natural soils. The following sections provide more detailed geotechnical recommendations for the proposed development. We recommend that construction be observed by the geotechnical engineer or record to verify that the geotechnical recommendations as outlined in this report are being followed.

As discussed earlier, the weathered rock, which was encountered, consisted of weathered diabase rock. As a result of the underlying rock, refusal was encountered in all test borings. Therefore, some excavation difficulties should be expected during earthwork operations, and/or site utility work, especially where the inverts lie below the top of the fresh/hard bedrock surface. Depending on the invert elevation, the use of rock excavation equipment (such as hoe ram) may be required during excavation and/or installation of deep utility line. It shall be noted that the bedrock surface in this area is erratic in vertical depth over very short horizontal distances. This condition is due to various weathering forces and processes. Pinnacles and peaks of relatively unweathered bedrock are common in this area, making accurate determination of the bedrock surface difficult. In addition, large floating boulders are also possible in this geology. Therefore construction inspections by an experienced geotechnical engineer shall be considered a necessity.

A. Earthwork

The proposed building and pavement areas shall be stripped of all vegetation, rootmat, topsoil, any existing fill/backfill and any other soft or unsuitable material. We recommend the earthwork clearing be extended a minimum of 10 feet beyond the building and 5 feet beyond the pavement limits. Special attention shall be given in the earthwork operations to ensure that all existing fill are completely removed. As discussed earlier, the existing fill materials are not suitable for supporting any settlement sensitive structures (such the proposed building). However, considering the proposed final grades, the existing fill/possible fill will be removed during excavation necessary for establishing the first floor elevation of the building. Verification of complete removal of the existing fill materials by the geotechnical engineer of record shall be considered a necessity for successful completion of the project.

The stripped surface should be observed and evaluated by an authorized representative of the Geotechnical Engineer of Record by utilizing a Dynamic Cone Penetrometer and a hand auger. Based on the exposed subgrade condition, the geotechnical engineer may require proofrolling of the subgrade to assist in its evaluation. Proofrolling shall be done using a fully loaded dump truck with a minimum axle weight of 20 tons or equivalent. Any soft or unsuitable materials encountered during this evaluation should be removed and replaced with an approved backfill compacted to the criteria given in Section B.

Please note that perched water conditions are possible during the earthwork operations, and the contractor shall be prepared to handle the perched water conditions with an acceptable method. All excavations must be in accordance with OSHA and VOSHA safety regulations.

The surficial soils contain fines, which are moisture sensitive and are considered moderately erodible. The Contractor shall provide and maintain good site drainage during earthwork operations to help maintain the integrity of the surficial soils.

The stripping operations shall be observed on a full-time basis by the geotechnical engineer of record to verify that the unsuitable materials are removed.

High plasticity Clay (CH) soils were observed in the borings. The CH soils have high shrink/swell potential and severe restrictions with regard to their use as engineered fill. The following recommendations are provided if CH soils are encountered. In footing areas, the CH soils shall be undercut to a minimum depth of 6 feet below final exterior grades, or to the depth of CH layer, whichever is less. The over excavated area shall be backfilled with footing concrete or “Lean Concrete”. In the pavement and slab areas, the CH soils shall be undercut to a minimum depth of 2 feet below subgrade elevations. The purpose of this encapsulation/undercut and minimum embedment is to place the footings at an elevation below the normal seasonal moisture change (since the moisture content will be stabilized, then shrinking and swelling will be minimized/not occur). The CH soils are not suitable for use as engineered fill.

The rock materials can generally be excavated in mass grading operations where single tooth rippers are used to a depth of approximately 2 feet below the refusal depths. Below this depth, blasting and/or hoe ramming are normally required. In trench excavations, the maximum depth of excavation with conventional excavation equipment is normally to the refusal depths, although "power shovels" can sometimes advance to greater depth, though with difficulty. If blasting is utilized for rock excavations, all disturbed materials must be removed and replaced with lean concrete or engineered fill compacted to the requirements of this report.

