DMY CAPITOL

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Geotechnical Engineering Report

Hearst Park and Pool
3950 37th Street NW
Washington, DC

Prepared for

Mr. Tom Wheeler
Cox, Graae and Spack Architects
December 21, 2017

DMYCAPITOL

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Cox, Graae and Spack Architects
2909 M Street NW
Washington, DC 20007

Reference: Geotechnical Engineering Report
Hearst Park and Pool
3950 37th Street NW
Washington, DC
DMY Project No. 02.02340.02

Dear Mr. Wheeler:

DMY Capitol, LLC (DMY) is pleased to submit this report of our geotechnical exploration and infiltration testing for the above-referenced project. This report presents a review of the information provided to us, a discussion of the site and subsurface conditions encountered, and our geotechnical recommendations.

We appreciate the opportunity to be of service to you on this project and would be happy to discuss our findings with you. We look forward to serving as your geotechnical engineer on the remainder of this project and on future projects.

Respectfully,

DMY CAPITOL, LLC

DISTRICT OF COLUMBIA
MARK E. CLIPPINGER
PE906794
CIVIL
PROFESSIONAL ENGINEER

Mark E. Clippinger, PE
Principal Engineer

Paul Zhang, PE
Partner

4400 MacArthur Boulevard NW, Suite 304, Washington, DC 20007 • www.dmycapitol.com • Phone: (202) 741-9159

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EXECUTIVE SUMMARY

This Executive Summary is provided a s brief overview of our geotechnical engineering evaluation of the project and is not intended to replace the detailed information included elsewhere in the report. This summary inherently omits important details that are vital to the proper application of the provided geotechnical design recommendations. This report should be read in its entirety prior to being implemented into design and construction.

- The planned park facility at Hearst Park will include a new playground area, new tennis courts, replacement of the existing soccer field, replacement of the existing tennis courts, a new outdoor pool and pool house. Based on the conceptual plans provided, the main level Finished Floor Elevation (FFE) for the planned pool house will be EL. 354.0 feet. As a result, excavation depths of 3 to 18 feet are anticipated for foundation construction. It is our understanding that the planned pool will be a lap type pool with a maximum depth of approximately 6 feet.

- The field exploration consisted of drilling a total of eleven (11) Standard Penetration Test (SPT) borings (B-1 through B-11) to depths of 20 to 30 feet to explore the subsurface soil and groundwater conditions at the planned tennis courts, pool and pool house, the north end of the existing soccer field and adjacent east-facing slope.

- The pool house building will retain approximately 16 feet of earth along the south and west walls. A retaining wall retaining approximately 16 feet of earth will be required along the west property line, north of the pool house. Because of the close proximity to the property lines, it will be required that temporary support of excavations (SOE) be installed for construction of the planned pool house. We recommend extending the SOE north of the pool house and incorporating the SOE into the design of the retaining wall. This will require that the SOE be designed to be a permanent retention system, likely incorporating tiebacks into the design.

- Structural loading for the pool house building was not available at the time this report was prepared; however, based on our experience with similar projects, a maximum interior column load of 50 kips and exterior wall load of 3 kips/lineal foot are expected. It is anticipated that the foundation system supporting the pool house building will bear on deep deposits of existing fill materials underlain by soft natural (residual) soils. In order to provide adequate support for the foundation systems, we recommend ground improvement in the form of rammed aggregate piers.

- A retaining wall will be required the entire length along the south side of the service drive lane, which will retain approximately 15 feet of earth. We anticipate this retaining wall will consist of either a segmental wall or concrete cantilever wall. Because of the proximity of the retaining wall to the south property line, it is anticipated that SOE will be needed along the south side of the site during construction of the retaining wall to prevent undermining of Quebec Street NW, or the sidewalk along the roadway. The SOE along the south property line will likely be designed to provide only temporary support.

- It is anticipated that the foundation system supporting the retaining wall located along the south side of the service drive lane will bear on deep deposits of existing fill materials underlain by soft natural (residual) soils. In order to provide adequate support for the foundation systems and provide adequate global stability for the planned retaining wall, we recommend ground improvement in the form of rammed aggregate piers.

- We anticipate that footings supported by the aggregate piers can be designed for allowable bearing pressures ranging from 4,000 to 6,000 per square foot (psf) depending on the spacing and number of piers below the footings.

- We understand during and after rainfall events, the local residents have observed water cascading down the concrete steps that lead up to the soccer field from the cul-de-sac at the end of Springland Lane NW. Based on our assessment, the potential for seepage on the east-facing slope at the east side of the existing soccer field is negligible. Reportedly, many of the storm sewer inlets at the east side of the soccer field were recently found to be completely covered by debris. The water observed cascading down the concrete steps and the east-facing slope is most likely associated with malfunctioning stormwater inlets and storm sewerage and is not the result of groundwater seepage.

1.0 PROJECT OVERVIEW

1.1. PROJECT INFORMATION AND SITE CONDITIONS

The project site is located at 3950 37th Street NW in Washington, DC. The site is bordered to the west by 37th Street NW, to the north by Phoebe Hearst Elementary School, to the south by Quebec Street NW, and to the east by Idaho Avenue NW and existing residential properties. A Site Location Map showing the approximate location of the project is included in Appendix A.

The existing park includes several recreational amenities such as tennis courts, a soccer field and rough trail. The site is terraced with moderately steep slopes around the perimeter of the soccer field and tennis courts. The existing topography around the soccer field ranges from a topographic high of EL 360 feet at the northwest corner to a topographic low of EL 340 feet at the southeast corner. Very large deciduous trees surround the soccer field. The existing topography around the tennis courts ranges from a topographic high of EL. 366 feet at the south and west sides to a topographic low of EL. 346 feet at the northeast corner. The tennis court area located south of the soccer field, slopes moderately to the north toward the soccer field.

The planned park facility at Hearst Park will include new tennis courts, replacement of the existing soccer field, replacement of the existing tennis courts, a new outdoor pool and pool house. Based on the conceptual plans provided, the main level Finished Floor Elevation (FFE) for the planned pool house will be EL. 354.0 feet. As a result, excavation depths of 3 to 18 feet are anticipated for foundation construction. It is our understanding that the planned pool will be a lap type pool with a maximum depth of approximately 6 feet deep. The pool house and bath house building will retain approximately 16 feet of earth along the south and west walls. A retaining wall retaining as much as approximately 16 feet of earth will be required along the west property line, north of the pool house. A retaining wall will also be required the entire length along the south side of the service drive lane, which will retain as much as approximately 15 feet of earth. We anticipate this retaining wall will consist of either a segmental wall or concrete cantilever wall. A conceptual plan showing the planned project is included as Figure 2 in Appendix A.

Structural loading for the planned pool house was not available at the time this report was prepared; however, based on our experience with similar projects, a maximum interior column load of 50 kips and exterior wall load of 3 kips/lineal foot are expected. The civil plans provided indicate stormwater management (SWM) facilities are proposed as part of the project. Specifically, a bio-retention filter is planned at the east side of the existing soccer field.

The description of the proposed project given above is based on the information provided to us by the project team and information gathered during our site reconnaissance. If any of the assumptions or project information is inaccurate or changed, DMY should be informed so that we may revise our geotechnical recommendations, if necessary.

1.2. SCOPE OF SERVICES

The purposes of this study were to obtain the subsurface soil and groundwater information for the proposed construction. Our scope of services included the following:

- Reviewing the project information provided to us.
- Obtaining the drilling permit from the Department of Consumer and Regulatory Affairs (DCRA) and the District Department of Energy and Environment (DOEE) for the subsequent field work.
- Drilling soil test borings at eleven (11) locations to evaluate the subsurface soil and groundwater conditions for the existing soccer field, proposed pool, pool house and retaining walls located at the south and west sides of the planned facility.
- Drilling boring and performing infiltration testing at one (1) location selected by the Project Civil Engineer.
- Performing laboratory tests on select soil samples.
- Evaluating field and laboratory data.
- Performing engineering calculations and analyses and preparing this geotechnical engineering report.

2.0 FIELD EXPLORATION AND LABORATORY TESTING

2.1. FIELD EXPLORATION

The field exploration consisted of drilling a total of eleven (11) Standard Penetration Test (SPT) borings (B-1 through B-11) to explore the subsurface soil and groundwater conditions at the planned tennis courts, pool and pool house, the north end of the existing soccer field and adjacent east-facing slope. The SPT borings were also drilled to explore the subsurface conditions within the footprint of the planned pool/pool house and retaining walls along the south and west sided of the pool area and tennis courts. With the exception of borings B-4 through B-6, which were drilled to depths of 20 feet below existing grades, all borings were drilled to depths of 30 feet below existing grades. Profile boring P-1 was drilled at the location of the planned bio-retention filter located at the east side of the existing soccer field. In close proximity to the profile boring, auger probe I-1 was also drilled to facilitate in-situ infiltration testing. The boring locations were chosen and located in the field by DMY personnel based on visual reference to the existing site features. The infiltration test location was selected by the civil engineer and located in the field by DMY personnel. The approximate locations of the borings and infiltration tests are shown on the Boring and Infiltration Location Plan included in Appendix A.

The SPT borings were drilled using a track-mounted CME-55 drill rig using the hollow stem auger method, mud rotary and casing. Groundwater levels were measured in each boring during and upon completion of drilling. Long-term groundwater monitoring was performed in borings B-1, B-2, B-3, B-7 and profile boring P-1. Upon completion of the field exploration, all boreholes were backfilled with grout in accordance with DOEE requirements. The field exploration procedures are included in Appendix B.

After drilling to the required depth in the profile boring, a section of perforated PVC pipe was inserted into the open hole to prevent caving and allow long-term groundwater monitoring. PVC pipe was inserted into the bottom of the infiltration auger probe to prevent caving. The bottom 2-feet of the PVC pipe placed in the infiltration boring was perforated to allow adequate flow into the surrounding soils during infiltration testing. Constant head borehole infiltration tests were conducted in each infiltration test boring using an automated permeameter device.

Following field operations, the soil samples were transported to our laboratory for further analysis and testing. The samples will be stored in our laboratory for a period of two weeks from the submittal date of this report. After this period, the samples will be discarded unless we are instructed otherwise.

2.2. LABORATORY TESTING

Representative soil samples were selected and tested in our laboratory to verify field classifications and to determine pertinent engineering properties. The laboratory testing program included the following:

- Visual Classification (ASTM D 2488) 82 Tests
- Natural moisture Content (ASTM D 2216) 8 Tests
- Grain size analysis (ASTM D 422) 6 Tests
- Atterberg Limits (ASTM D 4318) 6 Tests

The laboratory testing results are included in Appendix C of this report.

3.0 SITE GEOLOGY AND SUBSURFACE CONDITIONS

3.1. SITE GEOLOGY

According to the Geologic map of the Washington West Quadrangle, District of Columbia, Montgomery and Prince George's Counties, Maryland, and Arlington and Fairfax Counties, Virginia (1994), the surface geology at this site is mapped as the Georgetown Intrusive Suite of early Ordovician age and Soapstone and talc-bearing schist of Cambrian or Late Proterozoic age.

The member of the Georgetown Intrusive Suite is mapped as the Biotite-hornblende tonalite. It consists of medium to coarse-grained massive to foliated rock that has a strong relic igneous flow structure at many places. Unit contains many ultramafic and mafic xenoliths and/or autoliths, and xenoliths of metasedimentary rocks. Typically, it contains 40-50 percent dark minerals and contains small layers of biotite tonalite at many places.

The Soapstone and talc-bearing schist consists of fine to coarse-grained dark grayish-green soapstone and talc-chlorite-actinolite (carbonate) schist and fels. At many places it is associated with Actinolite schist.

3.2. SUBSURFACE CONDITIONS

The subsurface conditions encountered at the locations explored are shown in the boring logs in Appendix B. The records represent our interpretation of the subsurface conditions in accordance with generally accepted geotechnical engineering practice. The lines designating the interfaces between various strata on the boring logs are approximate, as the actual transitions between soil strata are often gradual. In the absence of foreign substances, it is difficult to distinguish between natural soils and clean soil fills. Although individual test borings are representative of the subsurface conditions at the precise boring locations on the dates shown, they are not necessarily indicative of the subsurface conditions at other locations or at other times.

Surficial Materials
Approximately 2 to 3 inches of topsoil was encountered at the existing surface in borings B-1, B-2, B-10, B-11 and P-1. The topsoil consisted primarily of sandy silt with varying amounts of organics and small roots. Asphalt and concrete pavement (with aggregate base at some locations) was encountered in borings B-4, B-5, B-7, B-8 and B-9, which were drilled at the existing tennis courts. Approximately 5 inches of asphalt pavement (without aggregate base) was encountered at the existing ground surface in boring B-6, which was drilled at the existing tennis practice wall court. No tree roots were encountered (detectable) during drilling in any of the borings.

Strata F1, Apparent Existing Fine-Grained Fill
Apparent fine-grained fill materials were encountered immediately below the surface or tennis court pavements in all of the borings, extending to depths of 1.5 feet to 20.0 feet below existing grades (termination depths in borings B-4 and B-5). The fine-grained fill materials encountered classified as sandy LEAN CLAY (FL-CL), sandy SILT (FL-ML), and sandy SILT with gravel (ML). Debris consisting or plastic, asphalt and coal fragments was encountered within the existing fill materials. SPT N-

values of 2 to 28 blows per foot (bpf) were recorded in these fill materials, indicating a soft to very stiff consistency. No compaction records were available for review and, consequently, the existing fine-grained fill materials are considered uncontrolled.

Strata F2, Apparent Coarse-grained Fill

Apparent coarse-grained fill materials were encountered immediately below the fine-grained fill materials in boring B-5 from a depth of 17.0 feet to the termination depth of 20.0 feet below current site grades. The coarse-grained fill materials classified as silty SAND with gravel (FL-SM). An SPT N-value of 26 bpf was recorded in the coarse-grained fill materials, indicating a medium dense relative density. No compaction records were available for review and, consequently, the existing coarse-grained fill materials are considered uncontrolled.

Strata R1, Fine-grained Residual Soils

Fine-grained residual soils were encountered below the existing fill soils in all of the borings, with the exception of borings B-4 and B-5, to depths of 17.0 to 30.0 feet below existing grades. The fine-grained residual soils classified as sandy SILT (ML), sandy ELASTIC SILT (MH) and LEAN CLAY (CL). SPT N-values of 2 to 59 bpf were recorded, indicating soft to very hard consistencies.

Strata R2, Coarse-grained Residual Soils

Coarse-grained residual soils were encountered in boring B-6 at depths of 17.0 to 20.0 feet below existing grades and in boring B-8 at depths of 13.0 to 18.0 feet below existing grades. The coarse-grained residual soils classified as a silty SAND (SM) and silty SAND with gravel (SM). SPT N-values of 8 to 11 bpf were recorded, indicating loose to medium dense relative densities.

Groundwater

Groundwater was encountered during drilling in all of the borings with the exception of borings P-1, B-2, and B-3. Groundwater was monitored in several of the borings for a period of 24 hours subsequent to drilling. Please refer to the table below summarizing the groundwater conditions encountered/observed during this investigation. It should be noted that groundwater levels fluctuate with seasonal and climatic variations and may be different at other times and locations than those stated in this report.

Boring Location	Depth/Elevation of Groundwater During Drilling (ft)	Depth/Elevation of Groundwater Observed 24 hours Subsequent to Drilling
B-1	20.0/319.0	12.0/329.0
B-2	Not encountered	23.0/316.0
B-3	Not encountered	28.0/292.0
B-4	14.0/339.0	Not monitored*
B-5	15.0/338.0	Not monitored*

Boring Location	Depth/Elevation of Groundwater During Drilling (ft)	Depth/Elevation of Groundwater Observed 24 hours Subsequent to Drilling
B-6	13.0/340.0	Not monitored*
B-7	22.0/333.0	11.0/343.5
B-8	14.0/340.0	Not monitored*
B-9	22.0/333.0	Not monitored*
B-10	22.0/348.0	Not monitored*
B-11	13.0/355.0	Not monitored*
P-1	Not encountered	Dry

* Because casing and mud rotary drilling methods were used, long-term groundwater monitoring was not possible.

4.0 GEOTECHNICAL RECOMMENDATIONS

4.1. BUILDING FOUNDATION DESIGN

Based on the conceptual plans provided, the Finished Floor Elevation (FFE) of the pool house building will be EL. 354.0 feet. Slab-on-grade construction is planned for the building. The pool house building will retain approximately 16 feet of earth along the south and west walls. As a result, excavation depths anticipated for foundation construction will vary from 16 feet along the south and west sides of the pool house building to a depth of 3 feet at the east side of the building.

The results of the subsurface exploration indicate the foundation system supporting these structures will bear on deep deposits of existing fill materials underlain by soft natural (residual) soils. Based on the anticipated structural loading and our engineering analyses, we recommend the planned pool house building be supported on shallow foundation systems consisting of spread and/or continuous footings with ground improvement in the form of rammed aggregate piers.

