HILLIS-CARNES
ENGINEERING ASSOCIATES

Report of Subsurface Exploration and Geotechnical Engineering Services
for
Marcey Road Park
2722 N. Marcey Road, Arlington, VA
HCEA Project Number: C20043

May 22, 2020

Prepared For:

Mr. Aaron Wohler, RLA
Arlington County
Office of Support Services
Arlington, VA 22201

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May 22, 2020

Mr. Aaron Wohler, RLA
Arlington County
Office of Support Services
Arlington, VA 22201

Re: Report of Subsurface Exploration and Geotechnical Engineering Services
Marcey Road Park
2722 N. Marcey Road, Arlington, VA
HCEA Project Number: C20043

Dear Mr. Wohler:

Hillis-Carnes Engineering Associates, Inc. (HCEA) is pleased to submit this Geotechnical Investigation and Testing Report for the proposed improvements on Marcey Road park located at the above referenced project site in Arlington, VA. The enclosed geotechnical report presents the results of the field exploration, laboratory testing program, engineering analysis, and recommendations for the proposed development. The recommendations contained in this report are intended for use by your office and for the use of other design professionals involved with the design and implementation of the proposed development at this site.

We thank you for your confidence in our services and appreciate the opportunity to serve you as a geotechnical consultant on this project. We will remain available for future consultation during the design and construction phases of the project.

Please contact our office if you have any questions.

Sincerely,
HILLIS-CARNES ENGINEERING ASSOCIATES, INC.

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1. PROJECT INFORMATION

Per your authorization by the acceptance of our proposal Number P200079MAN, dated April 20, 2020, Hillis-Carnes Engineering Associates, Inc. (HCEA) has completed the authorized Subsurface Exploration and Geotechnical Investigation carried out for the proposed improvement of Marcey Road Park located in Arlington, VA. We understand that the improvement consists of construction of a picnic shelter and segmental block walls, and replacement of the existing asphalt pavement of the parking area with a permeable pavement.

In the following sections, the scope of our services, field and laboratory explorations, our findings and geotechnical recommendations are presented.

1.1 Project Location and Site Description

The project site is located in Marcey Road Park at 2722 N. Marcey Road in Arlington, Virginia. The park consisted of asphalt tennis and basketball courts and a brick gazebo. Potomac Overlook Park was located just north of the Marcey Road park. A project location map is attached in Appendix I at the end of this report.

1.2 Purpose and Scope

The purpose for this study is to document the geotechnical investigation performed at the above referenced project site for the proposed improvement of Marcey Road Park.

The services provided by HCEA involved exploring the subsurface conditions at the proposed picnic shelter, SWM and pavement exploratory borings locations, the performance of laboratory tests, review of published information, engineering analyses to develop appropriate engineering recommendations, and preparation of report containing our findings and geotechnical recommendations. In addition, in-situ infiltration tests were performed at the proposed parking area with the objective of evaluating the suitability of the subsurface soils for a permeable pavement. To accomplish these objectives, the following scopes of services were undertaken:

- Consulted available published geologic and project references relative to the project site.
- Visited the site to observe existing surface conditions and features.
- Coordinated utility clearance with VA Miss Utility.
- Explored and tested in-situ conditions at boring locations.
- Performed infiltration tests on selected borings at the pavement location.
- Performed laboratory tests on representative soil samples.
- Analyzed the results of our office, field, and laboratory studies.
- Prepared profiles (logs) of soil/rock conditions along the borings in the area of the proposed construction.
- Developed design criteria for foundations and related geotechnical considerations.

- Examined the relative merits of alternative methods of geotechnical designs.
- Provided permeable pavement recommendations.
- Provided SWM recommendations.
- Prepared a written report summarizing our work on the project, providing general descriptions of the encountered subsurface conditions, and other geotechnical related aspects of the proposed project that were readily apparent at the time in the areas of our investigation, and provide pertinent geotechnical recommendations including anticipated settlements, frost penetration depth, construction methods and anticipated construction problems.

Our scope of services did not include a precise survey of test boring locations using surveying instrumentation, quantity estimates, preparation of plans or specifications, the identification and evaluation of environmental related aspects of the project site, or any other item not specifically included in our scope of work.

2. EXPLORATION AND TESTING PROCEDURES

2.1 Subsurface Exploration

Geotechnical exploratory borings were drilled on May 7, 2020 at the locations of the proposed picnic shelter, SWM facility and pavement area.

A total of three (3) standard SPT soil borings were drilled to depths of 10 feet below existing site grades within the project site. Boring B-1 was located at the parking area of the park. Boring B-2 was drilled on the north side of the tennis court at the proposed location of the bioretention facility. Boring B-3 was advanced at the proposed location of the picnic shelter. Borings IT1 and IT2 were auger drilled to a depth of 6 feet for in-situ infiltration testing in the area of the pavement near SPT boring B-1.

A site map showing boring locations is included in Appendix II attached at the end of this report.

2.1.1 Standard SPT Borings

The SPT soil borings were drilled by HCEA drilling crew using an all-terrain vehicle (ATV)-mounted auger drilling rig. In this procedure, split-barrel sampler is driven into the soil 18 or 24 inches by a 140-pound hammer falling 30 inches. The hammer strikes required to drive the sampler through a 12-inch interval is called the 'standard penetration resistance', or otherwise called the ‘SPT or N-value'. The SPT value can provide a qualitative indication of the in-situ relative density of cohesionless soils and less reliably the consistency of cohesive soils. This indication is qualitative, since many factors can significantly affect the standard penetration resistance value and prevent a direct correlation between drill crews, drill rigs, drilling procedures, and hammer-rod-samples assemblies. The standard penetration resistance, when properly evaluated, is an index to the soil strength and compression characteristics.

Subsurface soils were sampled at 2.5 ft and 5 ft intervals. Samples were taken by driving a 1- 3/8-inch I.D. (2-inch O.D.) split-spoon sampler in accordance with ASTM D-1586 specifications. Drilling fluid was not used in the drilling portion of this process. The specific drilling method and SPT or N values are noted on the individual boring logs.

Upon completion of drilling, the boreholes were backfilled with the auger spoils generated during the drilling operations. Field logs of the soils encountered in the borings were maintained by the drill crew. The field observations include description of soil stratum encountered, estimated depth and thicknesses of each stratum, the SPT or N values, and groundwater observations. Please refer to the Records of Soil Exploration (borehole logs in Appendix III) appended to this report for details related to the subsurface conditions encountered in the test borings.

2.1.3 In-situ Infiltration Testing

The in-situ infiltration testing was performed in general accordance with the Arlington County's Geotechnical requirements for infiltration facilities. The infiltration test boring was drilled 4 feet below the planned SWM/infiltration structure bottom elevation to verify the existence of groundwater or bedrock within this depth.

In close proximity to the infiltration test borehole (B-1), two augered borings (IT-1 and IT-2) were drilled for conducting infiltration testing. A solid 5-inch diameter PVC pipe was installed at a depth of 6 feet below the existing site grades at each infiltration test location.

The PVC pipes were filled with 24 inches of water and were allowed to soak for 24-hours. Following the 24 hours soaking period, a 4-hour constant head infiltration test was conducted in each test boring, with an initial water head of 24 inches. The head drop was then measured hourly, and the average of readings over a period of 4 hours are reported as the infiltration rate measured in inches per hour (in/hr.).

2.1.4 Groundwater Observations

Groundwater observations were made during drilling, after drilling before the augers were removed, and at completion of the test borings by a visual examination of recovered samples from the standard penetration tests, auger cuttings, and water marks on the split-barrel sampler and drill rods. A 24-hour groundwater observation was also done.

2.2 Soil Laboratory Analyses

Representative soil samples were obtained by means of the split-barrel sampling procedure in accordance with ASTM Specification D-1586. The soil samples were placed in sealed jars and transported to the HCEA soil laboratory for visual evaluation, classification and material testing in accordance with ASTM Standard D-2488, "Classification of Soils for Engineering Purposes" by performing specific laboratory tests to check field classifications and determine pertinent engineering properties.

The laboratory tests included visual classifications, natural moisture content tests, particle size (gradation) analysis, and Atterberg limits tests of selected soil samples following the following test methods.

ASTM D-422	Particle Size Analysis of Soils
ASTM D-4318	Atterberg Limits (LL, PL &PI)
ASTM D-2216	Moisture Content of Soils
ASTM D-698	Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort

Each soil sample was classified on the basis of texture and plasticity in accordance with the Unified Soil Classification (USCS) method. A brief explanation of the USCS soil classification method is included in this report (Appendix IV).

These tests were performed to determine the physical characteristics and soil classification of the various soils encountered during the subsurface investigation. The laboratory test results are presented in the individual data sheets annexed to this report (Appendix IV-Soil Laboratory Test Results).

The soil samples will be retained in our laboratory for a period of 60 days from the date of this report, after which, they will be discarded unless otherwise instructions are received as to their disposition.

3. RESULTS

3.1 Regional/Site Geology and Soils

3.1.1 Regional/Site Geology

We reviewed the United States Geologic Survey (USGS) Geological Map of the Arlington County, Virginia. This review of the published geological information indicates that the project site is geologically situated in the Sykesville Formation. The formation mainly consists of light- to medium-gray, medium-grained metasedimentary melange consisting of a quartzofeldspathic matrix that contains quartz "eyes" and a heterogeneous suite of pebble to boulder and larger-size olistoliths. These include Mather Gorge Formation migmatite, phyllonite, and metagraywacke; also, ultramafic, metagabbroic, and felsic and mafic metavolcanic rocks, plagiogranite, and quartzite. The Sykesville is intruded by Occoquan Granite.

3.1.2 Soils

To examine soil units mapped at the site, we accessed the USDA, NRCS soils map and the soil survey of Arlington County, Virginia. Per the soils map, soils at the site belong to **6C - Glenelg loam,** (8 to 15 percent slopes). The Glenelg series consists of very deep, well drained soils formed in residuum weathered from micaceous schist on uplands of the Blue Ridge and the Northern Piedmont. Slopes range from 0 to 55 percent. Saturated hydraulic conductivity is moderately high in the subsoil and moderately high to high in the substratum. Mean annual temperature is 53 degrees ^{0}F and mean annual precipitation is 40 inches.

3.2 Subsurface Observations

Based on the results of the subsurface investigations and laboratory testing performed by our soil laboratory, the subsurface conditions are generally consistent with the regional/site geology and soils, and can be categorized into two generalized subsurface strata: Stratum I- Fill Soils and Stratum II- Residual Soils.

The following subsections provide a summary of the subsurface conditions encountered. Detailed information about the soils encountered in each soil exploratory boring are also presented on the soil boring logs (Appendix III) and laboratory test results (Appendix IV). It should be noted that stratification lines shown on the records of Soil Exploration represent approximate transitions between material types. In-situ strata changes could occur gradually or at slightly different levels. Note also that the test boring logs depict conditions at the particular locations and at the particular times indicated.