The contractor is solely responsible for designing and constructing stable, temporary excavations and shall shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. All excavations shall comply with applicable local, state, and federal safety regulations including the current OSHA and VOSHA Excavation and Trench Safety Standards. Construction site safety generally is the sole responsibility of the contractor for the means, methods, and sequencing of construction operations. We are providing this information solely as a service to our client. Under no circumstances shall the information provided herein be interpreted to mean that our office is assuming responsibility for construction site safety or the contractor's activities; such responsibility is not being implied and shall not be inferred.

In no case shall slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in local, state, and federal safety regulations. Specifically, the current OSHA and VOSHA Health and Safety Standards for Excavations, 29 CFR Part 1926 shall be followed. It is our understanding that these regulations are being strictly enforced and if they are not closely followed, the owner and the contractor could be liable for substantial penalties.

Materials removed from the excavation shall not be stockpiled immediately adjacent to the excavation, in as much as this load may cause a sudden collapse of the embankment. A slope stability analysis shall be performed to determine the factor of safety for cut or fill slopes. The contractor's "responsible person" shall establish a minimum lateral distance from the crest of the slope for all vehicles and spoil piles. Likewise, the contractor's "responsible person" shall establish protective measures for exposed slope faces.

B. Fill Placement and Compaction Requirements

Fill materials shall consist of an approved material, free of organic matter and debris, rocks greater than 3-inches, and have a Liquid Limit and Plasticity Index less that 45 and 20, respectively. The fill shall also have a maximum dry density of 105 pcf or higher. Unacceptable fill materials include topsoil, organic materials (OH, OL) and high plasticity silts and clays (CH, MH).

It shall be noted that the on-site soils are moisture sensitive and most likely will require moisture content adjustments, such as the application of discing or other drying techniques or spraying of water to the soils, prior to their use as controlled fill materials. The planning of earthwork operations shall account for these efforts and increased costs.

It shall also be noted that in a dry and undisturbed state, the majority of the soil at the site shall provide good subgrade support for fill placement and construction operations. When wet, this soil will degrade quickly with disturbance from contractor operations. Therefore, good site drainage shall be maintained during earthwork operations, which would help maintain the integrity of the soil.

Fill materials shall only be placed over a subgrade, which has been evaluated and approved by the geotechnical engineer. Fill materials shall be placed in lifts not exceeding 8-inches in loose thickness and moisture conditioned to within ±3% of the optimum moisture content. Soil bridging lifts within the expanded building limits shall not be used since excessive settlement of the structures will likely occur.

It is recommended that the fill soils be compacted to a minimum of 95% of the maximum dry density obtained in accordance with ASTM Specification D-698, Standard Proctor Method. The upper one-foot of soils supporting slabs-on-grade, pavements and sidewalks shall be compacted to a minimum of 100% of the aforementioned maximum dry density. The expanded footprint of the proposed building and pavement areas shall be well defined, including the limits of the fill zones at the time of fill placement. Grade control shall be maintained throughout the fill placement operations. Granular soils (Unified Soil Classification System SM or better) shall be compacted with a smooth drum vibratory roller or rubber-tired compactor. Cohesive soils shall be compacted with a sheepsfoot roller.

All areas receiving fill shall be graded to facilitate positive drainage of any free water associated with precipitation and surface runoff. Fill materials shall not be placed on frozen soils. All frozen soils shall be removed prior to continuation of fill operations. Borrow fill materials shall not contain frozen materials at the time of placement. All frost-heaved soils shall be removed prior to placement of fill, stone, concrete, or asphalt.

Temporary slopes constructed of on site native silty or clayey soils shall be limited to a maximum gradient of approximately 1.5H: 1 V, and maintained for not more than about 45 days. The temporary slopes shall also be thoroughly vegetated to help minimize erosion of the surficial soils. Permanent slopes constructed of native soils shall generally be flatter than 3H:lV.

The groundwater conditions at this site are highly variable, depending upon precipitation, the effects of grading operations, and rock fracture patterns. The surface of the site shall be kept properly graded in order to enhance drainage of the surface water away from the proposed building areas during the construction phase.