Rammed aggregate piers should improve a portion of the subgrade soils below the footings and are constructed using either augers or mandrels to drill into the ground. The resulting excavation is then replaced with aggregate material. The aggregate material is typically either vibrated, or compacted in place. The design and construction of aggregate pier ground improvement systems should be completed by a specialty contractor. The contractor will ultimately provide the foundation design bearing pressures and anticipated settlement as well as prepare drawings and specifications for the aggregate piers. Local contractors include GeoStructures (Geopiers™) in Leesburg, Virginia, TerraSystems in Purcellville, Virginia, and Hayward Baker in Odenton, Maryland.

Although the design-build specialty contractor will provide the required drawings and analyses, we anticipate that the rammed aggregate piers will be required below all wall and column footings. We do not believe that aggregate piers are required below the finished floor slab. Once the drawings and calculations are completed, we recommend that the design team review the drawings and specifications to insure any conflicts do not exist. In addition, we recommend that a minimum of two (2) rammed aggregate piers be load tested to verify the design assumptions used by the design-build contractor. Production piers should be installed using the same equipment, depths, and methods used for the tested piers.

Based on the anticipated structural loads, the rammed aggregate piers are expected to have a diameter on the order of 18 to 36 inches and be spaced approximately 7 to 8 feet apart. We anticipate the aggregate piers will extend approximately 10 to 15 feet below existing grades. It is anticipated groundwater will be encountered, therefore the contractor should be prepared for groundwater intrusion during installation. The contractor should consider the groundwater table in selecting the aggregate pier installation method and design. Wall footings may require additional reinforcing steel to allow the stresses from the footing to span between individual aggregate piers.

We anticipate that footings supported by the aggregate piers can be designed for allowable bearing pressures ranging from 4,000 to 6,000 per square foot (psf) depending on the spacing and number of piers below the footings. The final allowable bearing pressures should be determined by the design/build contractor. The allowable bearing pressures should be based on a minimum safety factor of 3.0 against bearing failure. Design settlement should be limited to one inch of total settlement and ½ inch of differential settlement.

During construction an authorized representative of the Geotechnical Engineer of Record should be present to witness the installation and load testing of the aggregate piers, and the construction of footings. At a minimum, the depths, stone type, lift thickness, and condition at the top of the aggregate piers should be evaluated. Additional testing requirements for the aggregate piers will be provided by the design-build contractor.

In order to prevent disproportionately small footing sizes, we recommend that continuous footings have a minimum width of 16 inches and that isolated column footings have a minimum lateral dimension of 24 inches. The minimum dimensions recommended above help reduce the possibility of foundation bearing failure and excessive settlement due to local shear or "punching" action. All footings should be placed at a minimum depth of 30 inches below finished exterior grade to provide adequate frost cover protection acceptable for this region.

Once the final design drawings are completed, it is recommended that the geotechnical engineer review the plans to ensure the recommendations presented in this report remain applicable. Depending on the final design, additional borings may be required to provide sufficient subsurface information.

4.2. SLAB-ON-GRADE

We anticipate that the floor slab of the planned pool house building can be supported on the natural and existing fill soils, if the fill is free from excessive debris. The floor slab does not need to be supported on aggregate piers. Undercut of unsuitable and soft soils on the order of 2 feet deep should be expected in isolated areas on the site. A modulus of subgrade reaction, k, of 90 psi/inch may be used in the design of floor slabs.

The floor slab should be isolated from the footings so that differential settlement of the structure will not induce stress on the floor slab. In order to minimize the crack width of any shrinkage cracks that may develop near the surface of the slab, we recommend that mesh reinforcement be included in the design of the grade slabs. The mesh should be in the top half of the slab to be effective.

Grade slabs should have a minimum thickness of four inches. A minimum six-inch thick washed gravel or crushed stone (No. 57 aggregate or equivalent) should be placed below the grade slabs to provide uniform bearing support, a capillary break, and drainage of any moisture accumulation. A minimum 10-mil thick impermeable plastic membrane should be installed over the gravel layer to serve as a vapor barrier to prevent transmission of water through the slab.

4.3. BELOW GRADE FOUNDATION WALLS

Design Parameters

Although the planned pool house building features slab-on-grade construction, the south and west walls of the building will serve as full depth basement walls/retaining walls. The basement level foundation walls should be designed to withstand lateral earth pressures and surcharge loads. We recommend that the following parameters be used for the basement wall design:

• Unit Weight of Soil Backfill	120 pcf
• Equivalent At-rest Fluid Pressure	60 pcf

The above recommended soil parameters assume that the foundation wall backfill will consist of properly compacted silty SAND (SM) or more granular natural or clean fill soils. The existing fill materials and natural soils should not be re-used as wall backfill due to debris, high plasticity and high moisture content. The recommended equivalent fluid pressures assume that constantly functioning drainage systems are installed between the walls and the soil backfill to prevent any accidental buildup of hydrostatic pressures. The wall design should also account for any surcharge loads within a 45-degree slope from the base of the wall.

For the basement walls, since the top of the walls will be laterally supported by the floor or roof diaphrams, active earth pressure conditions are not likely to develop in the soil backfill behind the walls. Therefore, we recommend that the at-rest pressures be used in the design of the below grade walls. Fill must not be placed behind the basement walls until the floor structure connecting to the top of walls has been constructed and thereby the earth pressure can be transferred to the floor structure.

The lateral pressures exerted on the south and west foundation walls may result in excessive sliding forces on the pool house building. The east foundation walls may need to be extended deeper than standard frost penetration depth in order to increase the passive earth pressure for sliding resistance.

All foundation wall backfills should be placed in accordance with the Compacted Fills section of this report. Heavy earthwork equipment should maintain a minimum horizontal distance away from the foundation walls of one foot per foot of vertical wall height. Lighter compaction equipment should be used close to the foundation walls.

Foundation Drainage

Proper drainage measures should be provided to minimize any hydrostatic pressure build-up (from groundwater and/or infiltration) behind the walls. Adequate drainage can be accomplished if a blanket of select granular backfill, such as VDOT No. 57 aggregate, is used behind the walls. To prevent migration of fines into the select granular backfill, a layer of filter fabric should be installed around the select granular backfill where it comes in contact with the general wall backfills. The filter fabric should have an apparent open size (AOS) of no greater than 0.21 mm (#70 sieve). Geocomposite drainage panel may be used in lieu of the select granular backfill adjacent to the walls. Examples of the geocomposite drainage materials include Enkadrain®, MiraDRAIN®, and Geotec drains. The select granular backfill or geocomposite drainage panel should be extended from the bottom to approximately two feet below the final grade behind the walls. Clayey material may be placed in the upper two feet to reduce the amount of surface water infiltration into the drainage system. The ground surface adjacent to the below grade walls should be kept properly graded to prevent ponding of water adjacent to the walls.

For the south and west foundation walls, we recommend that a perimeter drainage system be provided. This system may consist of perforated drainage pipes located around the perimeter of the below grade area, slightly above the footing grade. The perimeter drain lines should be surrounded by a minimum of six inches of drainage gravel wrapped in filter fabric, and should be either gravity drained to daylight or connected to a permanent sump to remove any water accumulation. All below grade walls should be water-proofed to minimize the migration of water through below grade walls. A Below Grade Drainage Detail is included in Appendix A of this report.

4.4. RETAINING WALLS

Design Parameters

A retaining wall retaining as much as approximately 16 feet of earth will be required along the west property line, north of the pool house. Because SOE will be required during construction along the entire south and west sides of the site to prevent potential undermining of the adjacent roadways and sidewalks, we recommend designing the SOE retaining system north of the planned pool house as a permanent retaining system. This will likely require that tiebacks be used.

A retaining wall will also be required the entire length along the south side of the service drive lane, which will retain as much as approximately 15 feet of earth. We anticipate this retaining wall will consist of either a segmental wall or concrete cantilever wall.

Based on the soil conditions encountered in the borings, we recommend that the following parameters be used for design of the retaining walls:

- Unit Weight of Soil Backfill: 120 pcf
- Coefficient of Sliding Friction: 0.3
- Equivalent Active Fluid Pressure: 43 pcf
- Equivalent Passive Fluid Pressure*: 330 pcf*
- Allowable Bearing Pressure** 4,000 to 6,000 psf

* In the design calculations, the resisting forces computed using the above recommended passive earth pressure coefficient and equivalent passive fluid pressure, should be reduced using a safety factor of 1.5.

** This applies to the segmental or concrete cantilever retaining wall along the south side of the service drive lane, which is supported by rammed aggregate piers. The rammed aggregate piers should be designed by a qualified specialty contractor. Refer to Section 4.1 of the report for additional requirements.

The above recommended soil parameters assume that the foundation wall backfill for the retaining wall along the south side of the service drive lane will consist of properly compacted silty SAND (SM) or more granular natural or clean fill soils. The existing fill materials contain debris and are not considered suitable for reuse. The natural soils encountered at this site do not meet the criteria for structural fill presented in the Compacted Fills section of this report, due to the high moisture content, high liquid limit and/or plasticity index. The recommended equivalent fluid pressures assume that constantly functioning drainage systems are installed between the walls and the soil backfill to prevent any accidental buildup of hydrostatic pressures. The wall design should also account for any surcharge loads within a 45-degree slope from the base of the wall.

Active earth pressure conditions apply to relatively flexible earth retention structures, such as freestanding walls, where some movement and rotation may occur to mobilize soil shear strength. This will likely be the case for the proposed site retaining walls.

All wall backfills should be placed in accordance with the Compacted Fills section of this report. Heavy earthwork equipment should maintain a minimum horizontal distance away from the walls of one foot per foot of vertical wall height. Lighter compaction equipment should be used close to the walls.

Subsurface Drainage for Retaining Walls

Proper drainage measures should be provided to minimize any hydrostatic pressure build-up (from groundwater and/or infiltrating rain water) behind the concrete retaining walls. Adequate drainage can be accomplished if a blanket of select granular backfill, such as VDOT No. 57 aggregate, is used behind the walls. To prevent migration of fines into the select granular backfill, a layer of filter fabric should be installed around the select granular backfill where it comes in contact with the general wall backfills. The filter fabric should have an apparent open size (AOS) of no greater than 0.21 mm (#70 sieve). The select granular backfill or geocomposite drainage panel should be extended from the bottom to approximately two feet below the final grade behind the walls. Clayey material can be placed in the upper two feet to reduce the amount of surface water infiltration into the drainage system. The ground surface adjacent to the below grade walls should be kept properly graded to prevent ponding of water adjacent to the walls.

For retaining walls, we recommend that a perforated collector pipe be installed at the base of the walls to drain water using gravity from the drainage blanket behind the wall to daylight. The collector pipe should be surrounded by a minimum of six inches of drainage gravel wrapped in filter fabric. Alternatively, weep holes may be provided for the retaining walls every eight feet with outlet at a height of six inches above the ground surface in front of the wall.

Global Stability Analysis for Retaining Walls

At the time we performed our global stability analyses, the project included two (2) optional layouts. A retaining wall was required along the west side of the site for one of the layouts. A retaining wall was required along the south side of the service drive lane at the south side of the site in both layouts. To analyze the worst case scenario for global stability, we assumed that concrete cantilever walls will be utilized along both the south and west sides of the site. When the final layout was selected by the design team, it was determined that a post and panel type retaining wall would be required along the west side of the site since a temporary construction easement for construction of a cantilever or segmental wall along the west property line will most likely not be granted by DDOT.

We recommend extending the support of excavation needed for the pool house building north and incorporate the SOE into the design of the retaining wall. This will require that the SOE north of the pool house building be designed to be a permanent retention system, likely incorporating tiebacks into the design. It is our understanding that tiebacks are allowed to encroach onto DDOT right-of-ways, as long as they do not damage or impact any existing underground utilities. A stone or brick fascia can be constructed to conceal the lagging and piles comprising the retaining wall and provide the architectural finish desired. Based on our experience, an experienced specialty contractor should be able to design the retaining wall.

The global stability analysis for the planned retaining wall located at the south side of the park was performed using a two-dimensional computerized slope stability method based on a limit equilibrium analysis. The SLIDE 7.017 computer program was utilized to perform these computations. The factor of safety against slope instability computed by the program is defined as the ratio of the sum of the moments (or forces) resisting failure divided by the sum of the moments (or forces) causing failure along a specified potential failure surface. Hence, a factor of safety greater than 1.0 indicates a marginally stable slope, while a factor of safety less than 1.0 indicates a potentially unstable or failed slope. During this analysis, numerous conditions and potential failure surfaces were analyzed; however, the computer outputs included in the attachment show only the most critical conditions.

We had already performed global stability analysis for retaining walls along both the south and west sides of the site when the final layout for the park was determined. In order to avoid confusion, we have only included the global stability analysis for the south side of the service drive lane in this report. Along the planned service drive lane located at the south side of the park, we assumed the retaining wall required will retain approximately 15 feet of earth. We first analyzed the short term, undrained condition. We also analyzed the long term, drained condition. Analyzing the global stability of the walls with the soils in their present condition, the Factors of Safety were well below the acceptable value of 1.3, as indicated below. Consequently, we analyzed a zone below the retaining wall footing that was improved with rammed aggregate piers, a common method of improving the soil conditions below retaining walls. Because we recommend using rammed aggregate piers to improve the bearing conditions below the planned pool house, we also analyzed the long-term drained condition at both sections, assuming rammed aggregate piers will be used to improve the subsurface conditions below the retaining wall. In our analysis, we assumed the rammed aggregate piers would be installed below the entire width of the footing, or a minimum of 10 feet wide and a minimum of 10 feet below bearing elevation. It was also assumed that the aggregate piers will comprise a minimum of 20% of the improved soil mass below the retaining wall footing.

Results of the global stability analyses are summarized in the tables below.

Analysis Scenario	Factor of Safety
Undrained Conditions-Short Term Stability	1.1
Drained Conditions-Long Term Stability	1.5
Undrained Conditions- Short Term Stability with Rammed Aggregate Piers	1.3

For both short term and long term stability, a minimum Factor of Safety of 1.3 is required for the retaining walls. The results of our global stability analysis indicate a minimum Factor of Safety of less than 1.3 for the existing soils in the undrained condition (short term). The results of our global stability analysis indicate a minimum Factor of Safety exceeding 1.3 for the existing soils in a drained condition (long term). Consequently, with Factors of Safety less than 1.3, the undrained condition requires mitigation. For mitigation against failure, as described above, we used rammed aggregate piers for ground improvement below the retaining wall. Using rammed aggregate piers for ground improvement as shown on the sections, the Factors of Safety met the minimum 1.3 requirement. The results of the global stability analyses are included in Appendix D.

4.5. HIGHLY PLASTIC SOILS

Highly plastic ELASTIC SILT (MH) soils were encountered at initial depths of 8 to 13 feet and extended to depths of 12 to 18 feet below existing grades in borings B-6 and B-9 during our subsurface exploration. These soils are common in this geology and will likely be present at unexplored areas of the site. Highly plastic soils can develop significant shrink/swell problems and should not be used as structural fill if encountered during construction.

If highly plastic soils are encountered at or below the foundation bearing level, the footings should have a minimum embedment depth of 4 feet below the finished exterior grade. The foundations may either step down to bear at 4 feet below finished exterior grade, or the footings may be undercut to a depth of 2 feet and backfilled with engineered fill to the original bearing elevation. Undercutting of the footings and backfilling with gravel is not recommended as this would create a reservoir condition which could saturate the plastic soils.

If highly plastic soils are encountered within 2 feet below the ground slab subgrade elevation, they should be undercut to a depth of 2 feet, or the thickness of the high plasticity soils, whichever is less. The undercut area should then be backfilled using engineered fill placed in accordance with the recommendations contained within the Compacted Fills section of this report.

4.6. SEISMIC DESIGN

The seismic site class and design response spectrum were determined in accordance with the procedures outlined in Section 1613 of the 2012/2015 International Building Code (IBC). Section 1613 of IBC outlines the procedures for seismic site classification, determination of maximum considered earthquake ground motion, and computation of design spectral response accelerations for various site classes. The current code site class definitions available range from A (hard rock) to F (very soft soil profile). Based on the analyses of the subsurface profile, using standard penetration data (SPT) and our local experience, we recommend a seismic Site Class “E” (Soft Clay Soil) be used for this site. Based on this site class, the design spectral response accelerations were computed as follows:

Short Period Acceleration, S_{DS} = **0.198 g**
1-second Period Acceleration, S_{D1} = **0.119 g**

4.7. NEW POOL

It is our understanding that the planned “lap” type pool will have a maximum depth approximately 6 feet. As a result, the bottom of the pool will be at elevation EL. 348.0 feet. Based on the subsurface conditions encountered in our borings, the pool is expected to bear on natural (residual) soils. Soft soil conditions are expected to be encountered at slab bearing elevation in the pool area, but due to the relatively light loads, rammed aggregate piers are not required for ground improvement below the pool.

Groundwater was encountered during drilling at depths of 22 and 14 feet in borings B-7 and B-8, respectively, which corresponds to EL. 333.0 and 340.0 feet, respectively. Groundwater was observed at a depth of 11 feet in boring B-7, 24 hours subsequent to drilling, which corresponds to EL. 343.5 feet. Based on a depth of 6 feet, it is not expected that groundwater conditions will be present above the bottom of the pool. Consequently, the pool does not need to be designed to resist the potential buoyancy forces associated with groundwater.