3.2.1 Stratum I: Man-Placed Fill Materials

Boring B-1 was located on the existing asphalt paved parking area. The pavement at the location of boring B-1 consisted of 3 inches of asphalt concrete underlain by 3 inches of aggregate base. The remaining two borings were located in grassy areas and consisted of 2 inches of topsoil. Apparent fill materials that extended to a depth of 2.5 feet were encountered in borings B-1 and B-3 below the surface materials. The fill materials consisted of medium stiff SILT (ML) and silty CLAY (CL-ML) soils.

The existing fill is believed to be related to previous construction/grading activities in the area and may be encountered in other areas of the site. Since the size of the samples obtained is relatively small in comparison to the area extent of the site and since fill materials could be of similar composition to the natural soils encountered at the site, it is often difficult to determine the presence and composition of fill materials from the SPT samples. It should be anticipated that man-placed fill materials may be encountered at other locations and to different depths below the existing ground surface than indicated by the Records of Soil Exploration.

3.2.2 Stratum II: Natural Soil

The second stratum associated with the site-specific general soil profile was encountered below the surface/fill layer and consists of SILT with sand (ML) and silty SAND (SM).

SPT N-values in this stratum soil ranges between 1 to 22 bpf, corresponding to a wide range of consistency ranging from very soft to very stiff for cohesive soils and loose to medium dense for non-cohesive soils. See the attached soil boring logs (Appendix III) and soil laboratory results (Appendix IV).

3.3 Groundwater

Groundwater was not encountered in the boreholes during the soil exploratory borehole drilling. However, the actual presence of groundwater is expected to fluctuate annually and seasonally at the site depending on variations in precipitation and evaporation; it is also expected to be significantly influenced by surface runoff and rainfall.

The actual level of the hydrostatic water table and the amount and level of perched water should be anticipated to fluctuate throughout the year, depending on variations in precipitation, surface run-off, infiltration, site topography, and drainage. Moreover, seasonally perched water will likely be encountered in wet seasons at the interface of soil and rock strata.

3.4 Laboratory Test Results

The results of the laboratory tests indicate that the onsite natural soils sampled generally classify as Silt (ML) and silty SAND (SM) soils.

All data obtained from the laboratory tests are included on the respective boring log and on separate sheets in the Appendix at the end of this report (Appendix III and Appendix IV).

3.4 In-Situ Infiltration Test Results

The individual infiltration test results are included as an attachment to this report (Appendix V) and summarized in the Table 2 below:

Table 1 - Summary of Infiltration Test Results

Infiltration Boring no.	Depth of Boring from Existing ground level (ft.)	Estimated Infiltration Rate (inches/hour)
IT-1	6.0	2.62
IT-2	6.0	0.51

All in-situ infiltration test rates were found to be higher than the required minimum infiltration rate of 0.5 in/hr.

4. GEOTECHNICAL RECOMMENDATIONS

Based on the findings of this geotechnical exploration and a review of available documents for the proposed development, we believe that the project site is generally suitable for the construction of the proposed two-story office building and related structures development provided the recommendations in this report are followed, and the proposed structure is designed and constructed in accordance with pertinent State and County Guidelines.

The following sections provide general construction guidelines for site grading and earthwork activities, also geotechnical requirements for foundation support and floor slabs.

4.1 General

4.1.1 Suitability of on-site Materials

The natural in-situ soil of Stratum II generally consists of silty SAND (SM) and SILT with sand soils. These soils are generally considered as suitable for use as structural fill and in support of building footings, grade slabs and pavements and as backfill over utilities.

Where expansive or highly plastic silts or clays (CH/MH) are encountered at or near footing subgrade during construction, the material shall be removed and replaced with properly compacted structural fill. As an alternative method, the foundation subgrade shall be extended to a depth of at least 4 feet below finished exterior grade, or through the (CH/MH) materials if less than 4 feet below finished exterior grade. The deeper embedment depth will extend the footings below the typical depth of seasonal moisture fluctuation in the expansive or high plasticity elastic silt. At the 4-foot minimum embedment depth, the footing may bear on high plasticity elastic (CH/MH) type soils or on non-expansive soils. Also, if footings placed at a normal embedment depth of 2.5 feet extend below the thickness of the high plasticity/elastic soils, then the footings can be structured at nominal depth.

Where expansive or highly plastic silts or clays (CH/MH) are encountered at design subgrade elevations in slabs and pavement areas, the subgrade should be undercut 2 feet and grades should be restored with approved fill materials.

4.1.2 Site Excavation

We anticipate that conventional earth-moving equipment will be suitable for the excavation of the fill materials of Stratum I and natural soils of Stratum II of the project site.

Temporary excavations greater than 4 ft shall be properly shored or sloped away from the excavation with minimum grade of 1.5H: 1V. Trench boxes shall be utilized if sloping of temporary trenches and pits is not desired. All excavations shall be performed in accordance with OSHA regulations. If any excavations, including utility trenches, are extended to depths of more than 20 ft, OSHA requires that the side slopes of such excavations be designed by a professional engineer.

4.2 Site Preparation and Earthwork

4.2.1 General Site Preparations

Site preparation and grading should consist of clearing, grubbing and stripping of topsoil, rootmat, and any other soft or unsuitable materials from the 10-feet expanded building and 5-feet expanded pavement limits and to 5 feet beyond the toe of structural fills. The clearing should also include removal of the existing brick gazebo and the full depth of the asphalt pavement.

Trees, snags, stumps, shrubs, brush, limbs, and other vegetative growth, and all evidences of their presence including sticks and branches greater than 1-inch in diameter or thickness should be cleared from the surface. After clearing, wood or root matter including stumps, trunks, roots, or root systems greater than 1-inch in diameter or thickness to a depth of 12 inches below the ground surface should be removed and disposed. Disposal of cleared material should be done in accordance with all local and state laws requirements.

Removal should also include topsoil; frozen, wet, soft, or very loose soils; expansive or highly plastic silts or clays (CH/MH); unapproved man-placed materials, and any other deleterious materials.

HCEA should be called on to verify that topsoil and unsuitable surficial materials have been completely removed prior to the placement of structural fill or construction of structures.

After the completion of clearing, grubbing and stripping, the exposed subgrade areas of the site to receive fill, or areas of the site at-grade where structures will be located should be thoroughly examined to identify any localized loose, yielding, or otherwise unsuitable materials, and then proofrolled.

The proofrolling operations should be performed using a 20-ton, fully loaded dump truck or another pneumatic-tire vehicle of similar size and weight. The purpose of the proofrolling is to assist in locating any near-surface pockets of soft or loose yielding materials requiring undercutting. The areas subject to proofrolling should be traversed by the equipment in two perpendicular (orthogonal) directions with overlapping passes of the vehicle under the observation of the geotechnical engineer or authorized representative.

If unstable or “pumping” subgrade is identified by the proofrolling, those areas should be marked for repair prior to the placement of any subsequent structural fill or other construction materials. Methods of repair for unstable subgrade, such as undercutting or moisture conditioning, use of soil bridging lifts (reinforcing geotextile or geo-grid), or chemical stabilization, should be discussed with HCEA to determine the appropriate procedure with regard to the existing conditions causing the instability. A test pit(s) may be excavated to explore the shallow subsurface materials in the area of the instability to help in determining the cause of the observed unstable materials and to assist in the evaluation of the appropriate remedial action to stabilize the subgrade.

4.2.2 Borrow Materials

All borrow materials, whether on-site or imported from an off-site source, should be tested for suitability and quality prior to its use as structural fill. We recommend that the material be tested to determine particle size (gradation), plasticity, and maximum dry density. The following standard tests should be performed to determine the above properties of all imported fill materials:

Particle Gradation	ASTM D-422
Atterberg Limits	ASTM D-4318
Standard Proctor	ASTM D-698

Structural fill material shall consist of quality, low plasticity, non-organic soil that classifies as GW, GP, GM, GM-GP, GC, SW, SP, SM-SP, SM or SC in accordance with ASTM D-2487 and shall have a maximum of 30% retained on a standard 3/4-inch sieve. Structural fill may consist of soils that classify as ML and CL provided that the material has a liquid limit and plasticity index less than or equal to 40 and 15, respectively; with a maximum dry density (MDD) of more than 105 pcf, and with a maximum of 70 percent passing the US Standard No. 200 sieve. All fill material shall be free of ice, snow, organic material (OH, OL), expansive soils of high plasticity/elasticity (CH/MH), construction debris, rock sizes greater than 3 inches, or other deleterious material.

Soils meeting all four of the following provisions shall be considered expansive, except that tests to show compliance with Items 1, 2 and 3 shall not be required if the test prescribed in Item 4 is conducted:

1. Plasticity index (PI) of 15 or greater, determined in accordance with ASTM D 4318.
2. More than 10 percent of the soil particles pass a No. 200 sieve (75 µm), determined in accordance with ASTM D 422.
3. More than 10 percent of the soil particles are less than 5 micrometers in size, determined in accordance with ASTM D 422.
4. Expansion index greater than 20, determined in accordance with ASTM D.

Expansive soils are not permitted as structural fill under building pads, foundation backfill, and backfill around structures.

4.3 Foundation System Recommendations

Based upon the results of our geotechnical study done to date, currently it is the opinion of Hillis-Carnes Engineering Associates that the proposed picnic shelter shade structure, from a geotechnical loading viewpoint, may be supported on a spread footing foundation system bearing on approved naturally occurring materials, or on controlled (structural) fill placed over approved materials, or on a combination thereof.

Based on the general soil conditions which were encountered in the area of the proposed picnic shelter, it is our professional opinion that footings supported on natural undisturbed soils or newly placed compacted structural fill may be designed for a net allowable soil bearing pressure of 2,000 psf.

The net allowable soil bearing pressure refers to that pressure which may be transmitted to the foundation bearing soils in excess of the final minimum surrounding overburden pressure.

As a minimum, isolated column footings should be constructed with a minimum of 24 inches in size for punching shear consideration only. In addition, adequate frost cover protection shall be provided for all footings. We recommend the footings to be located at a minimum depth of 2.5 feet below finished grade levels.

Approximate 2.5 feet of existing man-placed fill materials were encountered in boring B-3. Based on the SPT N-values and review of the split spoon samples, it appears that the fill materials were not placed for the purpose of supporting future development. Therefore, the existing fill materials should be over excavated until natural residual materials are encountered and replaced with controlled structural fill placed in accordance with the recommendations of Sections 4.1 and 4.2 of this report or with lean (2000 psi) concrete. Furthermore, if soft or loose pockets are encountered in the footing excavations, the unsuitable materials should be removed, and the footings should be located at a lower elevation. Alternatively, the unsuitable materials could be undercut and replaced with either new fill placed or with lean (2000 psi) concrete.

4.4 Ground Supported Concrete Slabs

For the design and construction of concrete slabs of the picnic shelter, the recommendations provided in the Site Preparation and Earthwork section should be followed. The concrete slab area should be observed by the geotechnical engineer or authorized representative to aid in locating any soft or unsuitable materials. Slab-on-grade shall be supported on low plasticity natural soils or on approved compacted structural fill.