All fill operations shall be observed on a full-time basis by an experienced geotechnical engineer to determine that minimum compaction requirements are being met.

C. Building Foundation

If the recommendations outlined in this report are followed, the proposed building can be supported on a system of shallow foundations consisting of spread and/or continuous footings. Based on the anticipated structural loading and the boring results, we recommend a net allowable bearing pressure of 3,000 psf be used to size the footings bearing on suitable firm natural soils.

The footings can also be supported on engineered fill constructed over firm natural soils. Such footings shall be sized with a soil bearing pressure of 3,000 psf. Firm natural soils are those, which are tested, evaluated and approved by a geotechnical engineer. Under no circumstances shall the footings be supported on or over any existing fill materials.

To reduce the possibility of foundation bearing failure and excessive settlement due to local shear or "punching" action, we recommend that continuous footings have a minimum width of 2.5 feet and that isolated column footings have a minimum lateral dimension of 3.0 feet. Footings shall be placed at a depth of 2.5 feet below finished grade to provide adequate frost cover protection. We also recommend three #5 reinforcements be placed horizontally for all footings, to control or lessen the detrimental effects of differential settlements due to varying subsurface conditions. This shall be evaluated and confirmed by the structural engineer.

Settlement of a structure is a function of the compressibility of the natural soils, the design bearing pressure, column loads, fill depths, and the elevation of the footing with respect to the original ground surface. Provided that all of our recommendations as outlined in this report are followed, it is estimated that the total settlement values of less than 1 inch are expected, and the differential settlement between individual column locations shall be relatively minor, on the order of 75 percent of the maximum total settlement over a 30-foot span.

Concrete for footings shall be placed the same day that excavations are dug, since exposure to the environment may weaken the soils at the footing bearing level. If the bearing soils are softened by surface water intrusion or exposure, the softened soils must be removed from the foundation excavation bottom immediately prior to placement of concrete. If the excavation must remain open overnight, or if rainfall becomes imminent while the bearing soils are exposed, we recommend that a 2-inch thick "mud mat" of "lean" concrete be placed on the bearing soils before the placement of reinforcing steel.

As discussed earlier, if CH soils are encountered in footing areas, the CH soils shall be undercut to a minimum depth of 6 feet below final exterior grades, or to the depth of CH layer, whichever is less. The over excavated area shall be backfilled with footing concrete.

All footing excavations shall be observed by the an authorized representative of the Geotechnical Engineer of Record to confirm the soil bearing pressure, and observe the reinforcement steel, prior to placing footing concrete.

D. Slab-On-Grade

It is recommended that the area of slab-on-grade be prepared in accordance with our recommendations outlined in this report (Sections A and B), which includes stripping and fill placement recommendations. The subgrade soils shall be firm and unyielding. Interior utility trenches shall be properly backfilled and compacted as specified in Section H.

We recommend that the floor slab be isolated from the foundation footings so that differential settlement of the structure will not induce stresses on the floor slab. Also, in order to minimize the crack width of any shrinkage cracks that may develop near the surface of the slab, we recommend mesh reinforcement be included in the design of the floor slab. The mesh shall be in the middle half of the slab. We also recommend the slabs-on-grade be underlain by a minimum of 4 inches of No. 57 stone. This granular layer will facilitate the fine grading of the subgrade and help prevent the rise of water through the floor slab.

Prior to placing the granular material, the floor subgrade soil shall be properly compacted, be free of standing water, mud, and frozen soil and evaluated and approved by the geotechnical engineer or records. The floor slab granular layer shall be hydraulically connected to underground drain tiles to remove any water that may reach the floor slab stone.

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

As discussed earlier, if CH soils are encountered in the slab areas, the CH soils shall be undercut to a minimum depth of 2 feet below subgrade elevations, and replaced with non-expansive soils.

E. Below Grade Walls

The below grade walls should be designed to withstand lateral earth pressures and surcharge loads. For below grade walls that are properly drained, the walls may be designed for an at-rest, equivalent fluid pressure of 65 pounds per cubic foot of wall height (fully drained), and does not include any surcharge loads.