4.8. SOCCER FIELD AND ADJACENT SLOPE

We understand during and after rainfall events, the local residents have observed water cascading down the east-facing slope at the east side of existing soccer field and the concrete steps that lead up to the soccer field from the cul-de-sac at the terminus of Springland Lane NW. Borings B-1 and B-2 were drilled to depths of 30 feet at the north end of the soccer field. Boring B-3 was drilled to a depth of 30 feet in the existing gravel trail located on the slope at the east side of the soccer field. The purpose of these borings was to investigation the subsurface conditions and determine if the groundwater conditions are contributing to the water observed flowing down the slope.

For reference, we have indicated the locations of borings B-1, B-2 and B-3 on the Historic Cut and Fill Plan developed by Stantec. The depth of fill encountered in the borings is relatively close to fill depths indicated on the Stantec plan. We also included the depth of groundwater observed 24 hours subsequent to drilling in each of the borings. The Historical Cut and Fill Plan is included as Figure 4 in Appendix A.

As presented in the summary table on Pages 8 and 9 of this report, groundwater was observed during drilling and 24 hours subsequent to drilling in the borings B-1, B-2 and B-3. Twenty-four hours subsequent to drilling, groundwater was observed at elevation EL. 329.0 feet in boring B-1, EL. 316.0 feet in boring B-2 and EL. 292.0 in boring B-3. The existing topography on the Wiles Mensch plan indicates the existing grade is EL. 307.0 feet in the front yard of the house located at the south side of the cul-de-sac of Springland Lane NW. Consequently, groundwater is projected to be approximately 20 feet below grade at this location, or approximately 12 feet below a potential basement FFE. It should be noted that boring B-3 was drilled during a period of recent significant precipitation. Subsurface profile B-B', using the subsurface information obtained from borings B-1, B-2 and B-3, is included as Figure 5 in Appendix A.

Based on our assessment, the potential for seepage on the east-facing slope at the east side of the existing soccer field is negligible. Reportedly, many of the storm sewer inlets at the east side of the soccer field were recently found to be completed covered by debris. The water observed cascading down the concrete steps and the east-facing slope is most likely associated with malfunctioning stormwater inlets and storm sewerage and is not the result of groundwater seepage.

4.9. INFILTRATION TESTING

Infiltration testing was performed at a depth of 6 feet at one (1) location at the east side of the existing soccer field selected by the Project Civil Engineer. The constant head method was used as required by DOEE. Based on the field infiltration test results, the calculated infiltration rates for soils at each test location are summarized in the following table:

Test Location	Field Infiltration Rate (in./hr.)	Soil Classification at Infiltration Stratum	
		USCS	USDA
I-1	0.75	Sandy Silt Fill (ML)	Silt Loam

5.0 CONSTRUCTION RECOMMENDATIONS

5.1. SITE PREPARATION

After demolition and removal of the existing tennis courts, subgrade preparation operations should consist of removing existing underground utilities, topsoil, and any other soft or unsuitable material from the proposed building and pavement areas. The resulting excavations should be brought back to proposed elevations using structural fill placed as detailed herein. Utilities such as pipes should be removed entirely or abandoned by filling the pipe with grout to prevent future migration of soils into the pipe.

An authorized representative of the Geotechnical Engineer of Record should be present on-site working with the contractor to aid in determination of the required depth of undercut and to observe and evaluate the exposed subgrades. The preparation of fill subgrades and the proposed building subgrades should be observed on a full-time basis. Soil bridging lifts should not be used to span over soft fill subgrade soils within the building footprint. All soft areas shall be excavated and removed.

5.2. COMPACTED FILLS

Based on the subsurface conditions observed in our exploration, the existing fill materials and natural soils should not be re-used as engineered fill due to debris, high plasticity and high moisture content. All engineered fills including wall backfill should have a Liquid Limit less than 40 and a Plasticity Index less than 15. Before field operations begin, a representative sample of each proposed engineered fill should be collected and tested to determine its Atterberg Limits, gradation, maximum dry density, optimum moisture content, and natural moisture content. The test results will be used to evaluate the suitability of each proposed engineered fill for quality control purposes during fill placement.

Engineered fill materials should be placed in lifts not exceeding eight inches in loose thickness and moisture conditioned to within two percentage points of the optimum moisture content. The engineered fill should be compacted to a minimum of 95% of the maximum dry density obtained in accordance with ASTM Specification D-698, Standard Proctor Method. The top one foot of soil supporting pavements, sidewalks, or gutters should be compacted to a minimum of 100% of the maximum dry density in accordance with ASTM Specification ASTM D-698, Standard Proctor Method.

All fill operations should be observed on a full-time basis by an authorized representative of the Geotechnical Engineer of Record to determine that compaction requirements are being met. All fill shall be periodically tested to confirm that compaction is being achieved. A sufficient number of tests shall be taken in each lift before the next lift is placed, on the order of at least three tests per lift. The elevation and location of the tests should be clearly identified and recorded at the time of fill placement.

5.3. FOUNDATION

All foundation excavations should be sloped or stepped back in accordance with Occupational Safety and Health Administration (OSHA) regulations for excavations. Exposure to the environment may weaken the soils at the footing bearing level if the foundation excavations remain open for too long a time. Therefore, foundation concrete should be placed the same day that excavations are made. 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 3-inch thick "mud mat" of "lean" concrete be placed on the bearing soils before the placement of reinforcing steel.

The Geotechnical Engineer of Record should document the type and competency of the soils exposed with those documented in the nearby hand auger probes. Any significant difference should be brought to the attention of the owner along with recommendations by the Geotechnical Engineer of Record.

5.4. SLAB-ON-GRADE

The slab subgrade should be proofrolled using heavy equipment such as a loaded dump truck during construction. As an option to proofrolling methods, we recommend that at least one DCP test be performed for every 100 square feet of floor area to document the subgrade strength. DCP resistance of 6 blows per increment or higher will verify adequate stability of the slab subgrade. Floor slab subgrades should be observed by an authorized representative of the Geotechnical Engineer of Record prior to placing the compacted fill or crushed stone base. Any soft, loose, or unsuitable soils should be removed and replaced with compacted fill or crushed stone. Compacted fill should consist of soil classified as SM or more granular per ASTM D-2487. The existing fill may meet this criteria, but may contain excessive debris. The clays present at this site should not be used for structural fill or backfill materials.

Fill should be placed in six-inch loose lifts and compacted to at least 95 percent of the maximum dry density per ASTM D-698. Gravel size larger than three inches in diameter should not be used as engineered fill below the slabs.

The Geotechnical Engineer of Record should document the type of soils exposed and compare the competency of the soils exposed with those documented in this subsurface exploration. Any significant difference should be brought to the attention of the owner along with recommendations by the Geotechnical Engineer of Record.

5.5. WATER CONTROL DURING CONSTRUCTION

It is not anticipated that the permanent groundwater table at the site will be encountered above the design subgrade levels during construction of the pool house building. Groundwater may be encountered during excavation for the pool, depending on the depth of the pool. In addition, excavations performed at this site may encounter perched groundwater conditions or surface water flowing from the higher elevations of the site. We anticipate that some localized areas within the excavations may not be completely dry and may require the use of trenches and sump pits to facilitate the placement of foundations. Although a totally dry subgrade should not be anticipated, the surface of the subgrade should be sufficiently dewatered to provide an adequate surface on which to construct the footings and grade slabs.

The surface of the site should be properly graded to keep drainage of the surface water away from the proposed construction areas. The actual extent of the dewatering system will need to be determined at the time the excavation is performed.

5.6. EXCAVATION

A support-of-excavation system will likely be used along the west side of the new pool house building to facilitate foundation construction. Depending on which option is selected, support-of-excavation systems will also likely be used along the south and west sides of the new park to facilitate construction of the new retaining walls. The design of a support-of-excavation system is beyond the scope of this report and is best performed by a specialty contractor. We anticipate that the support-of-excavation system will likely consist of soldier piles and wood lagging at the alleyway area. Soldier piles are typically spaced at 8 to 10 feet on center, and should have a minimum toe embedment of 5 feet below the bottom of the lowest excavation, including over-excavation if required. The final design of the system should include an overall (global) stability analysis in addition to the internal stability analysis. The soldier beam and wood lagging retention system should be freely draining, and designed to withstand lateral earth pressures. The influence of any surcharge loads should also be considered.

The Contractor should avoid stockpiling any excavated materials or equipment immediately adjacent to the excavation walls or slopes. All such materials should be kept back from the top of the excavation a minimum distance equal to the excavation depth. Where equipment or materials must be placed immediately adjacent to the excavation walls, the excavation walls and slopes should be designed for the anticipated surcharge loading, or additional bracing must be provided to support the anticipated surcharge loading. In addition, the earth retention system design should consider surcharge loads from cranes and other construction equipment during construction as well as buildings. Based on the boring data, we recommend the following parameters for design of the support of excavation system:

The support of excavation designer should also include any specific testing requirements. The installation of the temporary earth retention system should be observed on a full-time basis by the Geotechnical Engineer of Record, or his authorized representative, in accordance with the current jurisdiction requirements.

5.7. MONITORING

If used, the driving of the piles for the support of excavation is anticipated to generate vibrations. Vibration monitoring should be performed once pile driving commences. We recommend that threshold on vibrations be provided by the structural engineer.

We also recommend that the owner commission the performance of a pre-construction survey on the adjacent structures. It has been our experience that such pre-construction surveys can usually help prevent frivolous claims as a result of pre-existing damages that were not apparent to nearby property owners until they began to observe their building following the construction of adjoining properties. We recommend that the owners or property managers be invited to accompany the engineering crews on the survey of the building, and to receive a copy of the survey. Naturally, if there is any damage to the nearby buildings, this survey can be beneficial in helping develop an equitable resolution.

We recommend that any existing structure where underpinning is required be monitored using a three-dimensional monitoring program and the above pre-construction. The 3-D monitoring program will be used to monitor the performance of the underpinning contractor and any below grade retention system adjacent to existing structures. Allowable movement tolerances should be set to prevent damage to the existing structure. Baseline measurements and subsequent movement evaluation should be performed on at least a twice weekly basis during excavation of the underpinning pits and below grade levels, and then weekly until construction reaches the ground floor or until the structural engineer indicates that the below grade level will resist the lateral earth pressures.

We recommend that the monitoring data be transmitted verbally to the general contractor and the specialty contractor's representative while the crew is in the field, particularly if any significant vibrations or movement is observed. We also strongly recommend that the monitoring data be provided to the Structural Engineer and all interested party on a weekly basis or sooner if threshold values are approached or exceeded.

6.0 LIMITATIONS

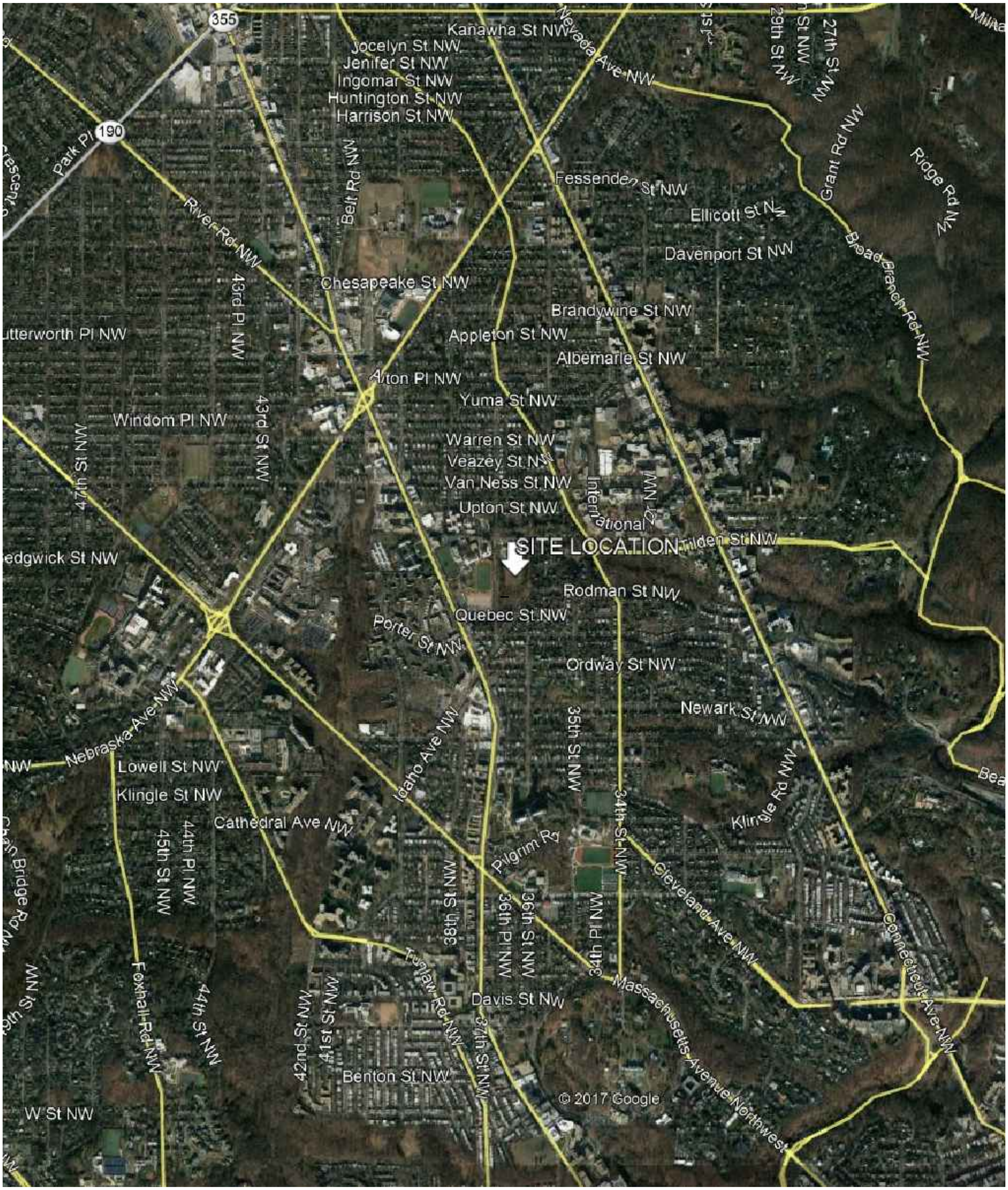
The recommendations provided are based in part on project information provided to us and are only applied to the specific project and site discussed in this report. If the project information section in this report contains incorrect information or if additional information is available, DMY should be contacted to review our recommendations. We can then modify our recommendations for the proposed project.

Regardless of the thoroughness of a subsurface investigation, there is always a possibility that subsurface conditions may vary from those documented during a subsurface exploration at specific locations. In addition, the construction process itself may alter subsurface conditions. Therefore, experienced geotechnical personnel should be engaged to observe and document the construction procedures used and the conditions encountered. Unanticipated conditions and inadequate procedures should be reported to the design team along with timely recommendations. We recommend that DMY be retained to provide this service based upon our familiarity with the project, the subsurface conditions, and the intent of the recommendations.

We have prepared this report for use by the design professionals for design purposes in accordance with generally accepted geotechnical engineering practices. No other warranty, expressed or implied, is made as to the professional advice included in this report.