A subgrade reaction modulus of 125 pci may be used for the design of concrete slabs-on-grade supported on low plasticity natural soils or approved compacted structural fill. Slab-on-grade subgrade preparation should be visually inspected, and subgrade should be proofrolled to check suitability and firmness prior to placement of the stone layer. If the visual inspection of the subgrade material and/or hand auger recovered material reveals the presence of fine-grain soils, i.e. clays or silts, we recommend that a sample of the soil subgrade be tested to ensure that high plasticity soils, having liquid limit and plasticity index values greater than 40 and 15, respectively, are not present at the slab subgrade. High shrink/swell potential and high plasticity/expansive soils (CH/MH), when encountered, should be undercut to at least 2 feet below the slab subgrade and replaced with properly compacted structural fill. If any soft or yielding soils are observed during proofrolling, then the existing soils may need to be removed and replaced with compacted structural fill in accordance with the recommendations contained in this report.

We also recommend that all concrete slabs be designed to be discontinuous at footings so that differential settlement will not induce shear stresses in the concrete slab. Furthermore, we recommend mesh reinforcement be included in the design of the floor slab to reduce shrinkage crack that may develop near the surface of the slab. The slab should rest upon a minimum of 6 inches of free draining granular base having a maximum aggregate size of 1.5 inches and no more than 2% fines placed on top of suitable and firm subgrade. A minimum 10-mil thick impermeable plastic membrane (vapor barrier) should be placed between the granular blanket and the overlying concrete slab on grade to limit moisture migration.

Utility or other construction excavations in the prepared concrete slab subgrade should be backfilled to controlled fill criteria to provide uniform floor slab support.

4.5 Subsurface Utilities

Most of the exploratory borings encountered firm natural residual soils that are expected to be suitable for support for subsurface utility lines. Where rock is encountered at the subgrade level, it should be removed to at least 6 inches below and 8 inches outside of the utility lines. The subgrade should be observed and probed to evaluate the suitability of materials encountered. All loose, organic and unsuitable materials should be removed and replaced with suitable compacted fill or bedding material.

Utility lines should be bedded on at least four inches of granular bedding materials meeting the specifications of the pipe manufacturer or local requirements. Granular bedding materials typically consist of at least 6 inches of coarse, open graded gravel or crushed stone.

Infiltration of water to the utility trenches must also be prevented before, during, and after construction. Excavations should not be allowed to remain open if rain is anticipated. Excavations should be backfilled with clean, suitable cohesive structural fill to minimize potential moisture infiltration.

We recommend that the utility trenches be backfilled in accordance with the State of Virginia Standard Specifications and requirements and based upon specific utility backfill requirements. Utility trench backfill is recommended to be compacted to a minimum 98 percent of the materials Standard Proctor maximum dry density at a moisture content of plus or minus two percent (± 2%) of optimum moisture content. Compaction for areas beyond 20 ft of new buildings and pavements may be reduced to 95 percent of the material maximum dry density. The backfill should be placed in 8 inches maximum loose lifts. In areas within the VDOT right-of-way, an increased compaction density of 100% of the maximum dry density will apply for the upper 6 inches of the pavement subgrade.

4.6 Fill Materials and Fill Placement

Fill materials should be placed in no greater than 8-inches thick loose lifts and compacted to at least 95% of the maximum available dry density as determined in accordance with the Standard Proctor as determined by ASTM D-698, AASHTO T99, or VTM-I. Where fill depths of more than 10 feet are required, we recommend that the compaction criteria be increased to 98% of the maximum dry density (standard proctor) obtained in accordance with ASTM D-698, AASHTO T99, or VTM-I for the full depth of the fill. The moisture content of the fill being placed should be within a 2 percent deviation from the optimum moisture content of the material (±2 % of OMC).

Backfill in areas not subject to vehicular traffic shall be compacted sufficiently (90% of the maximum dry density obtained in accordance with ASTM D-698, AASHTO T99, or VTM-I of the Standard Proctor Method) so that any subsidence that may occur shall not be objectionable or detrimental to normal use.

Backfill material shall be free of organic material, frozen clods, expansive clays or highly plastic silt (CH/MH) and other unsuitable material. Fill material shall not be placed on frozen soils. All frozen soils should be removed prior to continuation of fill operations. Fill materials shall not contain frozen materials at the time of placement. All frost-heaved soils should be removed prior to placement of fill, stone, concrete, or asphalt. Backfill and replacement work in existing or proposed roads to be accepted into the VDOT system shall be executed in accordance with all applicable VDOT standards. All surplus materials shall be disposed in approved areas.

All new fill materials should be properly benched into the existing slopes to prevent formation of shear planes at the interface of the fill mass and the existing natural soils.

In- place density tests should be performed with a minimum of one test per 2,500 square foot of fill area for each lift of fill paced. To ensure proper compaction efforts, field density determinations should be performed in accordance with specifications set forth in ASTM D-6938 (Nuclear Density Method) or D-1556 (Sand Cone Method). We recommend that density tests be performed on every lift of compacted structural fill placed in building, pavement and utility trench areas.

Compaction equipment should be suitable to the type of fill material. Although any equipment type can be used as far as the required density is achieved, ideally, a steel drum roller would be most efficient for compacting and sealing surface soils. All areas receiving fill should be graded to facilitate positive drainage from the embankment and slopes of any free water associated with precipitation and surface runoff.

4.7 Stormwater Management (SWM) Facility

4.7.1 Urban Bioretention

Based on the general site layout plan provided to us and information obtained from the client, the proposed development includes the construction of Level I Urban Bioretention on the north side the tennis court.

Soil exploratory boring B-2 was drilled at the location of the proposed bioretention. The exploratory boring indicates the presence of residual Silt (ML) and silty SAND (SM) soils. Groundwater was not encountered during drilling and when re-checked later after 24 hrs. Bedrock was also not encountered within the drilled depth. Infiltration testing was not performed at the boring location. Hence, an underdrain should be used as per "Virginia DCR Stormwater Design Specification No.9" requirement. The underdrains should be 6 inches diameter perforated rigid schedule 40 PVC pipe with 3/8-inch perforations at 6 inches on center length wise with a maximum of 3 rows of perforations as per "the Stormwater Manual of Arlington County". The Boring log is presented in Appendix III attached at the end of this report.

4.7.2 Permeable Pavement

As per the site plan submitted by the client, the full depth of the existing asphalt paved parking area is expected to be removed and replaced by a permeable interlocking paver. The permeable interlocking pavers shall conform to all requirements of Interlocking Concrete Paver Institute (ICPI) Technical Specification Number 18 (or equivalent) as required by "Virginia DCR Stormwater Design Specification No.9".

4.7.2.1 In-Situ Infiltration Testing

Soil Exploratory boring B-1 and infiltration test borings I-1 and I-2 were located at the proposed location of the permeable pavement. Infiltration tests were conducted at a depth of 6 feet below existing site grades. An infiltration rate of 2.62 and 0.51 in/hr was encountered at the location of boring IT-1 and IT-2, respectively. The infiltration rates encountered in the borings are higher than the required minimum infiltration rate 0.5 in/hr as per "Virginia DCR Stormwater Design Specification No.9". The specification also states that the bottom of the infiltration facility should be located a minimum of 4ft above the seasonally high-water table and/or bedrock. Bedrock and groundwater were not encountered within the drilled depth of the boring. Therefore, the underlying soils are considered to be suitable for infiltration practices.

4.8 Permeable Pavement

The following pavement section may be desired at the proposed location of the permeable pavement as per "the Stormwater Manual of Arlington County".

	Thickness (inches)
Permeable Interlocking Pavers	3.0 (minimum)
Bedding Course layer	2.0
Base Stone	4.0
Reservoir layer	8.0

Uncompacted Subgrade Soil (minimum CBR =4)

The California Bearing Ratio (CBR) of the underlying subgrade soils should be tested before construction to check the availability of a CBR of at least 4%. If a CBR of less than 4% is encountered, the underlying subgrade soils may need to be compacted to at least 95% of the Standard Proctor density, which will limit the infiltration capacity of the soils.

In this case, an underdrain should be placed within the reservoir layer. A minimum of 2 inches of aggregate should be placed above and below the underdrains. The underdrains should be 4 to 6 inches diameter perforated schedule 40 PVC pipe, with 3/8-inch perforations at 6 inches on center as per “the Stormwater Manual of Arlington County”. Each underdrain should slope down towards the outlet at a grade of 0.5% or steeper.

A separator geotextile filter fabric should be placed between the subgrade soils and the reservoir layer. The reservoir and bedding course layers should consist of No.2 and No. 57 stone, respectively. Both the No. 2 and No. 57 stone should be washed, clean and free of all fines and compacted with a 10-ton steel drum static roller. The bedding course material should consist of a No. 8 stone.

4.9 Groundwater control

As mentioned in section 3.3, groundwater was not encountered at the project during the subsurface exploration drilling. However, the groundwater is expected to fluctuate annually and seasonally at the site depending on variations in precipitation and evaporation; and it is expected to be significantly influenced by surface runoff and rainfall. We also anticipate seasonal perched water at the interface between soil-rock strata in wetter seasons. Hence, it is likely that earthwork operations encounter difficulties especially in wetter season.

As a result, construction phase dewatering plan may be required. We anticipate that a combination of sump pits, trenching and sump-pumping operations can adequately control infiltrating rainwater or groundwater to a depth of up to 5 ft below foundation elevation. Based on the groundwater elevations, deep wells may also be necessary. Groundwater observation wells would need to be installed and monitored at the site for several weeks to determine the static groundwater level. We recommend HCEA to be contacted to review proposed dewatering methods during the construction phase of the project.

4.10 Site Seismicity

According to the 2018 International Building Code, Section 1613.2.2 (Chapter 20 of ASCE 7), seismic Site Class D should be specified for this project.

4.11 Fill Slopes

Compacted fill slopes, less than 20 ft in height, may be constructed at 2.5 Horizontal to 1 Vertical (2. 5H:1V) or flatter. For steeper slopes, if slopes steeper than 3:1 are required, the Fredrick County may require steep slope waiver request to be submitted. A global slope stability analysis is also required to be prepared by project GER and attached onto the waiver request/submittal for justifications.

4.13 General Construction Considerations

4.13.1 Erosion Control

The site surface soils may be susceptible to erosion. Hence the contractor should provide and maintain appropriate site drainage measures during earthwork operations to maintain the integrity of surface soils. Erosion and sedimentation control measures should be in accordance with sound engineering practices and local requirements.

4.13.2 Subgrade Protection

Measures should be taken to limit site disturbances. It is recommended to designate a haul road and construction staging area to limit areas of disturbances and prevent construction traffic from excessively degrading sensitive subgrade soils. Haul road and construction staging areas could be covered with excess depths of aggregate, which can later be removed and used in pavement areas, to protect those subgrades.