The wall design should also account for any surcharge loads within a 45 degree slope from the base of the wall, and should be based on an at-rest pressure coefficient (ko) of 0.55. Passive soil resistance developed by the embedded base portion of the wall can be assumed to be 300 pcf. A base sliding coefficient of 0.2 (un-factored value) may be used in the wall footing analyses.

The below grade walls shall be thoroughly waterproofed as per County requirements. In addition, we recommend that a layer of Mirafi® G100N Drainage Composite (or approved equivalent) be placed over the waterproofing materials. The drainage composite should be placed in accordance with the manufacturer's recommendations. We also recommend that a combination of exterior draintile, weepholes, and interior drains be provided for the below grade walls. A Below Grade Drain Detail is provided in Appendix of this report.

The backfill against the wall may consist of materials classified as Sandy Silt (ML) or more granular, with a maximum particle size of 3 inches. The liquid limit and plasticity index values for the backfill material shall be less than 40 and 15, respectively. Backfill materials which are placed behind below-grade walls should be free of organic materials and debris, free-draining, non-frost susceptible, and should not include any highly plastic CH or MH materials.

Care shall be exercised during the operation of earthwork equipment in order to avoid damaging the walls. This will be particularly important during the backfilling operations. It is strongly recommended that the walls not be backfilled until the concrete has attained sufficient strength.

Backfill materials should consist of inorganic materials, free of debris and be free draining. The wall backfill and new compacted fill shall be placed in 8-inch maximum loose lift thickness and compacted to at least 95 percent of the maximum dry density per ASTM D-698. Loose lift thickness of 4 inches, and light-weight/hand-held compaction equipment, shall be used in close proximity to the wall face.

The fill placed adjacent to the below grade walls should not be over compacted. Heavy earthwork equipment should maintain a minimum horizontal distance away from the below grade walls of 1 foot per foot of vertical wall height. Lighter compaction equipment should be used close to the walls. The ground surface adjacent to the walls should be kept properly graded to prevent ponding of water adjacent to the walls.

If there is any space between the outside of the walls and the excavation, it should be backfilled with a granular fill extending to a level of approximately 2 feet below the final outside grade. The remaining 2 feet should consist of a clayey material to minimize the amount of surface water infiltration into the wall backfill. The ground surface adjacent to the below grade walls should be kept properly graded to prevent/minimize ponding of water adjacent to the walls.

F. Pavement Areas

It is recommended that topsoil and any other soft or unsuitable materials be removed from the paved area. The stripped surface shall be proofrolled and carefully observed and evaluated by a geotechnical engineer at the time of construction in order to aid in identifying the localized soft or unsuitable materials, which shall be removed. As discussed earlier, if CH soils are encountered in the pavement areas, the CH soils shall be undercut to a minimum depth of 2 feet below subgrade elevations, and replaced with non-expansive soils.

An important consideration with the design and construction of pavements is surface and subsurface drainage. Where standing water develops, either on the pavement surface or within the base course layer, softening of the subgrade and other problems related to the deterioration of the pavement can be expected. Furthermore, good drainage should minimize the possibility of the subgrade materials becoming saturated over a long period of time. Please note that boring results indicate that the pavement subgrade will most likely consists of fine-grained soils with relatively low to very low permeability. Therefore, if adequate drainage is not provided within the pavement structure, then accelerated deterioration of pavement structure will occur.

It should be noted that these design recommendations are for parking and driveways only and may not satisfy the State traffic guidelines. Any roadways constructed for public use, to be dedicated to the State for repair and maintenance must, be designed in accordance with the State requirements. We would be happy to provide such pavement designs as an additional service, if provided with traffic frequency information.