APPENDIX A FIGURES

N



SITE LOCATION MAP

DMYCAPITOL

DMY CAPITOL, LLC
4400 MACARTHUR BLVD NW, SUITE 304
WASHINGTON, DC 20007
PHONE: (202) 741-9159
FAX: (301) 768-4169

HEARST PARK AND POOL
3950 37TH STREET NW
WASHINGTON, DC

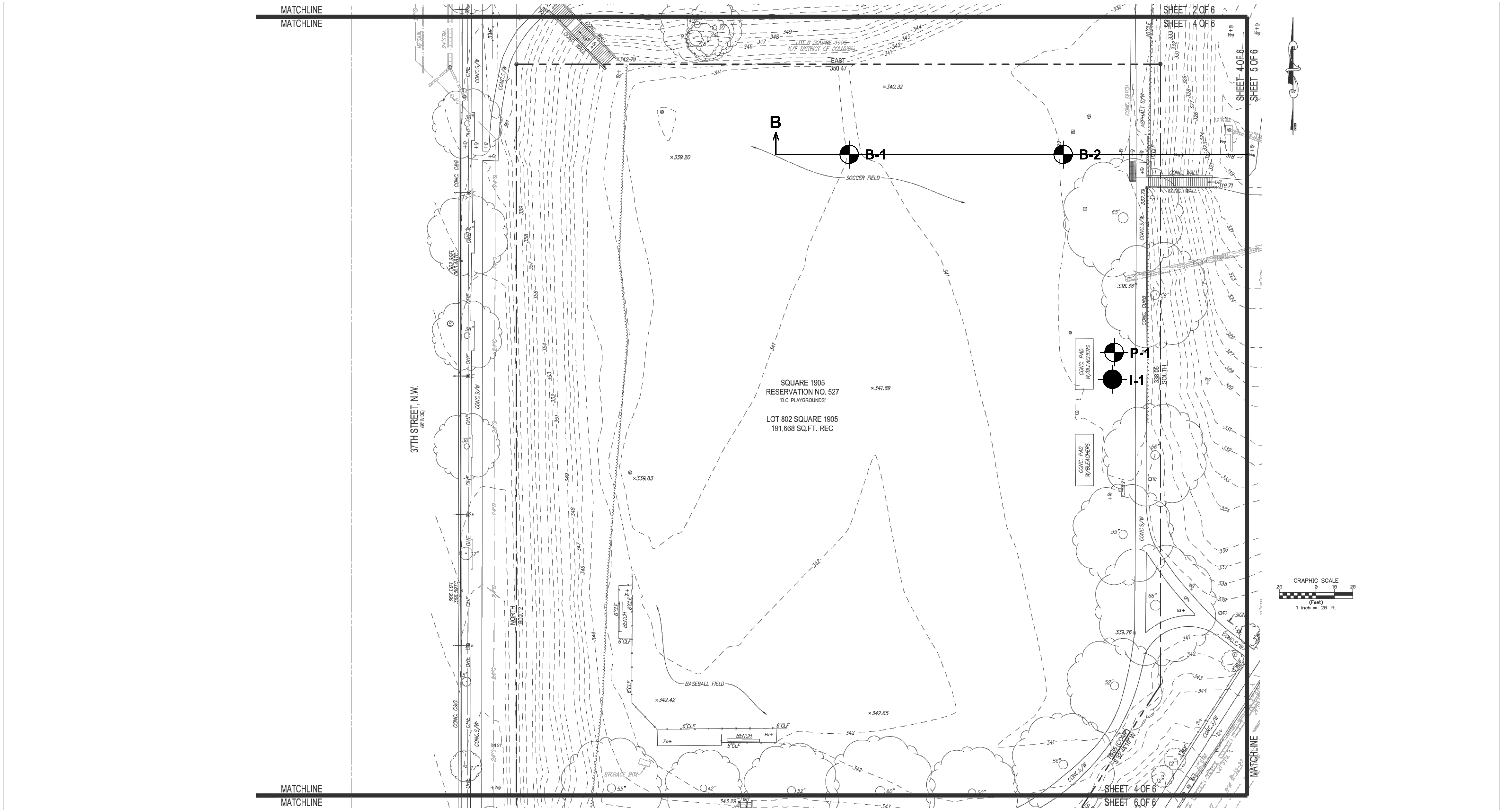
DATE:	DRAFTED BY:	PROJECT NO.:
10/25/17	SG	02.02340.02
SCALE:	CHECKED BY:	FIGURE NO.:
1"=2000'	MC	1

BASE DRAWING PROVIDED BY COX GRAAE + SPACK ARCHITECTS

CONCEPT PLAN	DMY CAPITOL DMY CAPITOL, LLC 4400 MACARTHUR BLVD NW, SUITE 304 WASHINGTON, DC 20007 PHONE: (202) 741-9159 FAX: (301) 768-4169	HEARST PARK AND POOL 3950 37TH STREET NW WASHINGTON, DC

DATE:	DRAFTED BY:	PROJECT NO.:
12/07/17	SG	02.02340.02
SCALE:	CHECKED BY:	FIGURE NO.:
AS SHOWN	MC	2

BASE DRAWING PROVIDED BY WMC-DC



APPROXIMATE BORING LOCATION

APPROXIMATE INFILTRATION LOCATION

BORING/INFILTRATION AND CROSS SECTION LOCATION PLAN

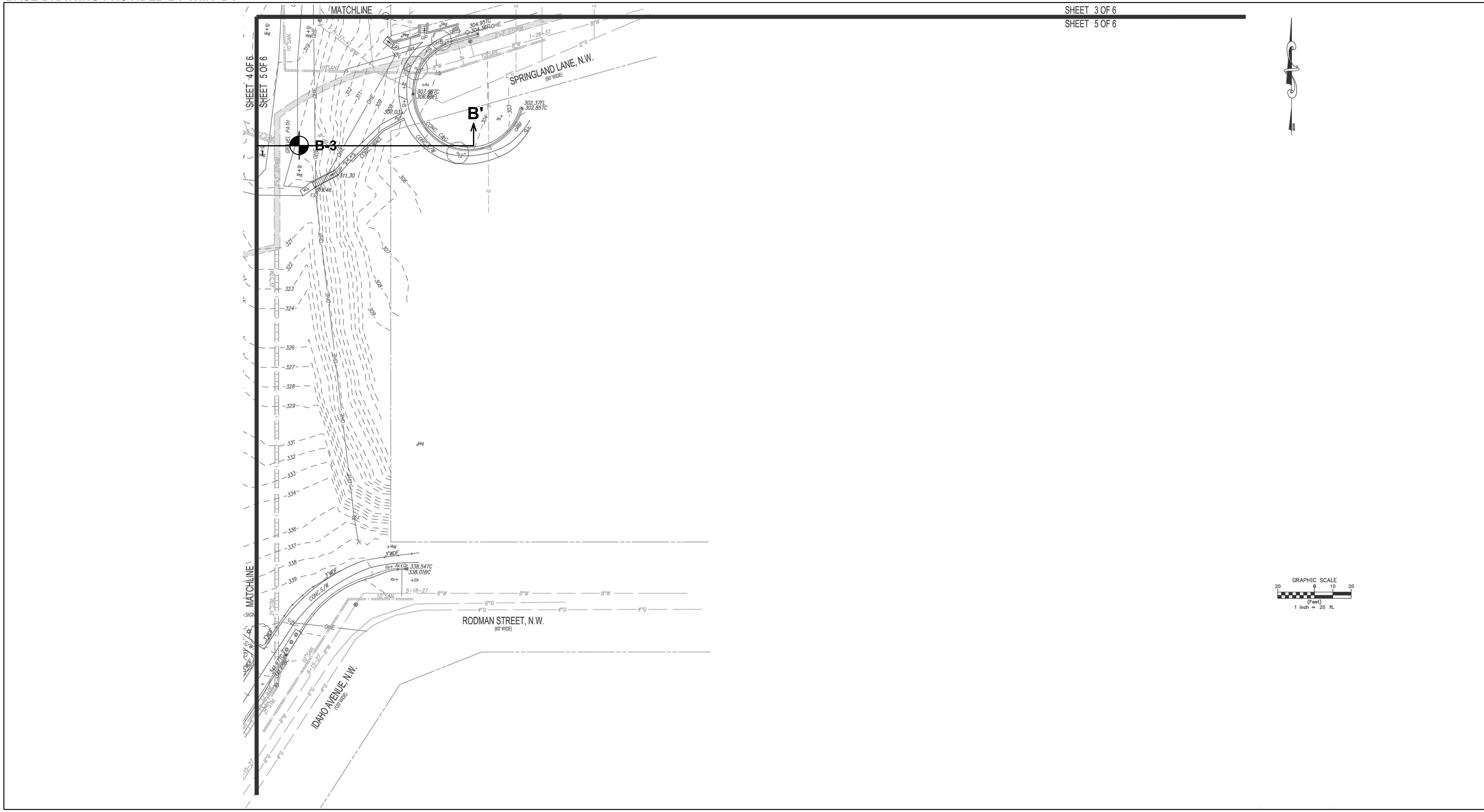
DMY CAPITOL

DMY CAPITOL, LLC
4400 MACARTHUR BLVD NW, SUITE 304
WASHINGTON, DC 20007
PHONE: (202) 741-9159
FAX: (301) 768-4169

HEARST PARK AND POOL
3950 37TH STREET NW
WASHINGTON, DC

DATE: 11/27/17	DRAFTED BY: SG	PROJECT NO.: 02.02340.02
SCALE: AS SHOWN	CHECKED BY: MC	FIGURE NO.: 3A

BASE DRAWING PROVIDED BY WMC-DC



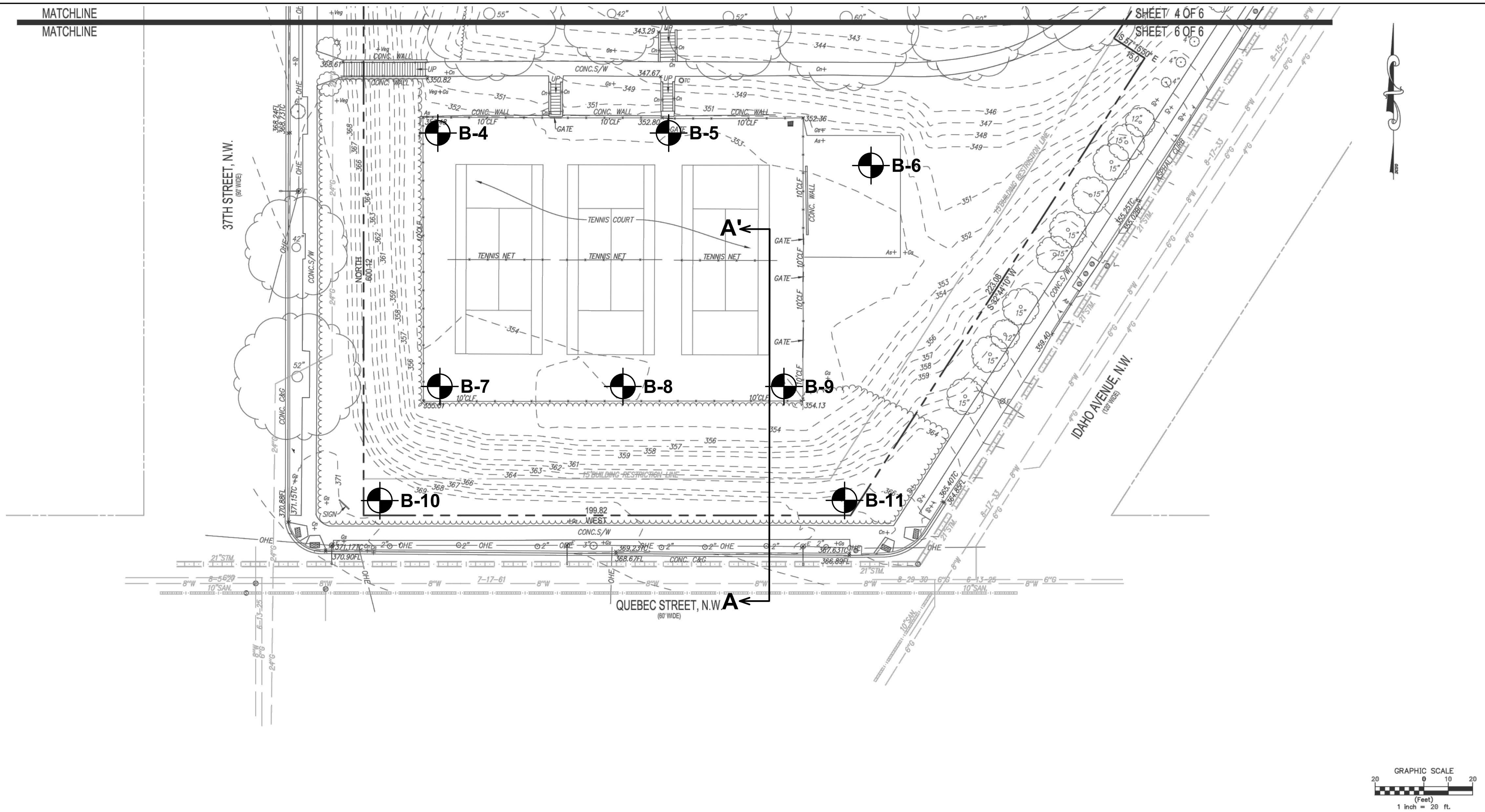
APPROXIMATE BORING LOCATION

APPROXIMATE INFILTRATION LOCATION

BORING/INFILTRATION AND CROSS SECTION LOCATION PLAN	DMY CAPITOL	HEARST PARK AND POOL 3950 37TH STREET NW WASHINGTON, DC		
		DATE: 11/27/17	DRAFTED BY: SG	PROJECT NO.: 02.02340.02
		SCALE: AS SHOWN	CHECKED BY: MC	FIGURE NO.: 3B

DMY CAPITOL, LLC
4400 MACARTHUR BLVD NW, SUITE 304
WASHINGTON, DC 20007
PHONE: (202) 741-9159
FAX: (301) 768-4169

BASE DRAWING PROVIDED BY WMC-DC



APPROXIMATE BORING LOCATION

APPROXIMATE INFILTRATION LOCATION

BORING/INFILTRATION AND CROSS SECTION LOCATION PLAN

DMY CAPITOL

DMY CAPITOL, LLC
4400 MACARTHUR BLVD NW, SUITE 304
WASHINGTON, DC 20007
PHONE: (202) 741-9159
FAX: (301) 768-4169

HEARST PARK AND POOL
3950 37TH STREET NW
WASHINGTON, DC

DATE: 11/27/17	DRAFTED BY: SG	PROJECT NO.: 02.02340.02
SCALE: AS SHOWN	CHECKED BY: MC	FIGURE NO.: 3C

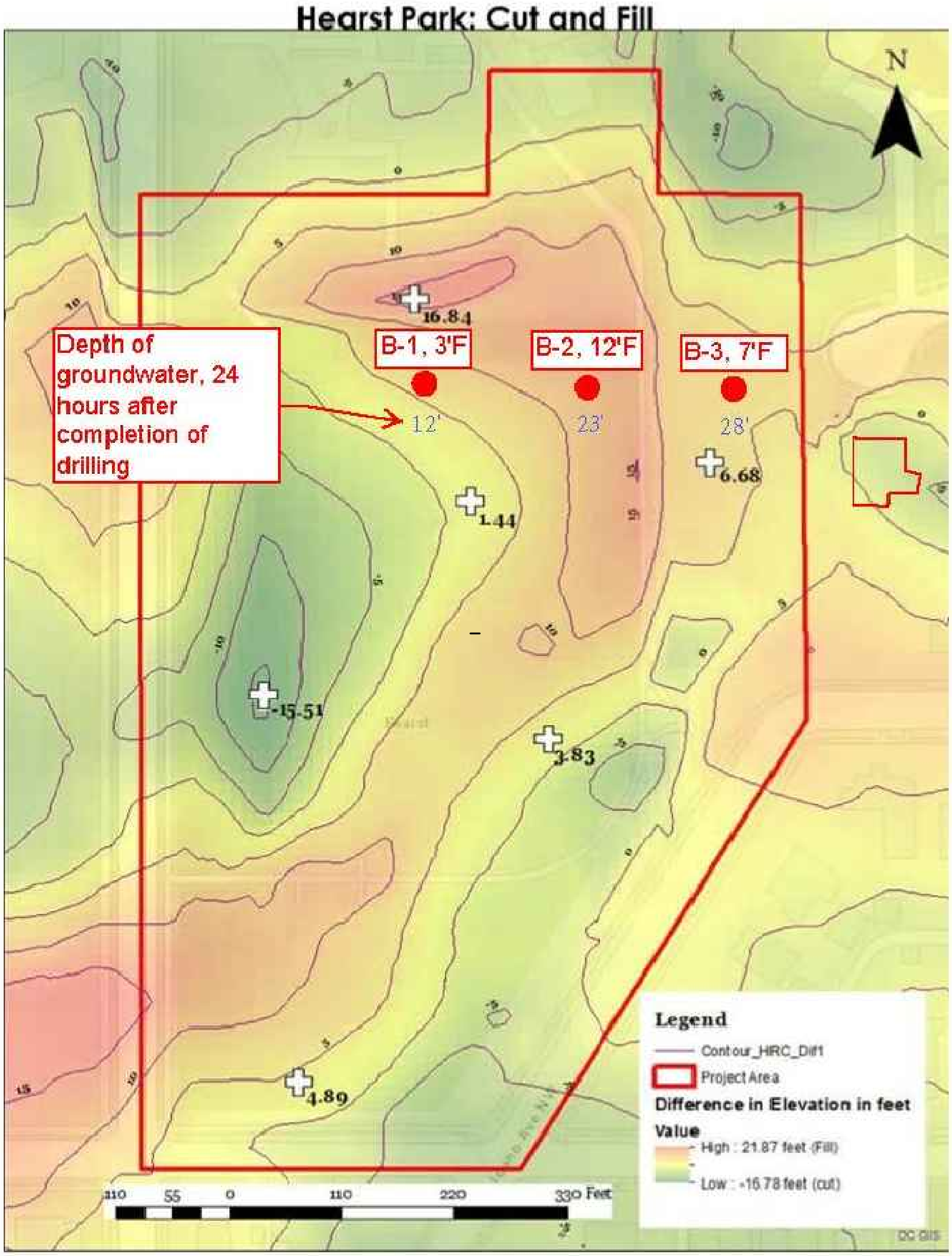


Figure 3. Results of Hearst Park and Pool elevation change analysis comparing the 1888 and modern topographic maps. Note: Red shades indicate fill, green shades indicate cut, and yellow shades indicate minor change.