Surface water should be directed away from the construction area, and the work area should be sloped away from the construction area ta a gradient of one percent or greater to reduce the potential of ponding water and subsequent saturation of subgrade soils. At the end of working day, the subgrade soil should be sealed by rolling the surface with a smooth drum roller to minimize infiltration of surface water. Surface drainage conditions should be properly maintained throughout the construction period.

4.13.3 Moisture Conditioning

Delays and additional costs should be anticipated during cool and wet seasons of the year. During these seasons, soil moisture reduction may be needed, which could be accomplished through a combination of mechanical manipulation and using of chemical additives such as lime or cement, to achieve optimal moisture levels appropriate for compaction. On the other hand, during dry seasons of the year, moisture may need to be added to achieve adequate moisture levels appropriate for compaction in accordance with the project requirement.

4.13.4 Excavation Safety

Excavations may require forming or bracing, slope flattening, or other physical measures to control sloughing and/or prevent slope failures. Contractors should be familiar with applicable OSHA codes and requirements to ensure adequate of excavations.

5. CONCLUDING REMARKS

This report is prepared for the exclusive use of Arlington County Office of Support Supplies to assist them and their engineers during the design and construction of the proposed Marcey Road Park improvement in Arlington, Virginia.

The opinions, conclusions, and recommendations contained herein are based upon the SPT soil borings, laboratory analyses, our interpretation of the data, and generally accepted principles of geotechnical engineering. Please note that there are important limitations to this and all geotechnical studies. Some of these limitations have been presented in this report, while others are discussed in the information prepared by The Association of Engineering Firms Practicing in the Geosciences (ASFE).

Please be advised that although the SPT exploratory borings were logged by experienced professional, it is sometimes difficult to record changes in subsurface stratigraphy within narrow limits; therefore, some deviation in the materials reported on the field logs and the materials encountered in the field should be anticipated. If there are any changes to the project characteristics as outlined in this report, HCEA should be retained to review the changes and determine if modifications to the recommendations are necessary and what additional geotechnical recommendations are required for the proposed development. We also recommend HCEA to be contacted once designs are further along to review this report prior to completing the design documents to confirm the recommendations contained herein.

In providing this exploration and professional recommendations, our services were performed in accordance with generally accepted engineering principles and practices. No warranty, either expressed or implied, is made to the professional advice included in this report. This report does not address any environmental issues or impact, if any, on the project.

In addition to geotechnical engineering services, HCEA has the in-house capability to perform multiple additional services such as construction material testing/ special inspections and seismic and optical monitoring as this project moves forward. We would be pleased to provide these services for you. If you have any questions with regard to this information or need any further assistance during the design and construction of the project, please feel free to contact us.

Appendix I: Location Map

N

HILLIS-CARNES
Engineering Associates, Inc.

PROJECT NO.: C20043

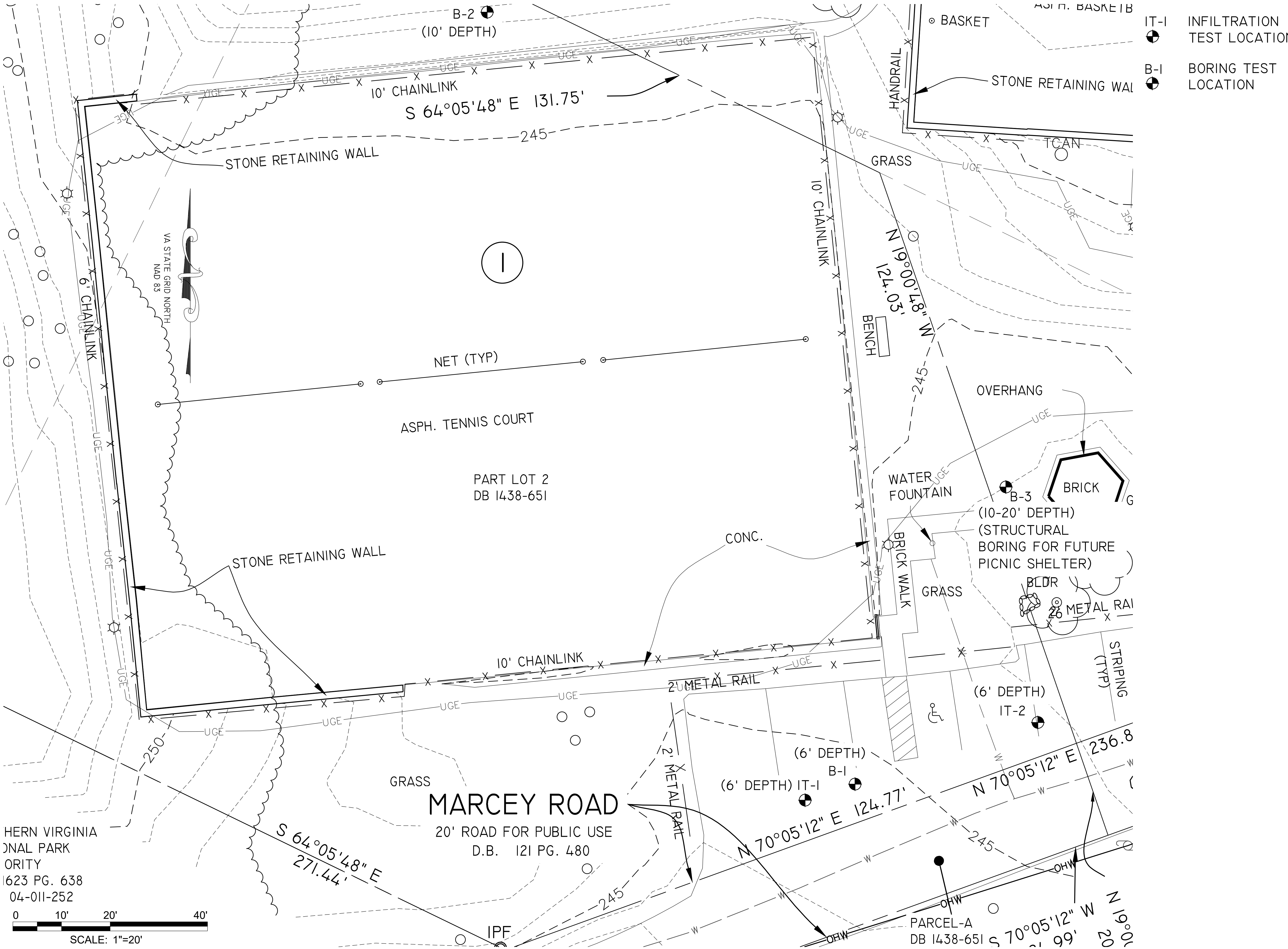
SCALE: NTS

DATE: May 22, 2020

Site Location, Marcey Road Park, Arlington, VA

Figure 1

Appendix II: Boring Location Plan



A. Morton Thomas & Associates, Inc.

14555 Avion Parkway, Suite 150
Chantilly, VA 20151
Phone: 703.817.1373
www.amtengineering.com

Project Title and Location

Marcey Road Park

2722 N. MARCEY ROAD
ARLINGTON, VA 22207

Sheet Title

GEOTECHNICAL BORING LOCATIONS

Drawn: KRF
Filename: ZBORINGEXHIBIT.DWG

Scale: 1" = 20
Date: 04-01-2020

Sheet
EXHIBIT

Appendix III: Boring Logs

HILLIS - CARNES
ENGINEERING ASSOCIATES, INC.

RECORD OF SOIL EXPLORATION

Project Name: **Marcey Road Park** Boring No. B1

Location: **2722 Marcey Rd, Arlington, VA 22207** Job # C20043

SAMPLER

Datum	OGL	Hammer Wt.	140 lbs.	Hole Diameter	in.	Foreman	James Burrowbridge
Surf. Elev. ±	ft.	Hammer Drop	30 in.	Rock Core Diameter		Inspector	FY
Date Started	5/7/2020	Pipe Size	2 in.	Boring Method	HSA-SPT	Date Completed	5/7/2020

Elev (ft)	Depth (ft)	Description	Soil Sym	Sample No	Sample Rec (in)	NM (%)	PI (%)	LL (%)	SPT blows per 6"	SPT Blows/foot N	Boring and Sampling Notes
	0										
-0.6		Brown, moist, medium stiff silt (ML)		1	13				7-4-4	8	3" Asphalt 3" base Probable fill
-2.5		Brown, moist, medium dense silty sand (SM)		2	18				6-5-8	13	
	5			3	18	13.0%	NP	NP	4-6-9	15	
				4	18				6-9-13	22	
-10.0	10										Boring terminated @10ft. depth.
	15										
	20										
	25										
	30										

SPT Graph: 10, 30, 50

SAMPLER TYPE	SAMPLE CONDITIONS		GROUND WATER DEPTH	CAVE IN DEPTH	BORING METHOD
Driven Split Spoon unless otherwise noted	D - Disintegrated	At Completion	Dry ft.	8.5 ft.	HSA - Hollow Stem Augers
PT - Pressed Shelby Tube	I - Intact	After 24 Hrs.	Dry ft.	ft.	CFA - Continuous Flight Augers
CA - Continuous Flight Auger	U - Undisturbed	After ___ Hrs.	ft.	ft.	DC - Driving Casing
RC - Rock Core	L - Lost				MD - Mud Drilling

STANDARD PENETRATION TEST-DRIVING 2" O.D. SAMPLER 1' WITH 140# HAMMER FALLING 30": COUNT MADE AT 6" INTERVALS.

HILLIS - CARNES ENGINEERING ASSOCIATES, INC.

RECORD OF SOIL EXPLORATION

Project Name: **Marcey Road Park** Boring No. B2

Location: **2722 Marcey Rd, Arlington, VA 22207** Job # C20043

SAMPLER

Datum: OGL | Hammer Wt.: 140 lbs. | Hole Diameter: ___ in. | Foreman: James Burrowbridge

Surf. Elev. ±: ___ ft. | Hammer Drop: 30 in. | Rock Core Diameter: ___ | Inspector: FY

Date Started: 5/7/2020 | Pipe Size: 2 in. | Boring Method: HSA-SPT | Date Completed: 5/7/2020

Elev (ft)	Depth (ft)	Description	Soil Sym	Sample No	Sample Rec (in)	NM (%)	PI (%)	LL (%)	SPT blows per 6"	SPT Blows/foot N	Boring and Sampling Notes
-0.2	0			1	6				2-0-1	1	2" Topsoil
		Brown, moist, very soft silty clay (CL) with organics									
-2.5		Brown, moist, very soft silt (ML) with traces of sand		2	13				W/H-W/H-1	1	
-5.0	5			3	14				1-2-4	6	
		Brown, moist, loose silty sand (SM)		4	13	22.6%	NP	NP	6-4-2	8	
-10.0	10										Boring terminated @10ft. depth.
	15										
	20										
	25										
	30										

SAMPLER TYPE
Driven Split Spoon unless otherwise noted
PT - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core

SAMPLE CONDITIONS
D - Disintegrated
I - Intact
U - Undisturbed
L - Lost

	GROUND WATER DEPTH		CAVE IN DEPTH	
At Completion	Dry	ft.	9	ft.
After 24 Hrs.	Dry	ft.		ft.
After ___ Hrs.		ft.		ft.