It is common practice in construction to install only the base aggregate and the base (or intermediate) course asphalt during initial construction, and then the final topping surface asphalt much later in the construction process. Often, depending upon the sequence and timing of construction, the final pavement surface may not be placed until several months to even years after the initial base asphalt is placed. The most critical load conditions for most developments occur during the construction phase. In particular, the pavement system is subjected to its maximum loading and frequency, which commonly includes trucks, construction materials deliveries, and other heavy, high concentrated truck loading which does not occur once the development is finished. Not only does this represent the highest traffic loading condition, but it occurs at a time when the pavement section is not at its full strength, simply because the surface asphalt has not been placed. Depending upon the time in which the temporary construction is used as a service road, some failures should be expected. If the construction pavement system fails, it will be necessary to remove this failed section and replace it with the initial design section or an equivalent repaired section.

Large, front-loading trash dumpsters frequently impose concentrated front-wheel loads on pavements during loading. This type of loading typically results in rutting of the pavement and ultimately pavement failures. Hence, in these cases, we recommend utilizing a Portland Cement Concrete (PCC) section consisting of 6-inches of concrete (4,000 psi) underlain by 8-inches of compacted VDOT 21A or 57. California Bearing Ratio (CBR) tests and pavement design were not requested and therefore not included in the scope of work for this project. It is recommended that CBR tests be performed on the pavement subgrdae soils to permit proper design of the pavement section.

As discussed earlire, no specific analysis has been made for the design of the proposed pavements. The following comments are basic considerations which will not eliminate the need for a careful review, analysis and laboratory-testing program. In designing the proposed pavemnt areas, the existing subgrade conditions must be considered together with the expected traffic use and loading conditions. The conditions that will influence the design can generally be summarized as: 1) Bearing values of the subgrade (these can be represented by CBR for the design of flexible pavements, or a Modulus of Subgrade Reaction (k) for rigid pavement structures), 2) Ground water conditions (variations in water levels, expansive considerations and the necessity for underdrains), 3) Vehicular traffic, in terms of the number and frequency of vehicles and their range of axle loads), 4) Probable increase in vehicular use over the life of the structure, and 5) The availability of suitable materials to be used in the construction of the pavement and their relative costs.

It should be noted that generally, flexible pavements derive their strength from 1) The existing subgrade soils, 2) Any additional compacted fill soils, 3) Stabilization of the subgrade, 4) The base course, and 5) The asphaltic concrete. It should also be noted that the strength of the granular soils may be increased by proof compacting or by stabilization with cement, whereas the stability of clay subgrades may be increased by lime stabilization. Subgrades of higher strength generally require less pavement thickness. Please note that any roadways constructed for public use, to be dedicated to the State for repair and maintenance must, be designed in accordance with the State requirements. We would be pleased to provide these additional services.

G. Site Utilities

Special attention should be given to ensure that the utilities are not located within the zone of influence of new foundations. The influence zone of a foundation is defined as the area beneath a 45° line extending downward from the exterior edge of the exterior footing. For the installation of utilities, the proposed building foundations should be located outside of an envelope defined by a 45° line starting at 10 feet beyond the edge of the pipe.

If it is determined that the foundations are to be located within the above described envelope, then the foundation should be lowered to an elevation where it is outside of this envelope. Special consideration should also be given to the extent of the utility easement. The utility itself may be outside of the zone of influence, however, any excavations made within the easement, such as for repairs, may fall within the influence zone and have an effect on adjacent house foundations. Undermining of adjacent foundations must be prevented.

H. Closing

This report has been prepared for the exclusive use of Dr. Navdeep Sharma and his consultants to assist in the development of the above referenced site. Once the plans and specifications are prepared, our office should be retained to review them and revise the recommendations if deemed necessary. The subsurface conditions as outlined in this report are based on the limited number of augers performed and represents our interpretation of the data presented on the hand auger logs; other variations may occur on-site. In providing this limited exploration and general recommendations, our services were performed in accordance with generally accepted engineering principles and practices. Our office will not be responsible for interpretations made by others based on the data contained in this report. We also recommend that we be given the opportunity to provide construction-testing services for this project.

We have appreciated the opportunity to be of service to you. If you have any questions with regard to the information and recommendations contained in this report, of if we can be of further assistance to you during development, please do not hesitate to contact us.

Sincerely,

Mirkazem Farhangi, P.E.

cc: Mr. Nasser Alikhani, P.E.-Sitech Consulting Group