DMY BORINGS ON HISTORICAL CUT AND FILL PLAN	DMY CAPITOL, LLC 4400 MACARTHUR BLVD NW, SUITE 304 WASHINGTON, DC 20007 PHONE: (202) 741-9159 FAX: (301) 768-4169	HEARST PARK AND POOL 3950 37TH STREET NW WASHINGTON, DC		
		DATE: 12/04/17	DRAFTED BY: SG	PROJECT NO.: 02.02340.02
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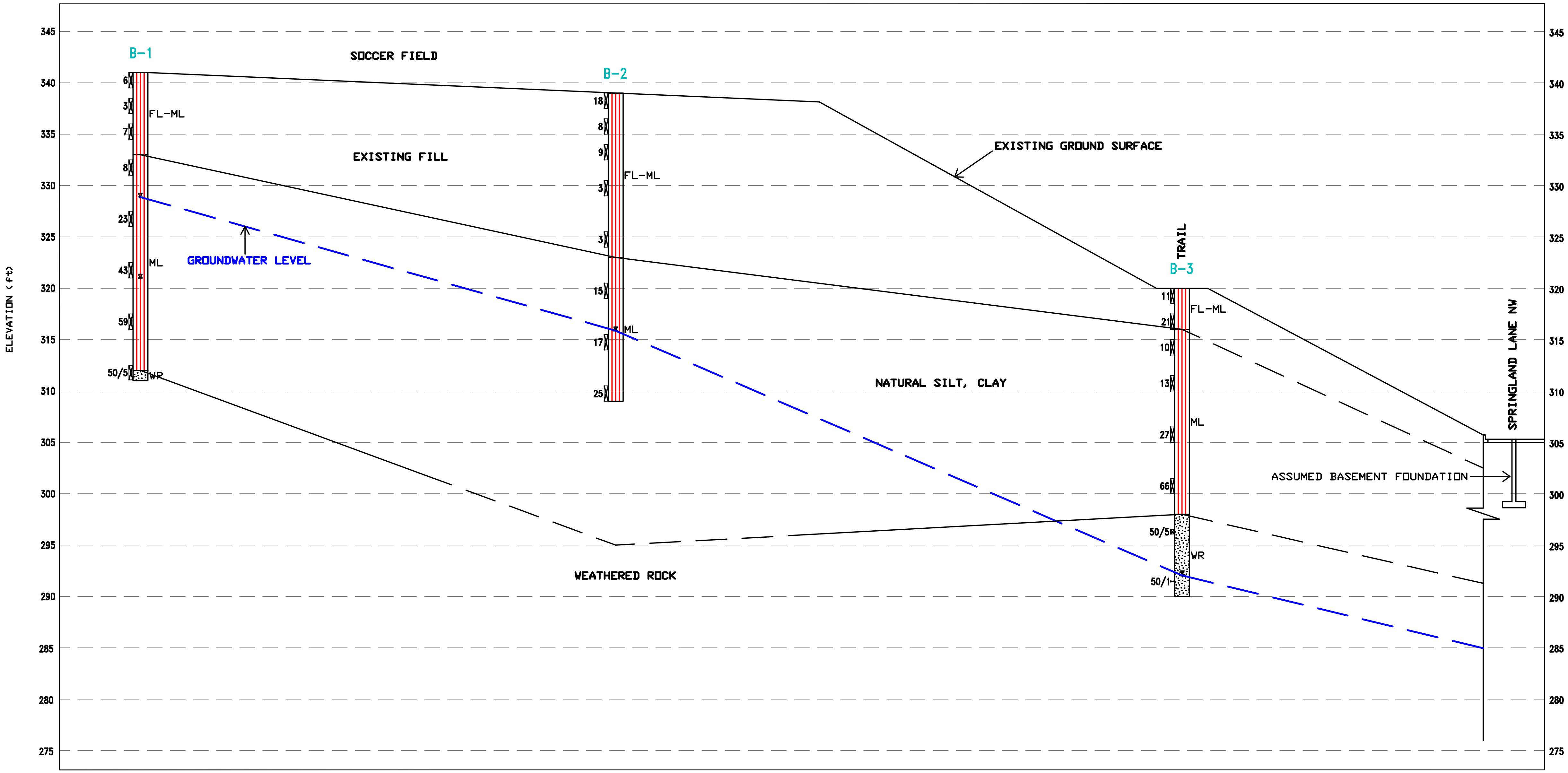


Notes: See borehole logs for complete data

Horizontal Scale: 1"=30'
Vertical Scale: 1"=12'

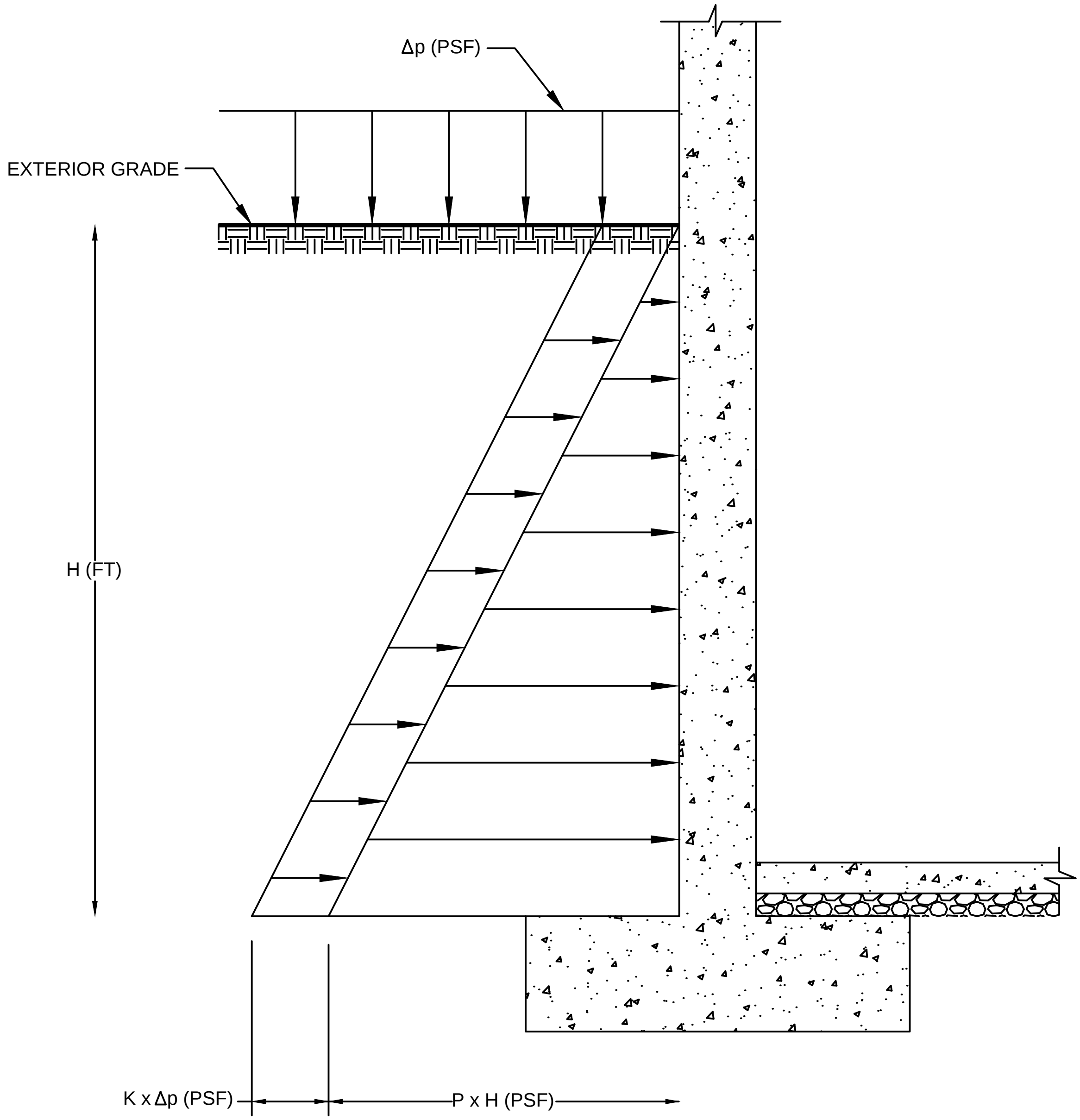
SUBSURFACE PROFILE C-C'

DMY CAPITOL

DMY CAPITOL, LLC
4400 MACARTHUR BLVD NW, SUITE 304
WASHINGTON, DC 20007
PHONE: (202) 741-9159
FAX: (301) 768-4169

HEARST PARK AND POOL
3950 37TH STREET NW
WASHINGTON, DC

DATE: 11/29/17	DRAFTED BY: SG	PROJECT NO.: 02.02340.02
SCALE: AS SHOWN	CHECKED BY: MC	FIGURE NO.: 5



LEGEND:

P	LATERAL EARTH PRESSURE, = 60 PCF
H	BACKFILL HEIGHT
Δp	APPLICABLE SURCHARGE
K	AT REST HORIZONTAL EARTH PRESSURE COEFFICIENT, =0.5

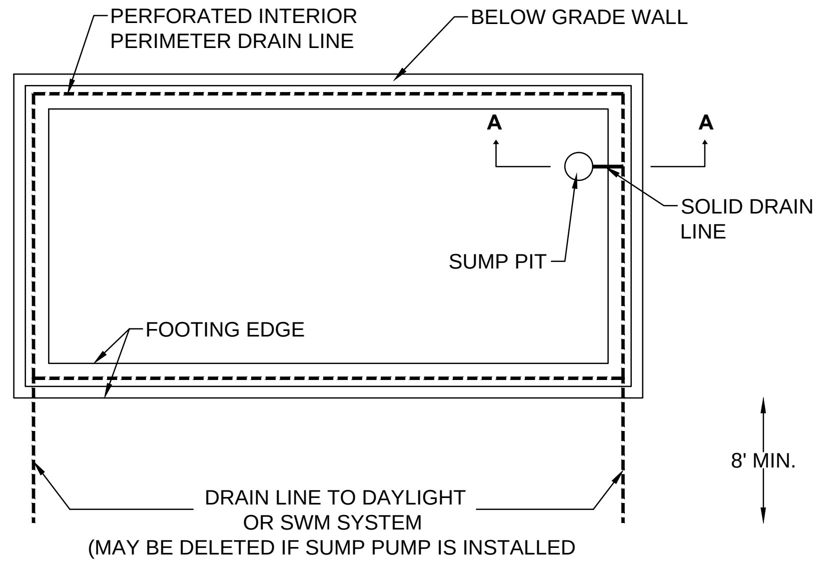
LATERAL EARTH PRESSURE DIAGRAM

DMY CAPITOL

DMY CAPITOL, LLC
4400 MACARTHUR BLVD NW, SUITE 304
WASHINGTON, DC 20007
PHONE: (202) 741-9159
FAX: (301) 768-4169

HEARST PARK AND POOL
3950 37TH STREET NW
WASHINGTON D.C.

DATE: 11/17/17	DRAFTED BY: SG	PROJECT NO.: 02.02340.02
SCALE: N.T.S.	CHECKED BY: MC	FIGURE NO.: 6



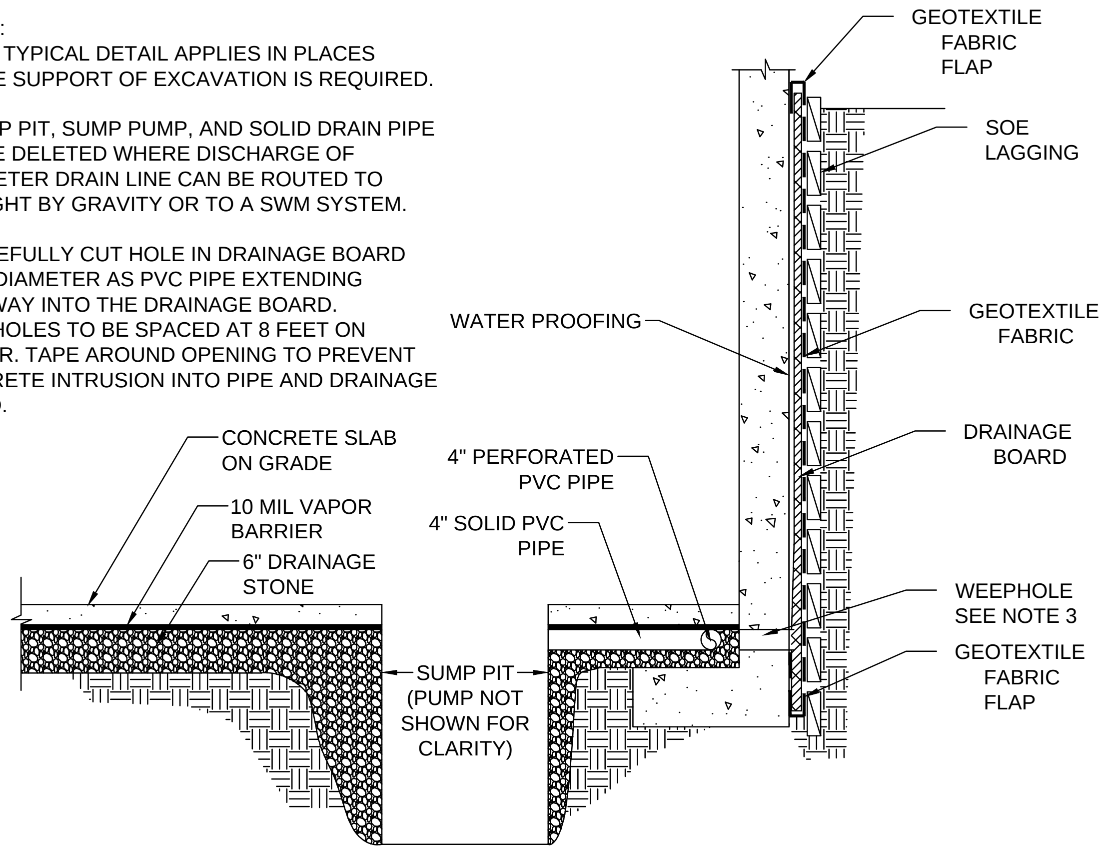
FOUNDATION PLAN SCHEMATIC

NOTES:

1. THIS TYPICAL DETAIL APPLIES IN PLACES WHERE SUPPORT OF EXCAVATION IS REQUIRED.

2. SUMP PIT, SUMP PUMP, AND SOLID DRAIN PIPE CAN BE DELETED WHERE DISCHARGE OF PERIMETER DRAIN LINE CAN BE ROUTED TO DAYLIGHT BY GRAVITY OR TO A SWM SYSTEM.

3. CAREFULLY CUT HOLE IN DRAINAGE BOARD SAME DIAMETER AS PVC PIPE EXTENDING HALF-WAY INTO THE DRAINAGE BOARD. WEEPHOLES TO BE SPACED AT 8 FEET ON CENTER. TAPE AROUND OPENING TO PREVENT CONCRETE INTRUSION INTO PIPE AND DRAINAGE BOARD.



SECTION A-A

BELOW GRADE DRAINAGE DETAIL SUPPORT OF EXCAVATIONS	DMYCAPITOL DMY CAPITOL, LLC 4400 MACARTHUR BLVD NW, SUITE 304 WASHINGTON, DC 20007 PHONE: (202) 741-9159 FAX: (301) 768-4169	**HEARST PARK AND POOL** **3950 37TH STREET NW** **WASHINGTON, D.C.**	
		DATE: 11/17/17	DRAFTED BY: SG
		PROJECT NO.: 02.02340.02	SCALE: N.T.S.
		CHECKED BY: MC	FIGURE NO.: 7

APPENDIX B FIELD OPERATIONS

SUBSURFACE EXPLORATION PROCEDURES

Soil Borings – Hollow Stem Auger

In hollow stem auger drilling, the drill rig utilizes continuous flight, hollow stem (center opening ranges from 2-1/4 to 4-1/4 inches in size) augers to advance the boreholes. During drilling or formation cutting, the center of the hollow augers is filled with rods connected to a plug at the bottom bit. Once the desired drilling depth is reached, the center plug and rods can be pulled out, leaving the hollow augers in place to hold the borehole open for sampling and well installation. Sampling is performed through the center opening in the hollow stem augers by means of the split-barrel sampling procedure in accordance with ASTM D1586. Usually, drilling fluid is not used during the soil drilling using this procedure.

Soil Borings – Mud Rotary

In mud rotary drilling, a drill bit is attached to a string of drilling rods, which are drilled into ground. "Mud" (a mixture of water and drilling additives) is pumped through the rotating rod and out the bit. As the bit cuts hole, the mud circulating out the bit carries the cut materials (known as cuttings) up the hole, out into the settling tank where the cuttings will settle. Once the desired drilling depth is reached, the rods can be pulled out, leaving the mud-supported borehole for sampling and well installation. Sampling is performed by means of the split-barrel sampling procedure in accordance with ASTM D1586.

Standard Penetration Tests

In this process, a 2 foot long, 2 inch outside-diameter split-barrel sampler attached to the end of a string of drilling rods is driven 18 inches into the ground by successive blows of a 140 pound hammer freely dropping 30 inches. The number of blows needed for each 6 inches of penetration is recorded. The blows required for the first 6 inches of penetration are allowed for seating the sampler into any loose cuttings, and the sum of the blows required for penetration of the second and third 6 inch increments constitutes the standard penetration resistance or N-value. After the test, the sampler is extracted from the ground and opened to allow visual examination and classification of the retained soil sample. The N-value can be used as a qualitative indication of the in-place relative density of cohesionless soils (sands). In a less reliable way, it also indicates the consistency of cohesive soils (clays/silts). This indication is qualitative, since many factors can significantly affect the N-value and prevent a direct correlation among drilling crews, drill rigs, drilling procedures, and hammer-rod-sampler assemblies. The N-value also has been empirically correlated with various soil properties including strength, compressibility and potential for difficult excavation.