BORING METHOD
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
DC - Driving Casing
MD - Mud Drilling

STANDARD PENETRATION TEST-DRIVING 2" O.D. SAMPLER 1' WITH 140# HAMMER FALLING 30": COUNT MADE AT 6" INTERVALS.

HILLIS - CARNES
ENGINEERING ASSOCIATES, INC.

RECORD OF SOIL EXPLORATION

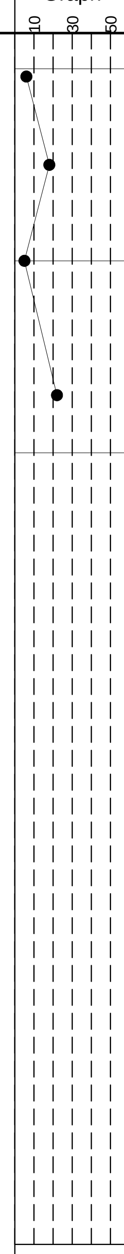
Project Name: **Marcey Road Park** Boring No. B3

Location: **2722 Marcey Rd, Arlington, VA 22207** Job # C20043

SAMPLER

Datum	OGL	Hammer Wt.	140 lbs.	Hole Diameter	in.	Foreman	James Burrowbridge
Surf. Elev. ±	ft.	Hammer Drop	30 in.	Rock Core Diameter		Inspector	FY
Date Started	5/7/2020	Pipe Size	2 in.	Boring Method	HSA-SPT	Date Completed	5/7/2020

Elev (ft)	Depth (ft)	Description	Soil Sym	Sample No	Sample Rec (in)	NM (%)	PI (%)	LL (%)	SPT blows per 6"	SPT Blows/foot N	Boring and Sampling Notes
-0.2	0										2" Topsoil Probable fill
		Brown, moist, medium stiff silty clay (CL)		1	10				2-2-4	6	
-2.5				2	10	28.7%	NP	NP	14-13-5	18	
	5	Brown, moist, loose to medium dense silty sand (SM)		3	12				2-2-3	5	
-8.5		Brown, moist, very stiff silt with sand (ML/SM)		4	18				13-9-13	22	
-10.0	10										Boring terminated @10ft. depth.
	15										
	20										
	25										
	30										



SAMPLER TYPE
Driven Split Spoon unless otherwise noted
PT - Pressed Shelby Tube
CA - Continuous Flight Auger
RC - Rock Core

SAMPLE CONDITIONS
D - Disintegrated
I - Intact
U - Undisturbed
L - Lost

	GROUND WATER DEPTH	CAVE IN DEPTH
At Completion	Dry ft.	9 ft.
After 24 Hrs.	Dry ft.	ft.
After ___ Hrs.	ft.	ft.

BORING METHOD
HSA - Hollow Stem Augers
CFA - Continuous Flight Augers
DC - Driving Casing
MD - Mud Drilling

STANDARD PENETRATION TEST-DRIVING 2" O.D. SAMPLER 1' WITH 140# HAMMER FALLING 30": COUNT MADE AT 6" INTERVALS.

KEY TO SYMBOLS

Symbol Description

Strata symbols

Symbol	Description
	Paving
	Topsoil
	Silt (ML)
	Silty sand (SM)
	Low plasticity clay (CL)
	Elastic silt/ sand (ML/SM)

Soil Samplers

Symbol	Description
	Auger

Notes:

1. Exploratory borings were drilled on 5/7/2020 using a 4-inches diameter continuous flight power auger.

2. No groundwater was encountered at the time of drilling or when re-checked the following day.

3. Boring locations were staked from existing features from the design schematic plan.

4. These logs are subject to the limitations, conclusions, and recommendations in this report.

5. Results of tests conducted on samples recovered are reported on the logs.

REFERENCE NOTES FOR BORING LOGS

I. Drilling and Sampling Symbols:

SS	Split Spoon Sampler	RB	Rock Bit Drilling
ST	Shelby Tube Sampler	BS	Bulk Sample of Cuttings
RC	Rock Core; NX, BX, AX	PA	Power Auger (no sample)
PM	Pressuremeter	HSA	Hollow Stem Auger
DC	Dutch Cone Penetrometer	WS	Wash Sample

Standard Penetration Test (SPT) resistance refers to the blows per foot (bpf) of a 140 lb hammer falling 30 inches on a 2 in. O.D. split-spoon sampler as specified in ASTM D-1586. The blow count is commonly referred to as the N-value.

II. Correlation of Penetration Resistances to Soil Properties:

Relative Density of Cohesionless Soils

SPT-N (bpf)	Relative Density
0 – 3	Very Loose
4 – 9	Loose
10 – 29	Medium Dense
30 – 50	Dense
>51	Very Dense

Consistency of Cohesive Soils

SPT-N (bpf)	Consistency
0 – 1	Very Soft
2 – 4	Soft
5 – 8	Firm
9 – 15	Stiff
16 – 30	Very Stiff
31 – 50	Hard
>51	Very Hard

Weathered Rock (WR) may be defined as SPT-N values exceeding 60 bpf depending on site specific conditions. Refer carefully to boring logs.

Rock Fragments, gravel, cobbles, boulders, or debris may produce N-values that are not representative of actual soil properties.

III. Unified Soil Classification Symbols:

GP – Poorly Graded Gravel
GW – Well Graded Gravel
GM – Silty Gravel
GC – Clayey Gravels
SP – Poorly Graded Sands
SW – Well Graded Sands
SM – Silty Sands
SC – Clayey Sands

ML – Low Plasticity Silts
MH – High Plasticity Silts
CL – Low Plasticity Clays
CH – High Plasticity Clays
OL – Low Plasticity Organics
OH – High Plasticity Organics
CL-ML – Dual Classification (Typical)

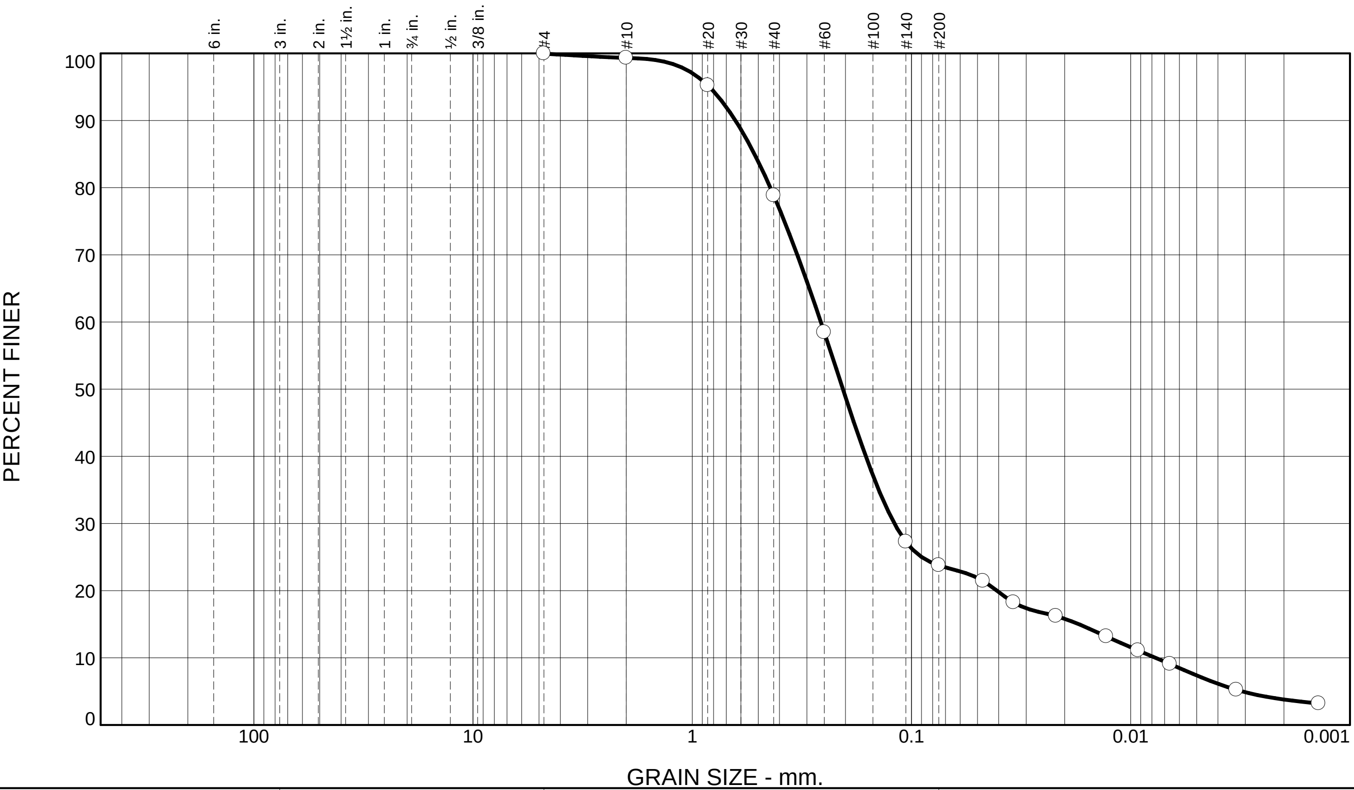
IV. Laboratory Testing and Water Level Symbols:

LL	- LIQUID LIMIT (%)
PI	- PLASTIC INDEX (%)
W	- MOISTURE CONTENT (%)
DD	- DRY DENSITY (PCF)
NP	- NON PLASTIC
-200	- PERCENT PASSING NO. 200 SIEVE
PP	- POCKET PENETROMETER (TSF)

▽	Water Level at Time Drilling, or as Shown
▼	Water Level at End of Drilling, or as Shown
▼	Water Level After 24 Hours, or as Shown

Appendix IV: Laboratory Results

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
			0.7	20.5	55.0	16.4	7.4

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#4	100.0		
#10	99.3		
#20	95.2		
#40	78.8		
#60	58.5		
#140	27.3		
#200	23.8		

* (no specification provided)

Material Description

Brown silty sand

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 0.6356 D_{85}= 0.5205 D_{60}= 0.2592
D_{50}= 0.2056 D_{30}= 0.1197 D_{15}= 0.0172
D_{10}= 0.0077 C_u= 33.60 C_c= 7.16

Classification

USCS= SM AASHTO= A-2-4(0)

Remarks

Mica very high
Natural moisture content: 13.0%

Location: 2722 N. Marcey Road, Arlington VA, Sample B-1
Sample Number: 810-1 **Date:** 05-12-20