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
>50	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
>50	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:

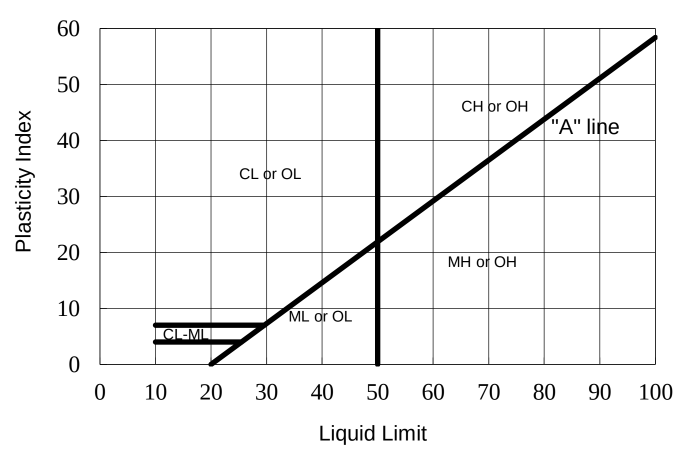
LL	- LIQUID LIMIT (%)
PI	- PLASTIC INDEX (%)
W	- MOISTURE CONTENT (%)
DD	- DRY DENSITY (PCF)
NP	- NON PLASTIC
-200	- PERCENT PASSING NO. 200 SIEVE
PP	- POCKET PENETROMETER (TSF)

▽ 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

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

MAJOR DIVISIONS			GROUP SYMBOL	TYPICAL NAMES	LABORATORY CLASSIFICATION CRITERIA	
COARSE-GRAINED SOILS (Less than 50% passes No. 200 Sieve size)	**GRAVELS** (50% or less of coarse fraction ispasses No. 4 sieve)	**CLEAN GRAVELS** (Less than 5% passes No. 200 sieve)	**GW**	Well-graded gravels, gravel-sand mixtures, little or no fines	$C_u = D_{60}/D_{10}$ greater than 4 $C_c = (D_{30})^2/(D_{10}xD_{60})$ between 1 and 3	
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines	Not meeting all gradation requirements for GW	
		GRAVELS WITH FINES (More than 12% passes No. 200 sieve)	**GM**	Silty gravels, gravel-sand mixtures	Atterberg limits below “A” line or P.I. less than 4	Above “A” line with P.I. between 4 and 7 are borderline cases requiring use of dual symbols
			GC	Clayey gravels, gravel-sand-clay mixtures	Atterberg limits below “A” line or P.I. less than 7	
	SANDS (More than 50% of coarse fraction passes No. 4 sieve)	**CLEAN SANDS** (Less than 5% passes No. 200 sieve)	**SW**	Well-graded sands, gravelly sands, little or no fines	$C_u = D_{60}/D_{10}$ greater than 6 $C_c = (D_{30})^2/(D_{10}xD_{60})$ between 1 and 3	
			SP	Poorly graded sands, gravelly sands, little or no fines	Not meeting all gradation requirements for SW	
		SANDS WITH FINES (More than 12% passes No. 200 sieve)	**SM**	Silty sands, sand-silt mixtures	Atterberg limits above “A” line or P.I. less than 4	Limits plotting in CL-ML zone with P.I. between 4 and 7 are borderline cases requiring use of dual symbols
			SC	Clayey sands, sand-clay mixtures	Atterberg limits above “A” line with P.I. greater than 7	
FINE-GRAINED SOILS (50% or more passes No. 200 Sieve)	**SILTS AND CLAYS** (Liquid limit less than 50)		**ML**	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity		
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays		
			OL	Organic silts and organic silty clays of low plasticity		
	SILTS AND CLAYS (Liquid limit greater than 50)		**MH**	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts		
			CH	Inorganic clays of high plasticity, fat clays		
			OH	Organic clays of medium to high plasticity, organic silts		
	HIGHLY ORGANIC SOILS		**Pt**	Peat and other highly organic soils		

PLASTICITY CHART



DEGREE OF PLASTICITY OF COHESIVE SOILS

Degree of Plasticity	Plasticity Index
None to Slight	0-4
Slight	5-7
Medium	8-22
High to Very High	Over 22

PROJECT NAME: Hearst Park and Pool
PROJECT NO.: 02.02340.02
LOCATION: Washington, DC
CLIENT: Cox, Graae and Spack Architects

B-1

DATE(S) DRILLED:10-11-2017
DRILLING METHOD(S): 3.25 in HSA
DRILLING EQUIPMENT: CME 55C
DRILLER: M. Santos LOGGER: B. Colunga
SURFACE ELEVATION: 341.0 ft
NORTH: 478580.7203
EAST: 1313514.6382

GROUND WATER FIRST ENCOUNTERED AT: 20.0 ft

12.0 FT AFTER 24 HOURS

FIELD DATA / MATERIAL DESCRIPTION OF STRATA / LAB DATA

DEPTH (FT)	ELEVATION (FT)	SPT BLOW COUNTS	SAMPLE INTERVAL	% RECOVERY	ROCK QUALITY DESIGNATION %	RMR	GEOLOGIC STRATA	MATERIAL DESCRIPTION OF STRATA	LIQUID LIMIT (LL)	PLASTICITY INDEX (PI)	MOISTURE CONTENT (%)
0.0	341.0	3 3 3	0.0	100				0.0 / 341.0 TOPSOIL **Tops** -2 in			
0.2	340.8						F1	0.2 / 340.8 Orange and gray, sandy lean clay FILL, soft to firm, moist to very moist **FL-CL**			
2.5		2 1 2	2.5	89							
5.0		2 3 4	5.0	67							
8.0	333.0						R1	8.0 / 333.0 Brown and gray, sandy SILT, soft to very hard, moist to wet **ML**			
8.5		3 4 4	8.5	100							20.6
13.5		8 8 15	13.5	100				SAME, black lenses			
18.5		6 20 23	18.5	100				SAME, orange			
23.5		30 28 31	23.5	100				SAME, blue and gray			
28.5		29 45 50/5	28.5	100							
29.0	312.0						WR	29.0 / 312.0 Gray, weathered ROCK			
30.0	311.0							30.0 / 311.0 Boring Terminated			

REMARKS: Surface elevations were provided by Stantec and are approximate.

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Dulles, Virginia 20166
tel: (703) 665-0586 fax: (301) 768-4169

PROJECT NAME: Hearst Park and Pool
PROJECT NO.: 02.02340.02
LOCATION: Washington, DC
CLIENT: Cox, Graae and Spack Architects

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PAGE 1 OF 1

DATE(S) DRILLED:10-11-2017
DRILLING METHOD(S): 3.25 in HSA
DRILLING EQUIPMENT: CME 55C
DRILLER: M. Santos LOGGER: B. Colunga
SURFACE ELEVATION: 339.0 ft
NORTH: 478579.7892
EAST: 1313633.2763

GROUND WATER WAS NOT ENCOUNTERED DURING DRILLING

23.0 FT AFTER 24 HOURS

FIELD DATA / LAB DATA

DEPTH (FT)	ELEVATION (FT)	SPT BLOW COUNTS	SAMPLE INTERVAL	% RECOVERY	ROCK QUALITY DESIGNATION %	RMR	GEOLOGIC STRATA	MATERIAL DESCRIPTION OF STRATA	LIQUID LIMIT (LL)	PLASTICITY INDEX (PI)	MOISTURE CONTENT (%)
0.0		6 8 10	0.0	28			F1	0.0 / 339.0 TOPSOIL **Tops** -3 in			
0.3								0.3 / 338.8 Brown, sandy silt FILL, soft to very stiff, moist **FL-ML**			
	335	8 4 4	2.5	28							
5		4 4 5	5.0	28							13.3
	330	3 2 1	8.5	78				SAME, mottled black and white			
10											
	325	1 1 2	13.5	100							
15											
16.0							R1	16.0 / 323.0 Orange and brown, sandy SILT, stiff to very stiff, moist **ML**			
	320	4 6 9	18.5	100							
20											
	315	9 7 10	23.5	100							
25											
	310	13 13 12	28.5	100							
30								30.0 / 309.0 Boring Terminated			

REMARKS: Surface elevations were provided by Stantec and are approximate.

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SPT_LOG:C:\USERS\SGHANY\DESKTOP\HEARST PARK & POOL\0 BORING LOGS\02.02340.02 BORING LOGS - REVISED.GPJ:02.02340.02 BORING LOGS - REVISED.GPJ:11/27/17

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PROJECT NAME: Hearst Park and Pool
PROJECT NO.: 02.02340.02
LOCATION: Washington, DC
CLIENT: Cox, Graae and Spack Architects

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DATE(S) DRILLED:11-7-2017
DRILLING METHOD(S): 3.25 in HSA
DRILLING EQUIPMENT: CME 55C
DRILLER: M. Santos LOGGER: J. Holmes
SURFACE ELEVATION: 320.0 ft
NORTH: 478569.0817
EAST: 1313774.2464

GROUND WATER WAS NOT ENCOUNTERED DURING DRILLING

28.0 FT AFTER 24 HOURS

FIELD DATA / MATERIAL DESCRIPTION OF STRATA / LAB DATA

DEPTH (FT)	ELEVATION (FT)	SPT BLOW COUNTS	SAMPLE INTERVAL	% RECOVERY	ROCK QUALITY DESIGNATION %	RMR	GEOLOGIC STRATA	MATERIAL DESCRIPTION OF STRATA	LIQUID LIMIT LL	PLASTICITY INDEX PI	MOISTURE CONTENT (%)
		6 3 8	0.0	44			F1	0.0 / 320.0 Orange and brown, sandy silt FILL, trace gravel, stiff to very stiff, moist **FL-ML**			
		3 3 18	2.5	56							
								4.0 / 316.0 Orange and brown, sandy SILT, stiff to very hard, moist **ML**			
5	315	4 4 6	5.0	100							
		2 4 9	8.5	89							
10	310										
		8 12 15	13.5	100			R1				
15	305										
		12 28 38	18.5	100							
20	300										
								22.0 / 298.0 Gray, very weathered to weathered ROCK			
		50/5	23.5	100							
25	295						WR				
		50/1	28.5	100				SAME, wet			
30	290							30.0 / 290.0 Boring Terminated			

REMARKS: Surface elevations were provided by Stantec and are approximate.

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PROJECT NAME: Hearst Park and Pool
PROJECT NO.: 02.02340.02
LOCATION: Washington, DC
CLIENT: Cox, Graae and Spack Architects

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DATE(S) DRILLED:10-6-2017
DRILLING METHOD(S): 4 inch Mud Rotary
DRILLING EQUIPMENT: CME 55C
DRILLER: M. Santos LOGGER: B. Buckman
SURFACE ELEVATION: 353.0 ft

GROUND WATER FIRST ENCOUNTERED AT: 14.0 ft

NO LONG TERM MEASUREMENTS TAKEN

FIELD DATA / LAB DATA

DEPTH (FT)	ELEVATION (FT)	SPT BLOW COUNTS	SAMPLE INTERVAL	% RECOVERY	ROCK QUALITY DESIGNATION %	RMR	GEOLOGIC STRATA	MATERIAL DESCRIPTION OF STRATA	LIQUID LIMIT LL	PLASTICITY INDEX PI	MOISTURE CONTENT (%)
0.0	353.0							0.0 / 353.0 ASPHALT **Asph** -10 in			
0.8	352.2							0.8 / 352.2 AGGREGATE BASE **GB** -3 in			
1.0	352.0							1.0 / 352.0 CONCRETE **Conc** -3 in			
1.3	351.7						F1	1.3 / 351.7 Brown, sandy silt FILL, soft to very stiff, moist **FL-ML**			
1.5		8 5 7	1.5	67							
3.0		4 3 3	3.0	67							
5.0		2 3 4	5.0	89							
								SAME, coal fragments			
8.5		WH 1 1	8.5								
13.5		8 11 16	13.5	100							
18.5		7 12 12	18.5	100							
20.0	333.0							20.0 / 333.0 Boring Terminated			

REMARKS: Surface elevations were provided by Stantec and are approximate.

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PROJECT NAME: Hearst Park and Pool	**B-5**
PROJECT NO.: 02.02340.02	**PAGE 1 OF 1**
LOCATION: Washington, DC	
CLIENT: Cox, Graae and Spack Architects	

DATE(S) DRILLED:10-6-2017
DRILLING METHOD(S): 4 inch Mud Rotary
DRILLING EQUIPMENT: CME 55C
DRILLER: M. Santos   LOGGER: B. Colunga
SURFACE ELEVATION: 353.0 ft

▽ GROUND WATER FIRST ENCOUNTERED AT: 15.0 ft

NO LONG TERM MEASUREMENTS TAKEN

FIELD DATA / LAB DATA

SAMPLE INTERVAL	SPT BLOW COUNTS	% RECOVERY	ROCK QUALITY DESIGNATION %	RMR	GEOLOGIC STRATA	LIQUID LIMIT (LL)	PLASTICITY INDEX (PI)	MOISTURE CONTENT (%)	% Finer than #200
1.0	3 2 3	89			F1				
2.5	3 2 3	11			F1				
5.0	1 2 2	100			F1				
8.5	2 1 1	100			F1				
13.5	3 1 1	67			F1	32	13	19.3	50.3
18.5	13 10 16	67			F2				

MATERIAL DESCRIPTION OF STRATA

DEPTH (FT) / ELEVATION (FT)	DESCRIPTION
0.0 / 353.0	ASPHALT **Asph** -3 in
0.3 / 352.8	AGGREGATE BASE **GB** -6 in
0.8 / 352.3	CONCRETE **Conc** -3 in
1.0 / 352.0	Orange and brown, sandy lean clay with gravel FILL, soft to very stiff, moist to very moist **FL-CL** SAME, contains asphalt fragments at 4 feet
17.0 / 336.0	Light brown, fine to medium silty sand with gravel FILL, medium dense, wet **FL-SM**
20.0 / 333.0	Boring Terminated

REMARKS: Surface elevations were provided by Stantec and are approximate.

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DMY	PROJECT NAME: Hearst Park and Pool PROJECT NO.: 02.02340.02 LOCATION: Washington, DC CLIENT: Cox, Graae and Spack Architects

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DATE(S) DRILLED:10-5-2017
DRILLING METHOD(S): 4 inch Mud Rotary
DRILLING EQUIPMENT: CME 55C
DRILLER: M. Santos   LOGGER: C. Colunga
SURFACE ELEVATION: 352.5 ft

GROUND WATER FIRST ENCOUNTERED AT: 13.0 ft

NO LONG TERM MEASUREMENTS TAKEN

FIELD DATA / **LAB DATA**

DEPTH (FT)	ELEVATION (FT)	SPT BLOW COUNTS	SAMPLE INTERVAL	% RECOVERY	ROCK QUALITY DESIGNATION %	RMR	GEOLOGIC STRATA	MATERIAL DESCRIPTION OF STRATA	LIQUID LIMIT (LL)	PLASTICITY INDEX (PI)	MOISTURE CONTENT (%)	% Finer than #200
								0.0 / 352.5 ASPHALT **Asph** -5 in				
	350	1 1 1	0.5	83			F1	0.4 / 352.1 Gray and brown, sandy silt FILL, soft to firm, moist to very moist **FL-ML**				
		2 3 3	2.5	89								
5	345	6 4 4	5.0	83								
10		4 4 5	8.5	100			R1	8.0 / 344.5 Olive Brown To Olive Gray, sandy ELASTIC SILT, firm, moist **MH**	65	21	43.3	69.6
15	340	4 3 4	13.5	100			R1	12.0 / 340.5 Light gray, sandy SILT, firm, moist **ML** Uc = 3,000 psf using pocket penetrometer				
20	335	4 4 7	18.5	100			R2	17.0 / 335.5 Orange Brown, fine to medium SILTY SAND, medium dense, wet **SM**				
								20.0 / 332.5 Boring Terminated				

REMARKS: Surface elevations were provided by Stantec and are approximate.

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PROJECT NAME: Hearst Park and Pool
PROJECT NO.: 02.02340.02
LOCATION: Washington, DC
CLIENT: Cox, Graae and Spack Architects

DATE(S) DRILLED:10-10-2017
DRILLING METHOD(S): 3.25 in HSA
DRILLING EQUIPMENT: CME 55C
DRILLER: M. Santos LOGGER: B. Colunga
SURFACE ELEVATION: 354.5 ft

▽ GROUND WATER FIRST ENCOUNTERED AT: 22.0 ft

11.0 FT AFTER 24 HOURS

FIELD DATA / LAB DATA

DEPTH (FT)	ELEVATION (FT)	SPT BLOW COUNTS	SAMPLE INTERVAL	% RECOVERY	ROCK QUALITY DESIGNATION %	RMR	GEOLOGIC STRATA	LIQUID LIMIT (LL)	PLASTICITY INDEX (PI)	MOISTURE CONTENT (%)	% Finer than #200
		2 1 1	1.7	44							
5	350	WH WH 1	3.5				F1				
		3 2 4	5.0	89							
10	345	2 2 2	8.5	67			R1	32	10	28.2	86.4
15	340	2 3 4	13.5	100							
			15.0	100							
20	335	4 5 6	18.5	100							
25	330	4 7 8	23.5	100			R1				
30	325	8 9 11	28.5	100							

MATERIAL DESCRIPTION OF STRATA

0.0 / 354.5
ASPHALT **Asph** -18 in

1.5 / 353.0
CONCRETE **Conc** -2 in

1.7 / 352.8
Orange and brown, sandy silt FILL, soft, moist **FL-ML**

5.0 / 349.5
Orange and gray, LEAN CLAY WITH SAND, soft to firm, moist to very moist **CL**

Uc = 2,500 psf using pocket penetrometer

16.0 / 338.5
Gray and blue, sandy SILT, stiff to very stiff, wet **ML**

30.0 / 324.5 Boring Terminated

REMARKS: Surface elevations were provided by Stantec and are approximate.

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PROJECT NAME: Hearst Park and Pool
PROJECT NO.: 02.02340.02
LOCATION: Washington, DC
CLIENT: Cox, Graae and Spack Architects

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DATE(S) DRILLED:10-6-2017
DRILLING METHOD(S): 4 inch Mud Rotary
DRILLING EQUIPMENT: CME 55C
DRILLER: M. Santos LOGGER: B. Colunga
SURFACE ELEVATION: 354.0 ft

GROUND WATER FIRST ENCOUNTERED AT: 14.0 ft

NO LONG TERM MEASUREMENTS TAKEN

MATERIAL DESCRIPTION OF STRATA

GEOLOGIC STRATA	MATERIAL DESCRIPTION OF STRATA	LIQUID LIMIT (LL)	PLASTICITY INDEX (PI)	MOISTURE CONTENT (%)	% Finer than #200
	0.0 / 354.0 ASPHALT **Asph** -15 in				
	1.3 / 352.8 CONCRETE **Conc** -3 in				
F1	1.5 / 352.5 Orange and brown, sandy silt FILL, soft to stiff, slightly moist to moist **FL-ML**				
R1	8.5 / 345.5 Orange and brown to gray, sandy SILT, soft, moist **ML** Uc = 500 psf using pocket penetrometer				
R2	13.0 / 341.0 Light brown, fine to medium SILTY SAND WITH GRAVEL, loose, wet **SM**	38	13	26.6	39.2
R1	18.0 / 336.0 Gray, sandy SILT, stiff to very stiff, wet **ML**				
	35.0 / 319.0 Boring Terminated				

FIELD DATA

SAMPLE INTERVAL (top, ft)	SPT BLOW COUNTS	% RECOVERY	ROCK QUALITY DESIGNATION %	RMR
1.5	3 5 4	67		
3.0	2 2 2	67		
5.0	5 6 8	67		
8.5	WH 1 1	28		
13.5	7 4 5	56		
18.5	4 6 9	11		
23.5	5 6 9	100		
28.5	8 10 14	100		
33.5	9 11 12	100		

Depth scale (FT): 5, 10, 15, 20, 25, 30, 35; Elevation scale (FT): 350, 345, 340, 335, 330, 325, 320

REMARKS: Surface elevations were provided by Stantec and are approximate.

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PROJECT NAME: Hearst Park and Pool	**B-9**
PROJECT NO.: 02.02340.02	**PAGE 1 OF 1**
LOCATION: Washington, DC	
CLIENT: Cox, Graae and Spack Architects	

DATE(S) DRILLED:10-6-2017
DRILLING METHOD(S): 4 inch Mud Rotary
DRILLING EQUIPMENT: CME 55C
DRILLER: M. Santos LOGGER: B. Colunga
SURFACE ELEVATION: 354.0 ft

GROUND WATER FIRST ENCOUNTERED AT: 22.0 ft

NO LONG TERM MEASUREMENTS TAKEN

FIELD DATA

DEPTH (FT)	ELEVATION (FT)	SPT BLOW COUNTS	SAMPLE INTERVAL	% RECOVERY	ROCK QUALITY DESIGNATION %	RMR	GEOLOGIC STRATA
		5 9 12	1.7	56			F1
	350	12 10 8	3.5	83			F1
5		6 2 2	5.0	17			F1
10	345	1 2 1	8.5	100			R1
15	340	5 4 4	13.5	100			R1
20	335	2 3 3	18.5	100			R1
25	330	4 5 8	23.5	100			R1
30	325	6 8 11	28.5	100			R1

MATERIAL DESCRIPTION OF STRATA / LAB DATA

MATERIAL DESCRIPTION OF STRATA	LIQUID LIMIT (LL)	PLASTICITY INDEX (PI)	MOISTURE CONTENT (%)	% Finer than #200
0.0 / 354.0 ASPHALT **Asph** -18 in				
1.5 / 352.5 CONCRETE **Conc** -2 in				
1.7 / 352.3 Orange and brown, sandy silt with gravel FILL, soft to very stiff, moist **FL-ML**				
9.0 / 345.0 Orange and brown to gray, sandy SILT, soft to very stiff, very moist to wet **ML**				
13.0 / 341.0 Gray, sandy ELASTIC SILT, firm, moist **MH**	60	26	42.7	64.2
18.0 / 336.0 Gray and orange brown, sandy SILT, firm to very stiff, moist to wet **ML** Uc = 1,500 psf using pocket penetrometer				
30.0 / 324.0 Boring Terminated				

REMARKS: Surface elevations were provided by Stantec and are approximate.

PAGE 1 OF 1

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PAGE 1 OF 1

PROJECT NAME: Hearst Park and Pool
PROJECT NO.: 02.02340.02
LOCATION: Washington, DC
CLIENT: Cox, Graae and Spack Architects

DATE(S) DRILLED:10-5-2017
DRILLING METHOD(S): 4 inch Mud Rotary
DRILLING EQUIPMENT: CME 55C
DRILLER: M. Santos LOGGER: M. Clippinger/B. Colunga
SURFACE ELEVATION: 370.0 ft

GROUND WATER FIRST ENCOUNTERED AT: 22.0 ft

NO LONG TERM MEASUREMENTS TAKEN

FIELD DATA											LAB DATA			
DEPTH (FT)	ELEVATION (FT)	SPT BLOW COUNTS	SAMPLE LEGEND	SAMPLE INTERVAL	% RECOVERY	ROCK QUALITY DESIGNATION %	RMR	GEOLOGIC STRATA	GRAPHIC LOG	MATERIAL DESCRIPTION OF STRATA	LIQUID LIMIT (LL)	PLASTICITY INDEX (PI)	MOISTURE CONTENT (%)	% Finer than #200
		7 3 3	X	0.0	17			F1		0.0 / 370.0 TOPSOIL **Tops** -2 in				
		10 11 8	X	2.5	89					0.2 / 369.8 Brown, sandy lean clay FILL, firm to very stiff, moist **FL-CL**				
5	365	4 3 5	X	5.0	61						34	14	14.7	63.4
		7 10 13	X	8.5	67									
10	360													
		29 10 12	X	13.5	67									
15	355													
								R1		16.0 / 354.0 Light brown and gray, sandy SILT, soft to firm, moist to wet **ML**				
		4 3 4	X	18.5	67					Uc = 4,000 psf using pocket penetrometer				
20	350													
		WH WH 3	X	23.5	100									
25	345													
		1 1 3	X	28.5	100									
30	340									30.0 / 340.0 Boring Terminated				

REMARKS: Surface elevations were provided by Stantec and are approximate.

DMY ENGINEERING CONSULTANTS INC.
45662 Terminal Drive, Suite 110
Dulles, Virginia 20166
tel: (703) 665-0586 fax: (301) 768-4169

PROJECT NAME: Hearst Park and Pool
PROJECT NO.: 02.02340.02
LOCATION: Washington, DC
CLIENT: Cox, Graae and Spack Architects

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PAGE 1 OF 1

DATE(S) DRILLED:10-5-2017
DRILLING METHOD(S): 4 inch Mud Rotary
DRILLING EQUIPMENT: CME 55C
DRILLER: M. Santos   LOGGER: M. Clippinger/B. Colunga
SURFACE ELEVATION: 368.0 ft

GROUND WATER FIRST ENCOUNTERED AT: 13.0 ft

NO LONG TERM MEASUREMENTS TAKEN

FIELD DATA

DEPTH (FT)	ELEVATION (FT)	SPT BLOW COUNTS	SAMPLE LEGEND	SAMPLE INTERVAL	% RECOVERY	ROCK QUALITY DESIGNATION %	RMR	GEOLOGIC STRATA	GRAPHIC LOG	MATERIAL DESCRIPTION OF STRATA	LIQUID LIMIT (LL)	PLASTICITY INDEX (PI)	MOISTURE CONTENT (%)
		5 5 7		0.0	56			F1		0.0 / 368.0 TOPSOIL **Tops** -3 in			
	365	8 3 4		2.5	78					0.3 / 367.8 Light brown, sandy silt with gravel FILL, firm to very stiff, moist, contains coal fragments **FL-ML**			
5		13 5 3		5.0	78					SAME, orange and brown, plastic debris			
	360	10 14 14		8.5	78								
10													
	355	2 1 3		13.5	67			R1		12.0 / 356.0 Brown, sandy SILT, soft to very stiff, wet **ML**			
15										SAME, light gray and brown			
	350	4 3 2		18.5	100					Uc = 2,500 psf using pocket penetrometer			
20													
	345	WH 2 3		23.5	100					SAME, gray			
25													
	340	4 3 5		28.5	100					Uc = 2,000 psf using pocket penetrometer			
30										30.0 / 338.0 Boring Terminated			

LAB DATA

REMARKS: Surface elevations were provided by Stantec and are approximate.

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tel: (703) 665-0586   fax: (301) 768-4169

PROJECT NAME: Hearst Park and Pool
PROJECT NO.: 02.02340.02
LOCATION: Washington, DC
CLIENT: Cox, Graae and Spack Architects

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PAGE 1 OF 1

DATE(S) DRILLED:10-11-2017
DRILLING METHOD(S): 3.25 in HSA
DRILLING EQUIPMENT: CME 55C
DRILLER: M. Santos LOGGER: B. Colunga
SURFACE ELEVATION: 338.0 ft

GROUND WATER WAS NOT ENCOUNTERED DURING DRILLING

DRY AFTER 24 HOURS

FIELD DATA / **LAB DATA**

DEPTH (FT)	ELEVATION (FT)	SPT BLOW COUNTS	SAMPLE LEGEND	SAMPLE INTERVAL	% RECOVERY	ROCK QUALITY DESIGNATION %	RMR	GEOLOGIC STRATA	GRAPHIC LOG	MATERIAL DESCRIPTION OF STRATA	LIQUID LIMIT (LL)	PLASTICITY INDEX (PI)	MOISTURE CONTENT (%)
		2 4 6		0.0	28			F1		0.0 / 338.0 TOPSOIL **Tops** -2 in			
										0.2 / 337.8 Dark brown, sandy silt FILL, stiff, slightly moist **FL-ML**			
	335	3 3 3		2.5	28					1.5 / 336.5 Light brown, sandy SILT, firm to stiff, slightly moist, micaceous **ML**			
5		4 4 3		5.0	100			R1					
	330	4 3 4		8.5	28								
10										10.0 / 328.0 Boring Terminated			

REMARKS: Surface elevations were provided by Stantec and are approximate.

PAGE 1 OF 1

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SPT_LOG:C:\USERS\SGHANY\DESKTOP\HEARST PARK & POOL\0 BORING LOGS\02.02340.02 BORING LOGS - REVISED.GPJ:02.02340.02 BORING LOGS - REVISED.GPJ;11/27/17

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Dulles, Virginia 20166
tel: (703) 665-0586 fax: (301) 768-4169

Location: Hearst Park and Pool

Site: I-1

Time interval between readings: 0.5 minutes

Ksat Method: Earth Manual

Steady Flow Rate Condition

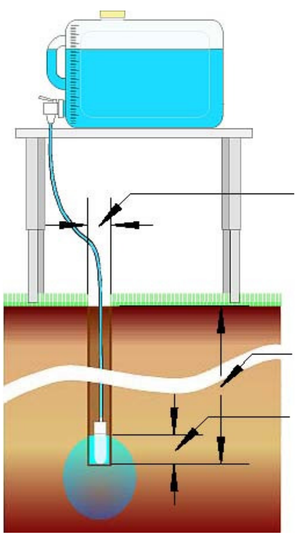
Steady Flow Rate achieved when Water Consumption Rate changes less than +/- 20 % for 3 consecutive readings

Steady Flow Rate:	33.600 ml/min
Temp. Adj. FR:	33.667 ml/min
Percolation Rate:	6.742 min/cm
Ksat:	0.75 Inches / hour

Notes:

Site GPS Position

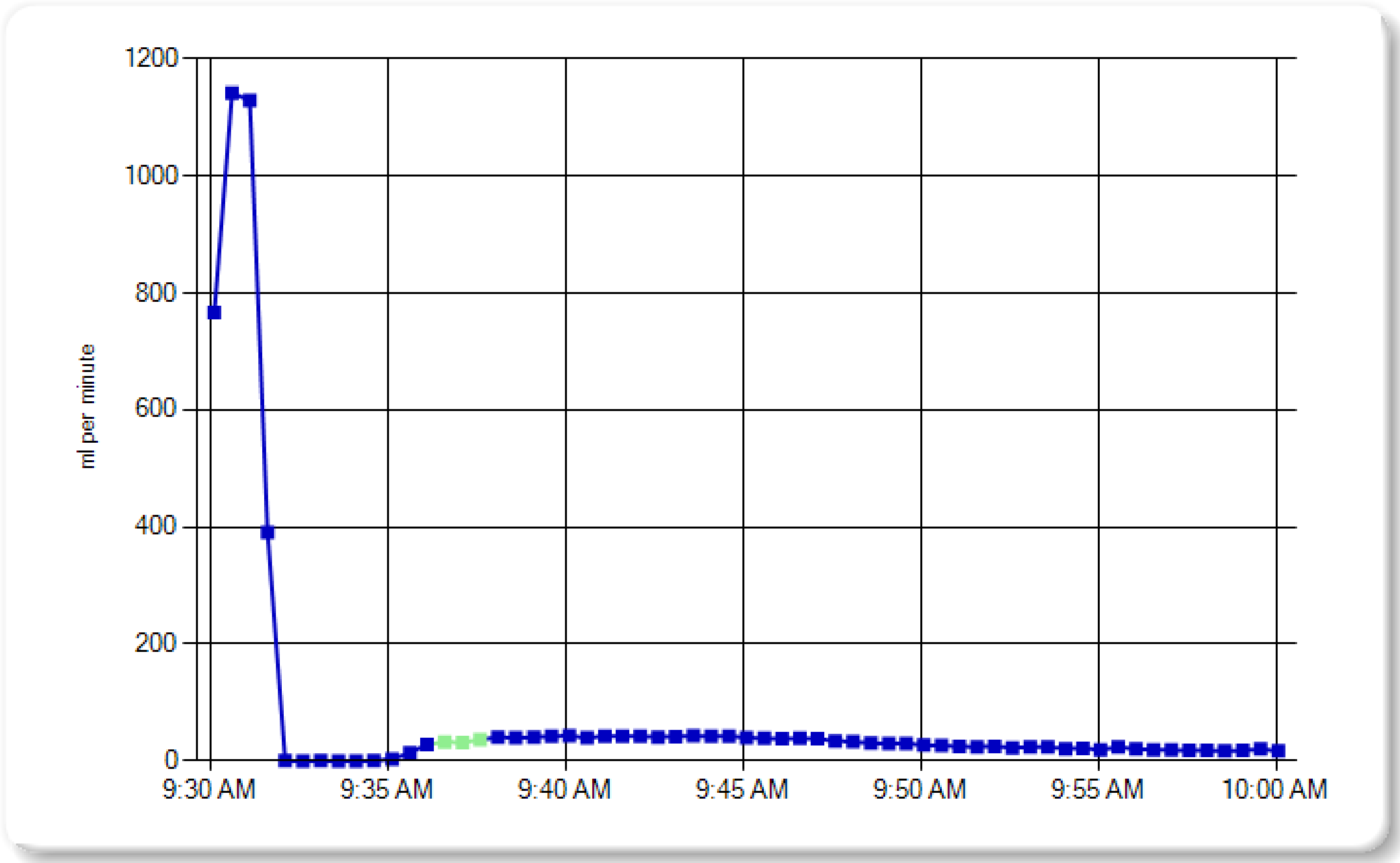
	Degrees	Minutes	Seconds	
Longitude:	0	0	0	East
Latitude:	0	0	0	North