HILLIS-CARNES ENGINEERING ASSOCIATES

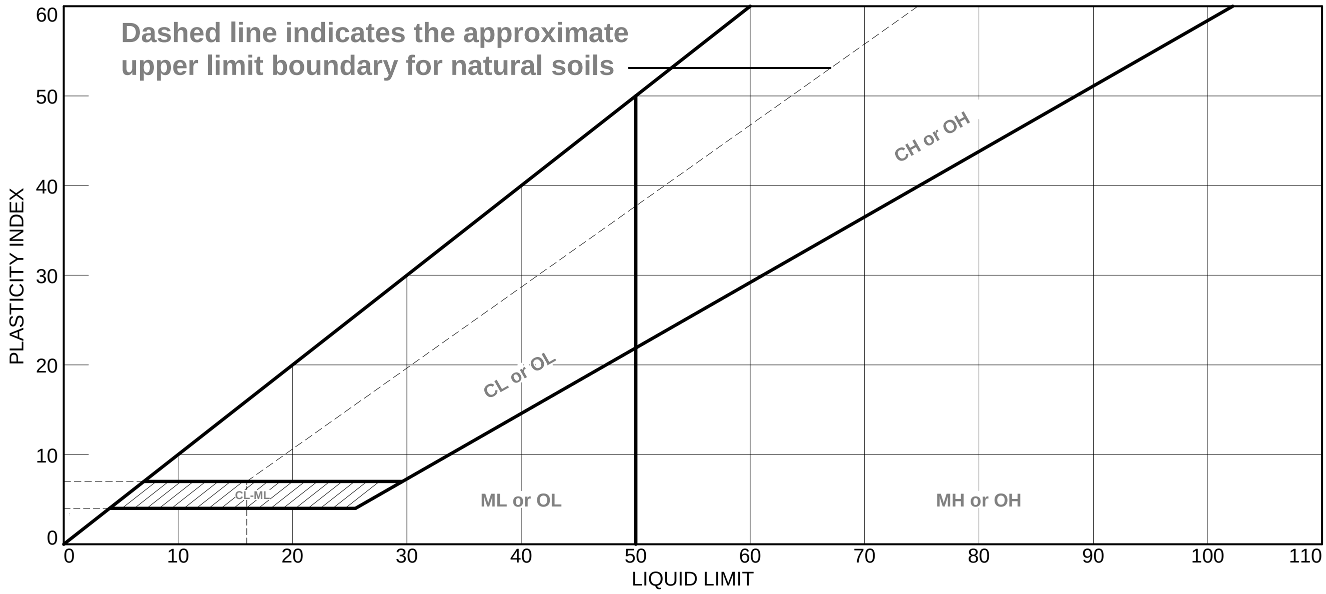
CHANTILLY, VA

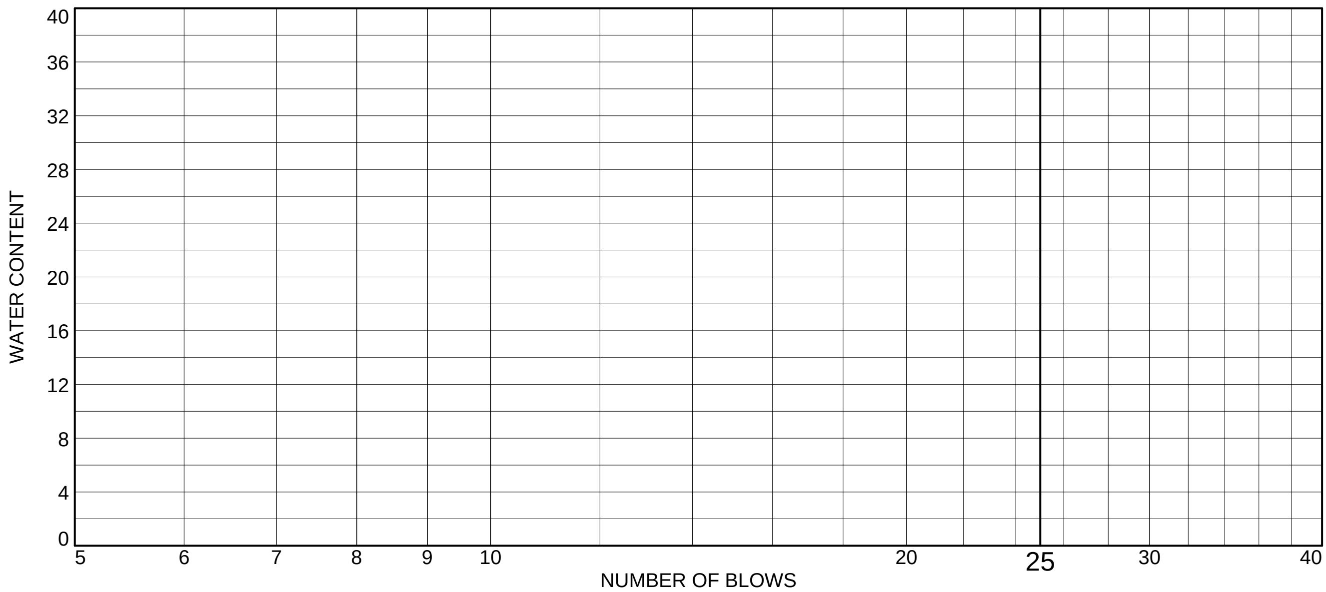
Client: Arlington County
Project: Marcey Road Park

Project No: C20043 **Figure**

Tested By: QD

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Brown silty sand	NV	NP	NP	78.8	23.8	SM

Project No. C20043 **Client:** Arlington County

Project: Marcey Road Park

Location: 2722 N. Marcey Road, Arlington VA, Sample B-1
Sample Number: 810-1

HILLIS-CARNES ENGINEERING ASSOCIATES

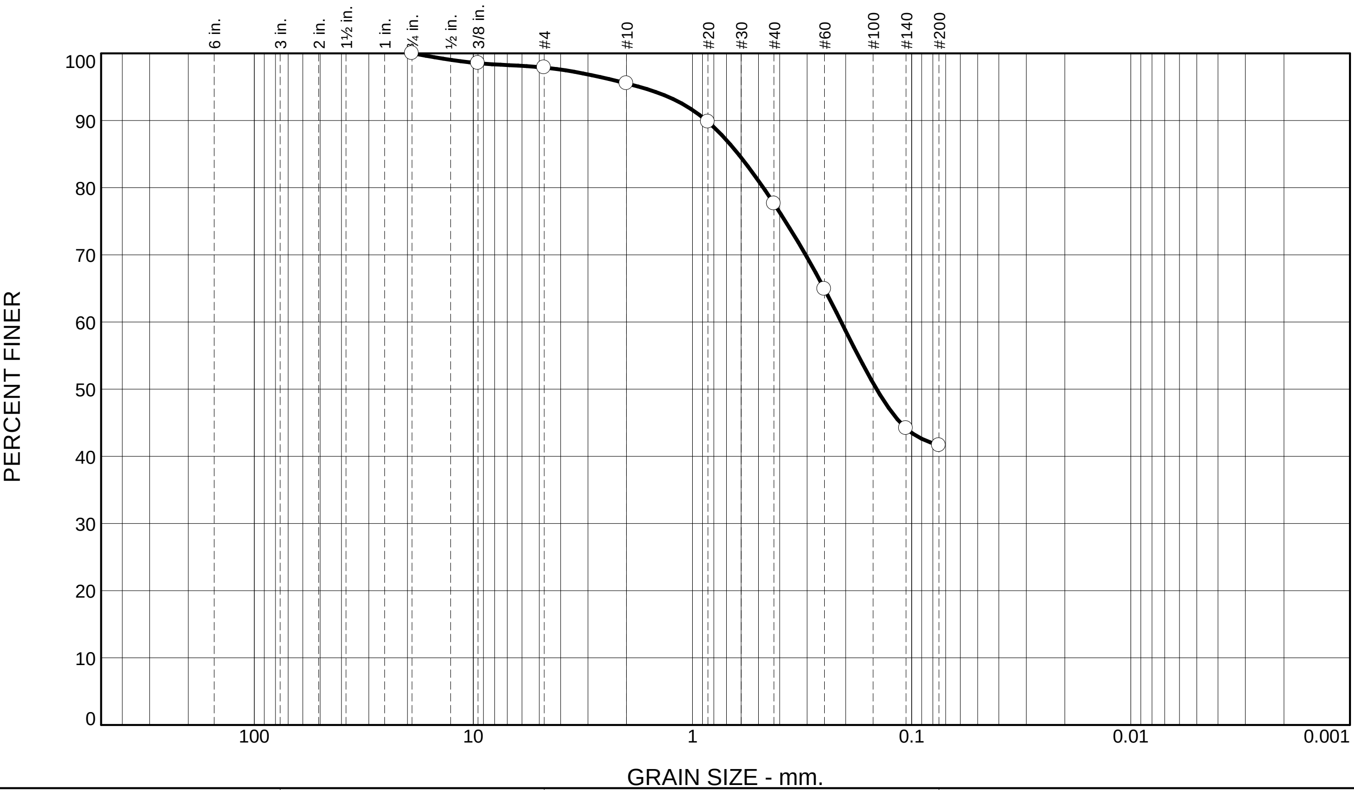
CHANTILLY, VA

Remarks:
●Mica very high

Figure

Tested By: QD

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	2.1	2.4	17.9	36.0	41.6	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
3/8"	98.5		
#4	97.9		
#10	95.5		
#20	89.8		
#40	77.6		
#60	64.9		
#140	44.2		
#200	41.6		

* (no specification provided)

Material Description

Brown silty sand

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 0.8644 D_{85}= 0.6178 D_{60}= 0.2097
D_{50}= 0.1445 D_{30}= D_{15}=
D_{10}= C_u= C_c=

Classification

USCS= SM AASHTO= A-4(0)

Remarks

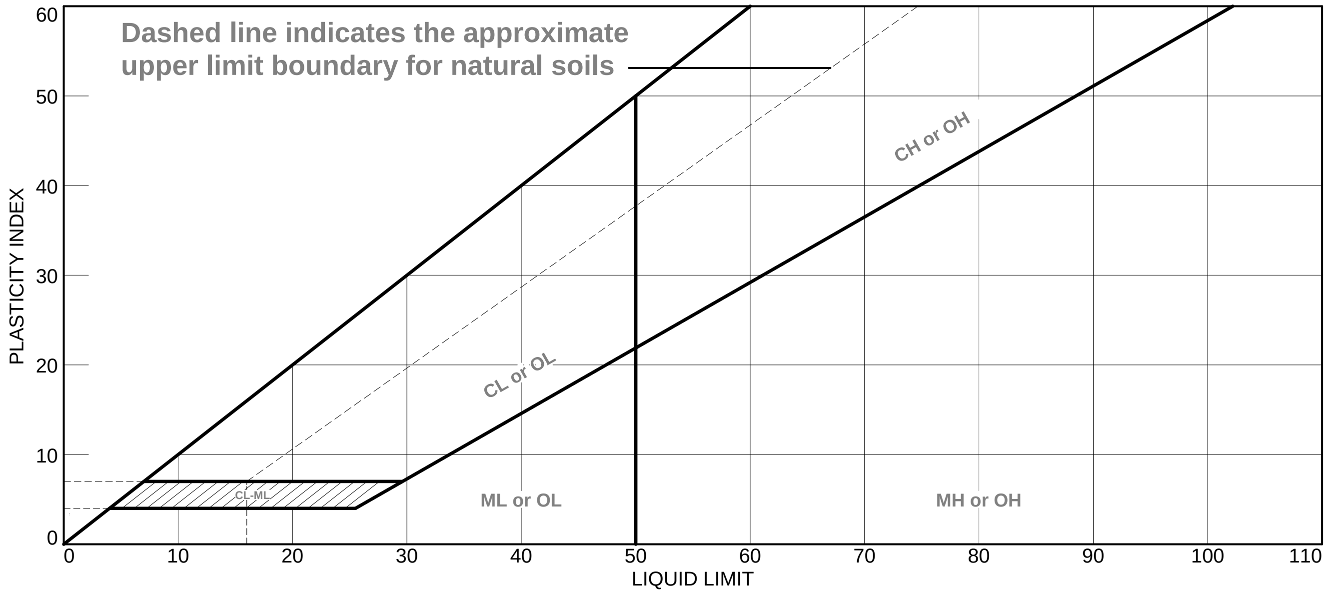
Mica very high
Natural moisture content: 22.6%

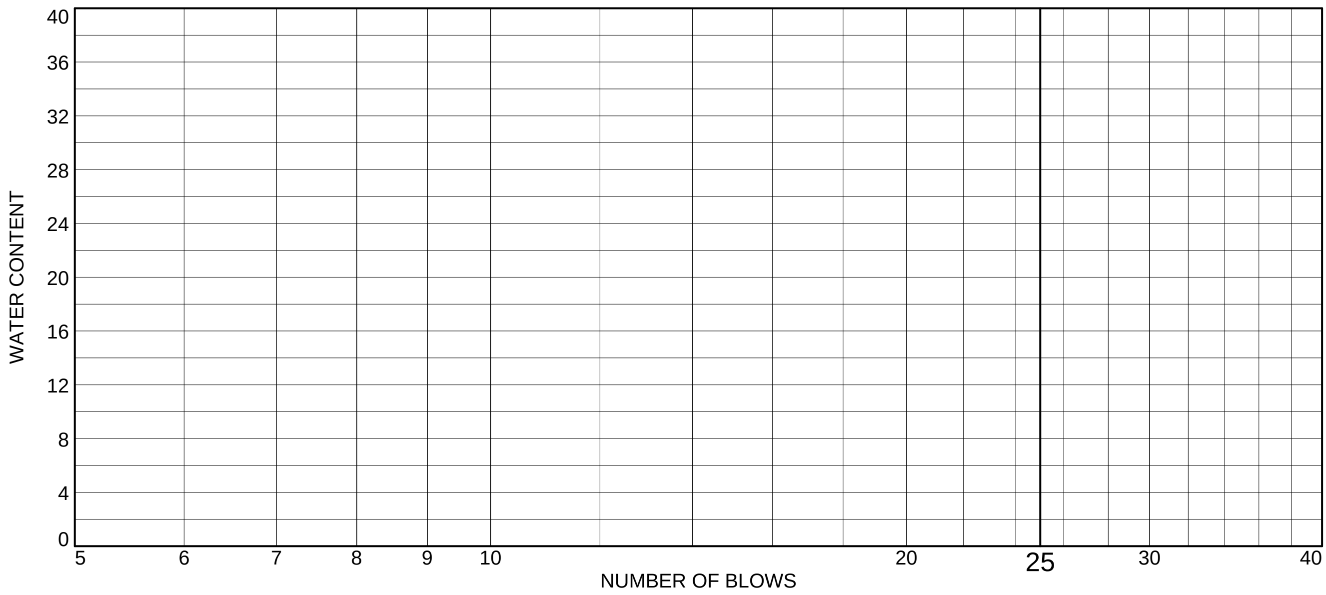
Location: 2722 N. Marcey Road, Arlington VA, Sample B-2
Sample Number: 810-2 **Date:** 05-13-20

HILLIS-CARNES ENGINEERING ASSOCIATES CHANTILLY, VA	**Client:** Arlington County **Project:** Marcey Road Park **Project No:** C20043 **Figure**

Tested By: QD

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Brown silty sand	NV	NP	NP	77.6	41.6	SM

Project No. C20043 **Client:** Arlington County

Project: Marcey Road Park

Location: 2722 N. Marcey Road, Arlington VA, Sample B-2
Sample Number: 810-2

HILLIS-CARNES ENGINEERING ASSOCIATES

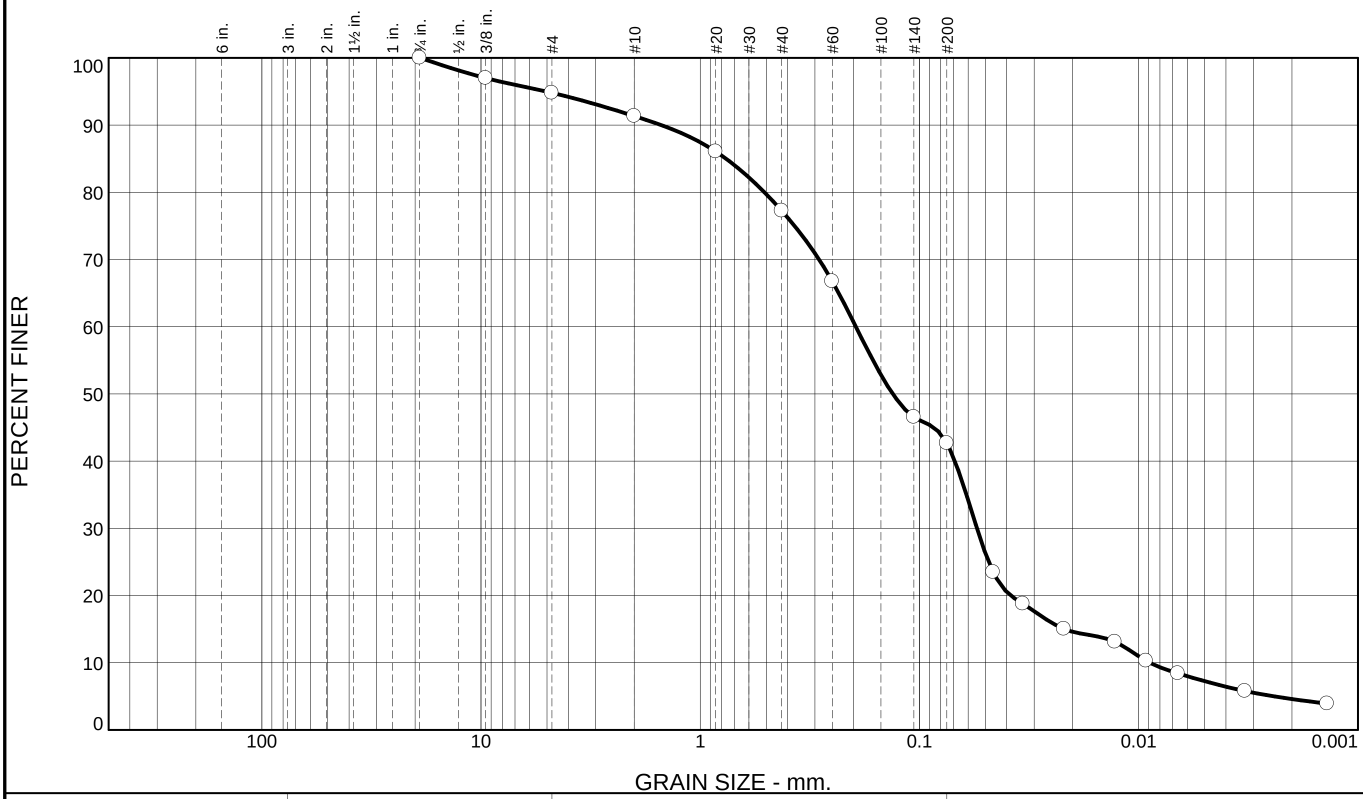
CHANTILLY, VA

Remarks:
●Mica very high

Figure

Tested By: QD

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0.0	0.0	5.2	3.5	14.1	34.5	35.4	7.3

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4"	100.0		
3/8"	97.0		
#4	94.8		
#10	91.3		
#20	86.0		
#40	77.2		
#60	66.7		
#140	46.5		
#200	42.7		

* (no specification provided)

Material Description

Dark brown silty sand

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{90}= 1.5083 D_{85}= 0.7638 D_{60}= 0.1952
D_{50}= 0.1324 D_{30}= 0.0547 D_{15}= 0.0218
D_{10}= 0.0089 C_u= 21.92 C_c= 1.72

Classification

USCS= SM AASHTO= A-4(0)

Remarks

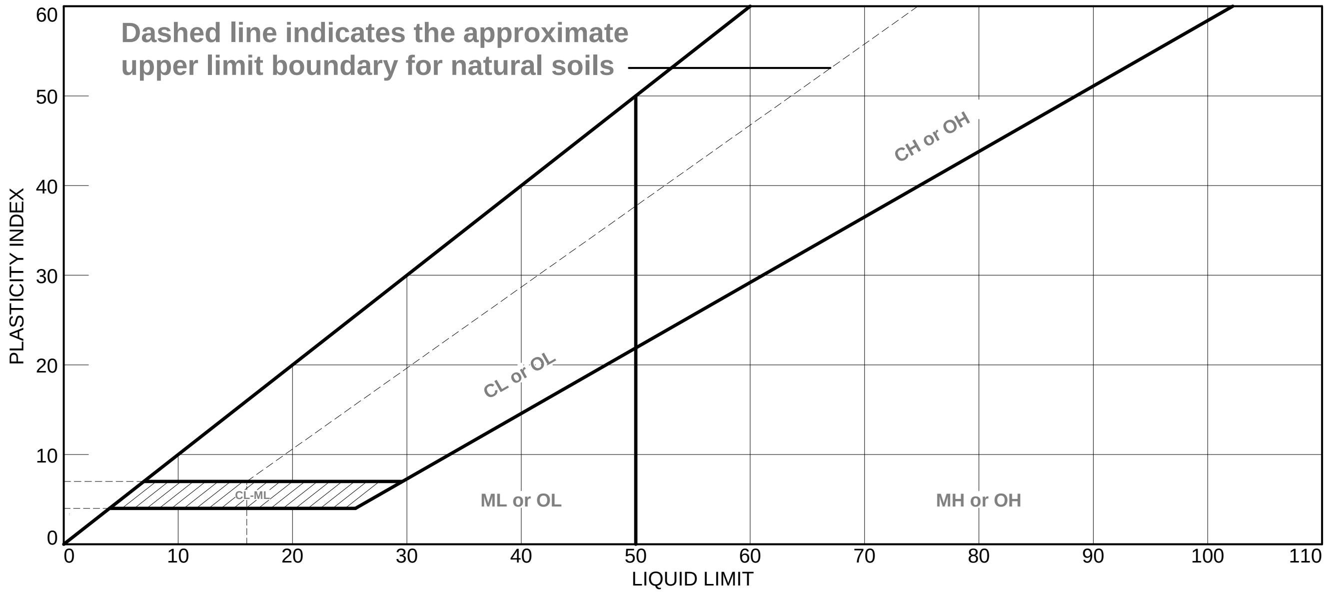
Mica very high
Natural moisture content: 28.7%

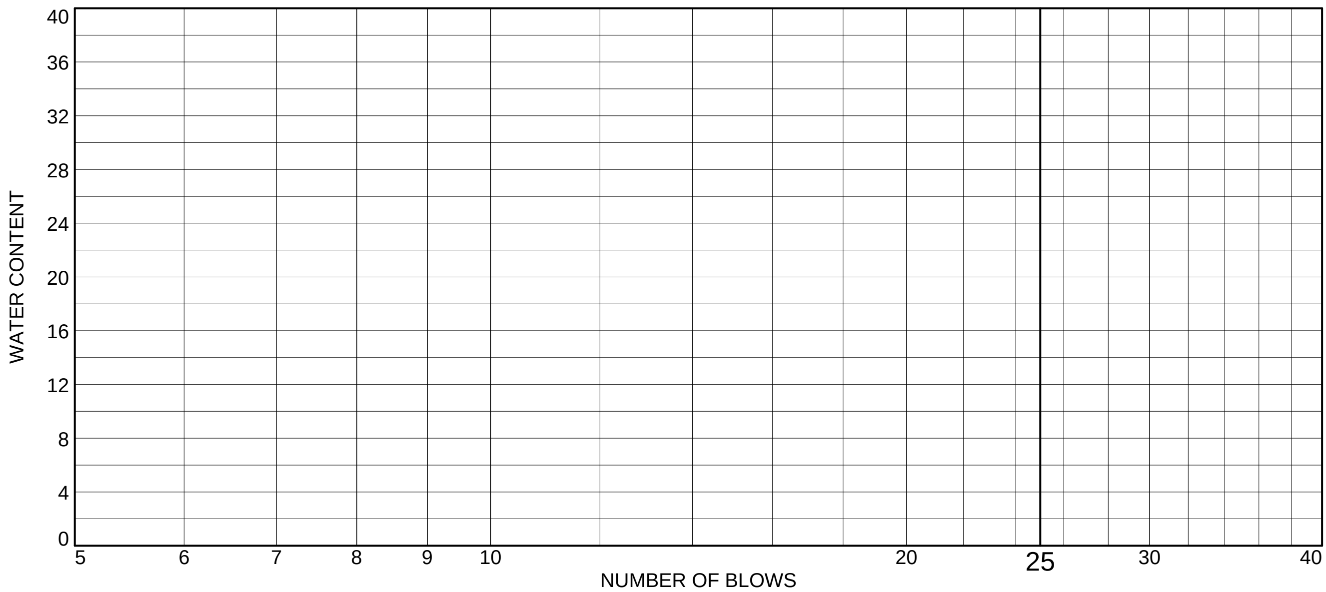
Location: 2722 N. Marcey Road, Arlington VA, Sample B-3
Sample Number: 810-3
Date: 05-13-20

HILLIS-CARNES ENGINEERING ASSOCIATES CHANTILLY, VA	**Client:** Arlington County **Project:** Marcey Road Park **Project No:** C20043 **Figure**

Tested By: QD

LIQUID AND PLASTIC LIMITS TEST REPORT





	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Dark brown silty sand	NV	NP	NP	77.2	42.7	SM

Project No. C20043 **Client:** Arlington County

Project: Marcey Road Park

Location: 2722 N. Marcey Road, Arlington VA, Sample B-3
Sample Number: 810-3

HILLIS-CARNES ENGINEERING ASSOCIATES

CHANTILLY, VA

Remarks:
●Mica very high

Figure

Tested By: QD

Appendix V: Infiltration Test Results

Test Location ID	**IT1**
Casing installed/pre-soak date	5/7/20
Test date	5/8/20
Ground surface elevation (GSE) (ft)	±6.00'
Test elevation (ft)	6" above grade
Total infiltration after pre-soak (inches)	24 inches (after 24 hrs of soaking)

Time	Reading (in)	Infiltration per hour (in/hr)	Remarks
10:11 AM	24.000	N/A	Start test
11:11 AM	21.500	2.500	
12:11 PM	19.000	2.500	
1:11 PM	17.000	2.000	
2:11 PM	13.500	3.500	Test complete

Average infiltration rate **2.625** **in/hr**

HILLIS CARNES ENGINEERING ASSOCIATES, INC.

14155 Sullyfield circle, Suite A
Chantilly, Virginia 20151
Phone (703) 817-1105, Fax (703) 817-1170

INFILTRATION TEST DATA

by: Mahboobullah Muradi

Test Location ID	**IT2**	
Casing installed/pre-soak date	5/7/20	
Test date	5/8/20	
Ground surface elevation (GSE) (ft)	±6.00'	
Test elevation (ft)	6" above grades	
Total infiltration after pre-soak (inches)	9" inches	(after 24 hrs of soaking)

Time	Reading (in)	Infiltration per hour (in/hr)	Remarks
2:50 PM	39.000	NA	Start test
3:50 PM	38.000	1.000	
4:50 PM	37.950	0.050	
5:50 PM	37.950	0.000	
6:50 PM	36.950	1.000	Test complete

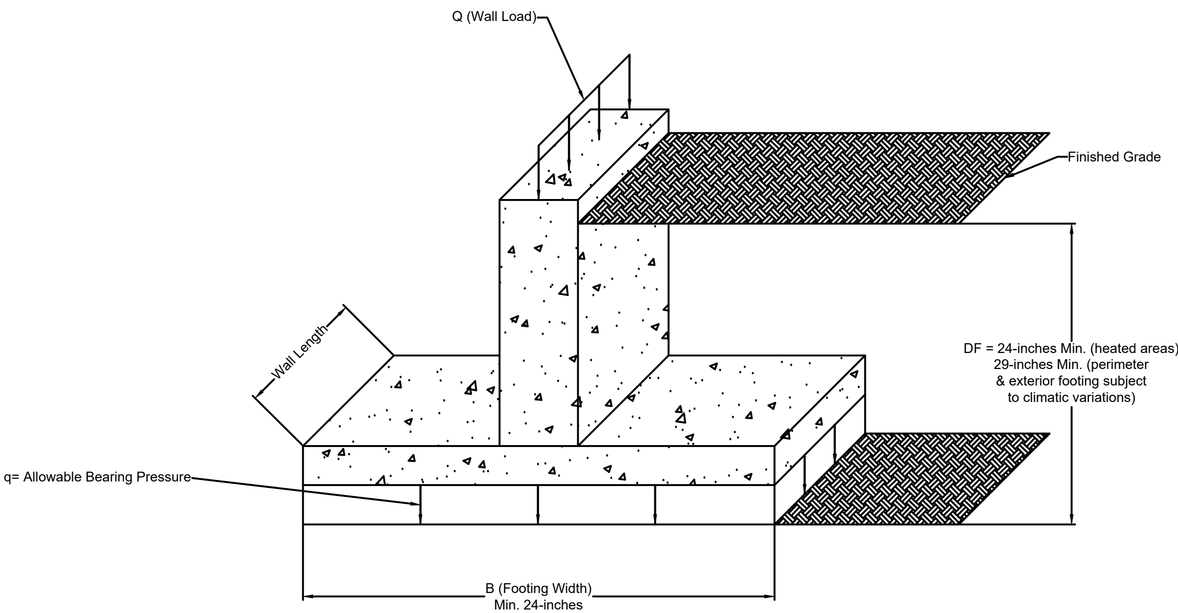
Average infiltration rate **0.512** **in/hr**

HILLIS CARNES ENGINEERING ASSOCIATES, INC.

14155 Sullyfield circle, Suite A
Chantilly, Virginia 20151
Phone (703) 817-1105, Fax (703) 817-1170

INFILTRATION TEST DATA

by: Mahboobullah Muradi



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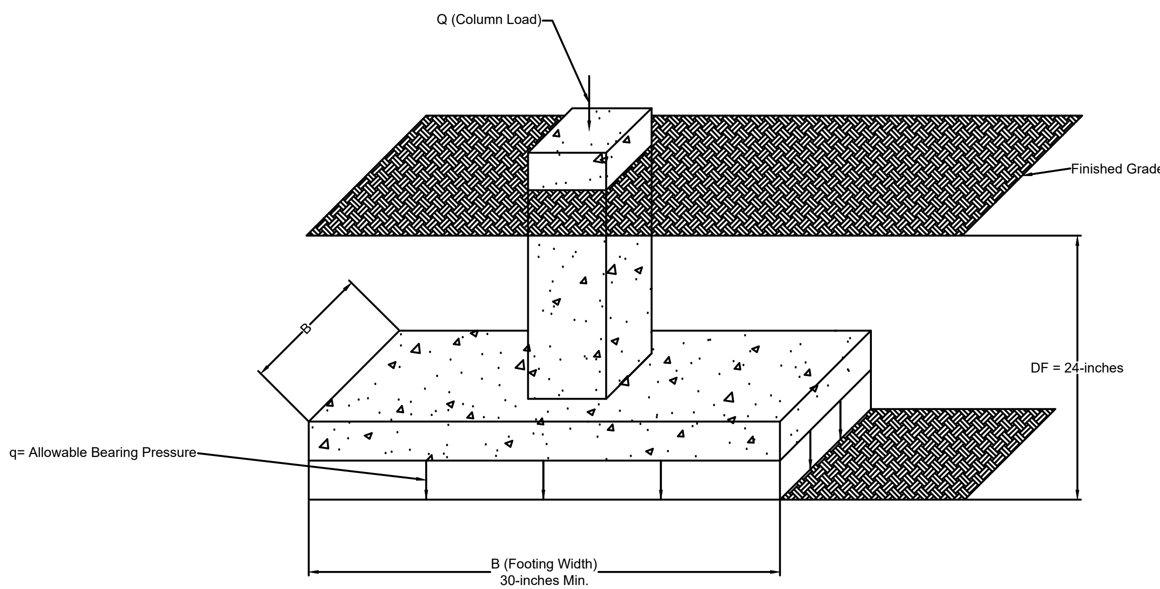

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TYPICAL FOUNDATION DETAIL

Marcey Road Park.
Arlington, VIRGINIA

PROJECT NO.	C20043	DESIGN BY:	
DATE:	5/22/20	DRAWN BY:	FY
SCALE:	NTS		
SHEET:	1 of 1	CHECKED BY:	FY

Important Information about Your

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.

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

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

Read the Full Report

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

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

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

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

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

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

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

Subsurface Conditions Can Change

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

Most Geotechnical Findings Are Professional Opinions

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

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

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

Do Not Redraw the Engineer's Logs

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

Give Contractors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

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

Geoenvironmental Concerns Are Not Covered

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

Obtain Professional Assistance To Deal with Mold

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

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

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

ASFE
THE BEST PEOPLE ON EARTH

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