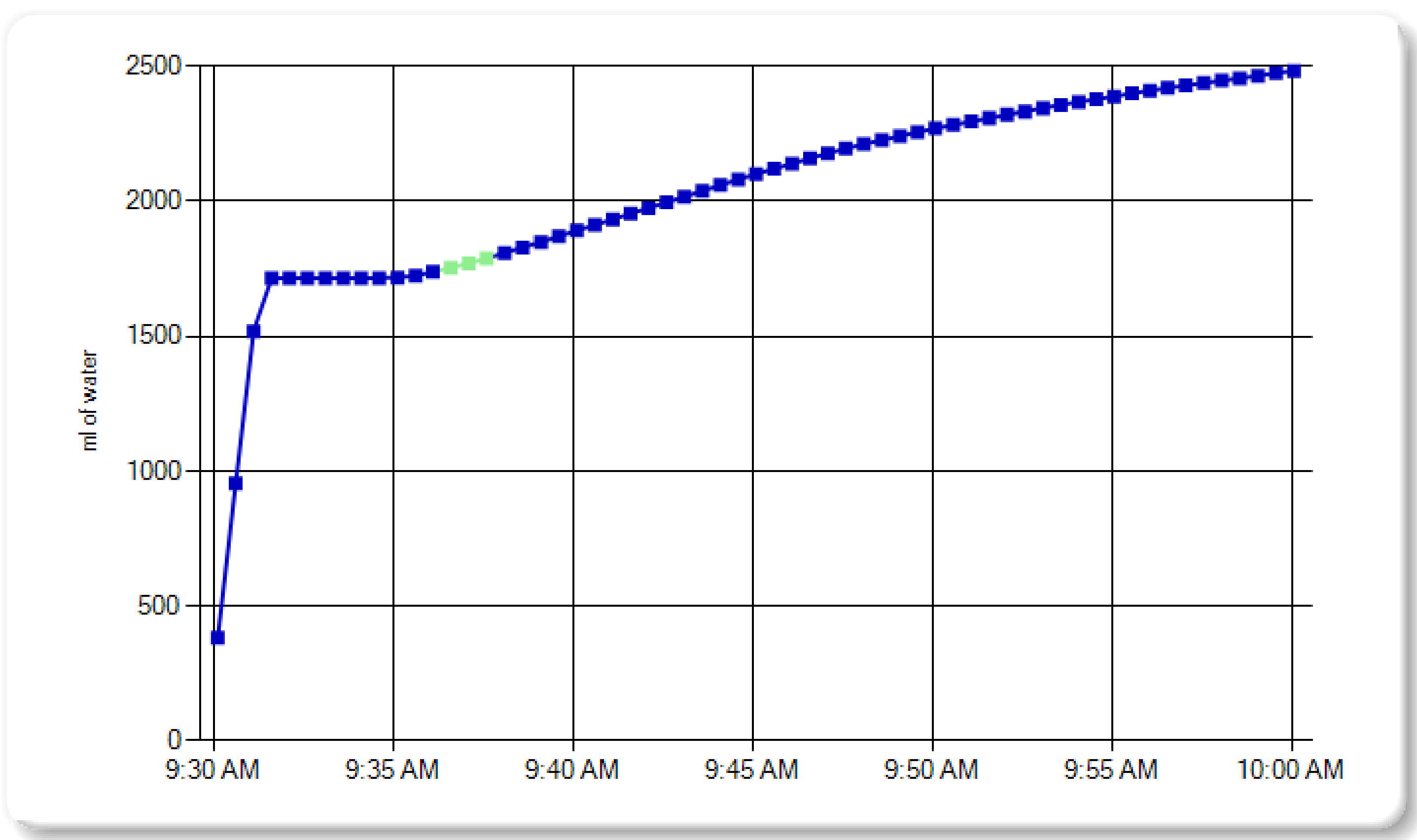
6.7 inches	Hole Diameter
70 º F	Water Temperature
70.9 inches	Hole Depth
3.5 inches	Water Height in Hole
300 inches	Water Table Depth

Soil Texture-Structure Category:

Water Consumption Rate



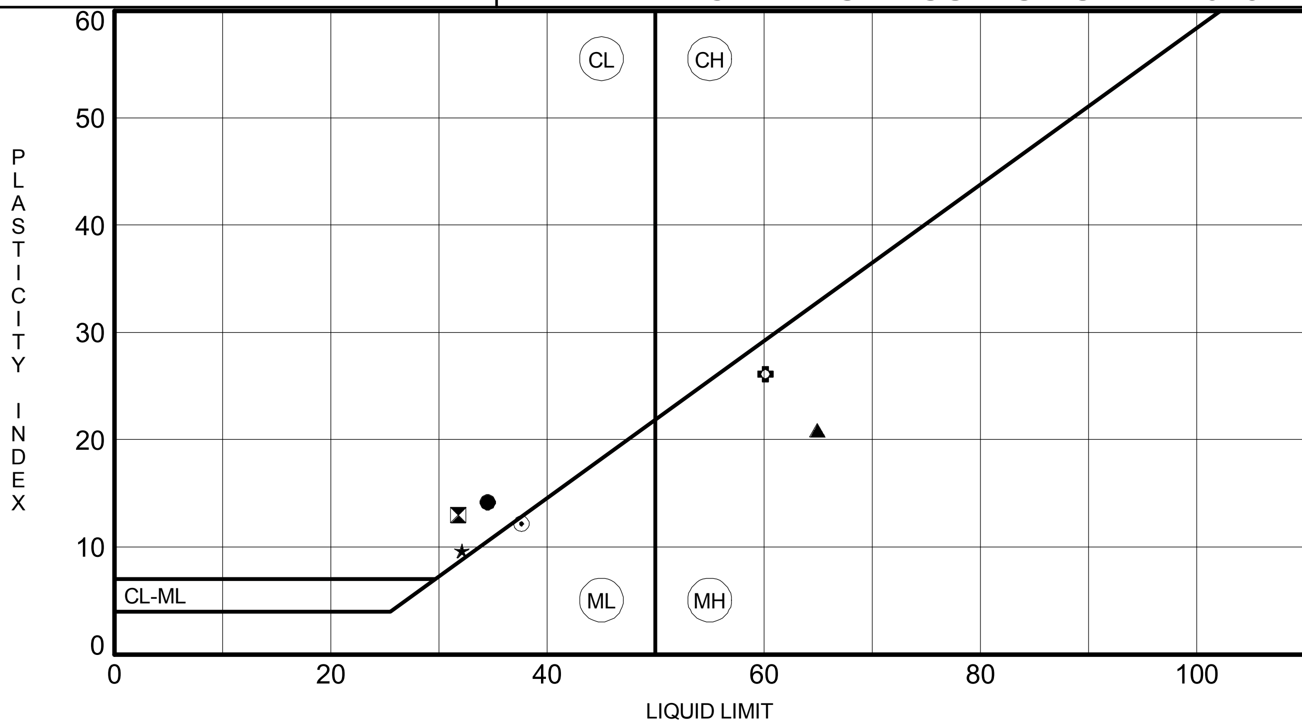
Total Water Consumed



APPENDIX C LABORATORY TESTING

PROJECT NAME: Hearst Park and Pool
PROJECT NO.: 02.02340.02
LOCATION: Washington, DC
CLIENT: Cox, Graae and Spack Architects

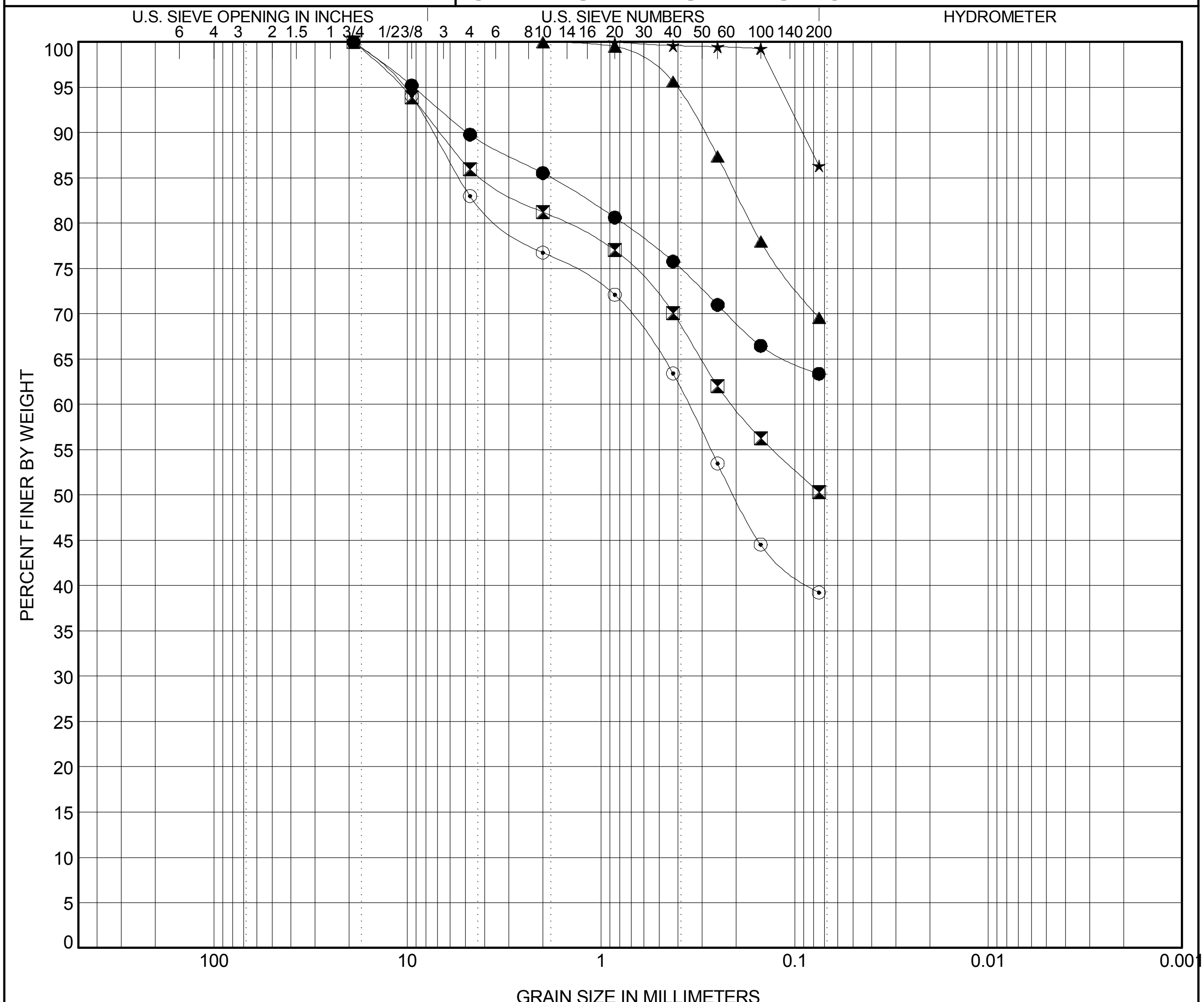
ATTERBERG LIMITS RESULTS ASTM D4318



	SAMPLE	DEPTH (FT)	LL	PL	PI	Fines	SAMPLE DESCRIPTION
●	**B-10-S-3**	**5.0-6.5**	**34**	**20**	**14**	**63.4**	Tan, Sandy Lean Clay (**CL**)
⧗	**B-5-S-5**	**13.5-15.0**	**32**	**19**	**13**	**50.3**	Brown, Sandy Lean Clay (**CL**)
▲	**B-6-S-4**	**8.5-10.0**	**65**	**44**	**21**	**69.6**	Brown, Sandy Elastic Silt (**MH**)
★	**B-7-S-4**	**8.5-10.0**	**32**	**22**	**10**	**86.4**	Brown, Lean Clay (**CL**)
⊙	**B-8-S-5**	**13.5-15.0**	**38**	**25**	**13**	**39.2**	Tan, Silty Sand With Gravel (**SM**)
✪	**B-9-S-5**	**13.5-15.0**	**60**	**34**	**26**	**64.2**	Gray, Sandy Elastic Silt (**MH**)

PROJECT NAME: Hearst Park and Pool
PROJECT NO.: 02.02340.02
LOCATION: Washington, DC
CLIENT: Cox, Graae and Spack Architects

GRAIN SIZE DISTRIBUTION



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

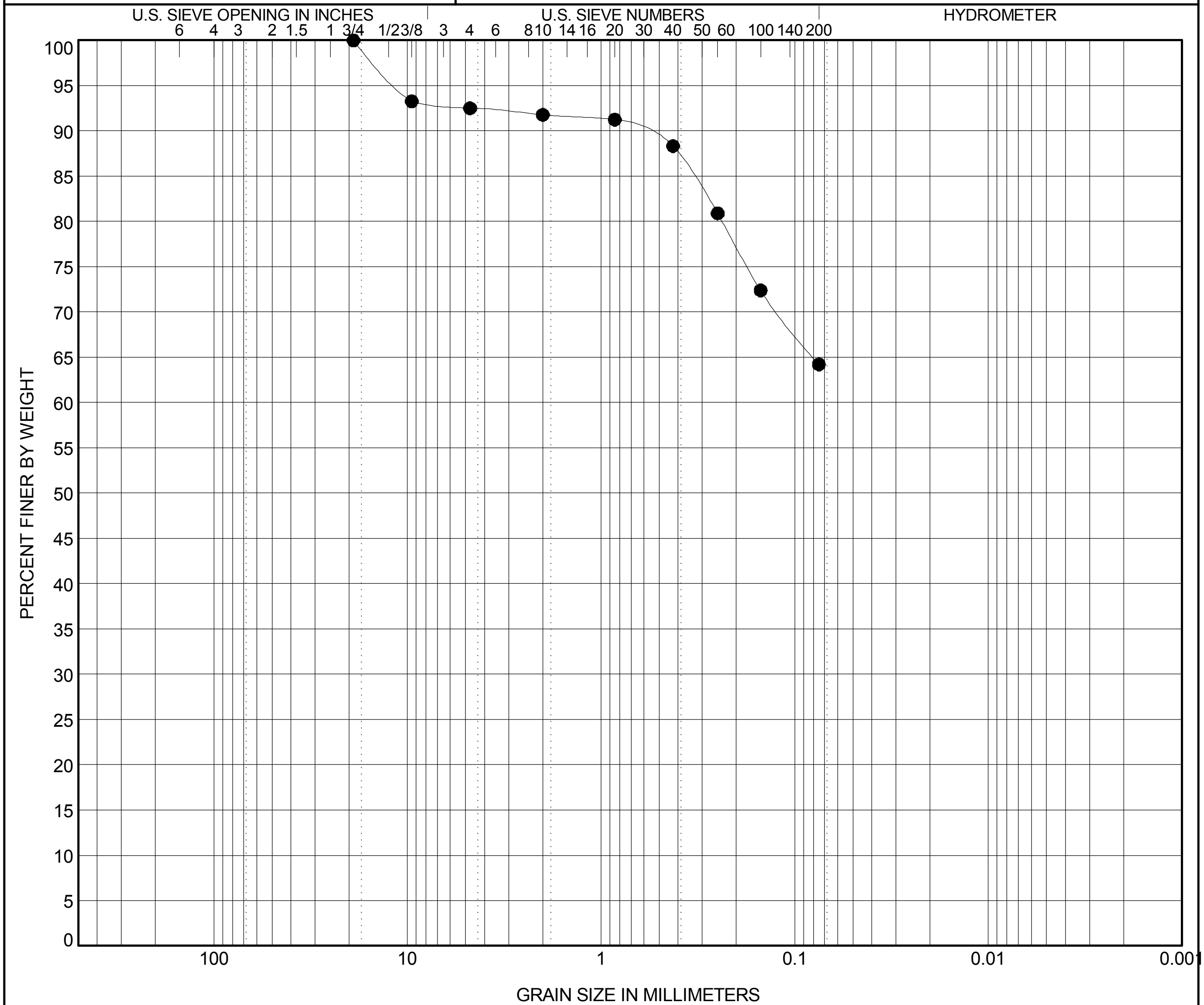
	SAMPLE	DEPTH (FT)	SAMPLE DESCRIPTION	LL	PL	PI
●	**B-10-S-3**	**5.0-6.5**	Tan, Sandy Lean Clay (**CL**)	**34**	**20**	**14**
⧗	**B-5-S-5**	**13.5-15.0**	Brown, Sandy Lean Clay (**CL**)	**32**	**19**	**13**
▲	**B-6-S-4**	**8.5-10.0**	Brown, Sandy Elastic Silt (**MH**)	**65**	**44**	**21**
★	**B-7-S-4**	**8.5-10.0**	Brown, Lean Clay (**CL**)	**32**	**22**	**10**
⊙	**B-8-S-5**	**13.5-15.0**	Tan, Silty Sand With Gravel (**SM**)	**38**	**25**	**13**

		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-10-S-3**	**19**				**10.2**	**26.4**	**63.4**	
⧗	**B-5-S-5**	**19**	**0.209**			**14.0**	**35.7**	**50.3**	
▲	**B-6-S-4**	**2**					**30.4**	**69.6**	
★	**B-7-S-4**	**0.85**					**13.6**	**86.4**	
⊙	**B-8-S-5**	**19**	**0.354**			**17.0**	**43.8**	**39.2**	

GRAIN SIZE 11/7/17

PROJECT NAME: Hearst Park and Pool
PROJECT NO.: 02.02340.02
LOCATION: Washington, DC
CLIENT: Cox, Graae and Spack Architects

GRAIN SIZE DISTRIBUTION



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	SAMPLE	DEPTH (FT)	SAMPLE DESCRIPTION	LL	PL	PI
●	**B-9-S-5**	**13.5-15.0**	Gray, Sandy Elastic Silt (**MH**)	**60**	**34**	**26**

		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**B-9-S-5**	**19**				**7.5**	**28.3**	**64.2**	

GRAIN SIZE 11/7/17

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Dulles, Virginia 20166
tel: (703) 665-0586 fax: (301) 768-4169

SUMMARY OF LABORATORY RESULTS

CLIENT Cox, Graae and Spack Architects

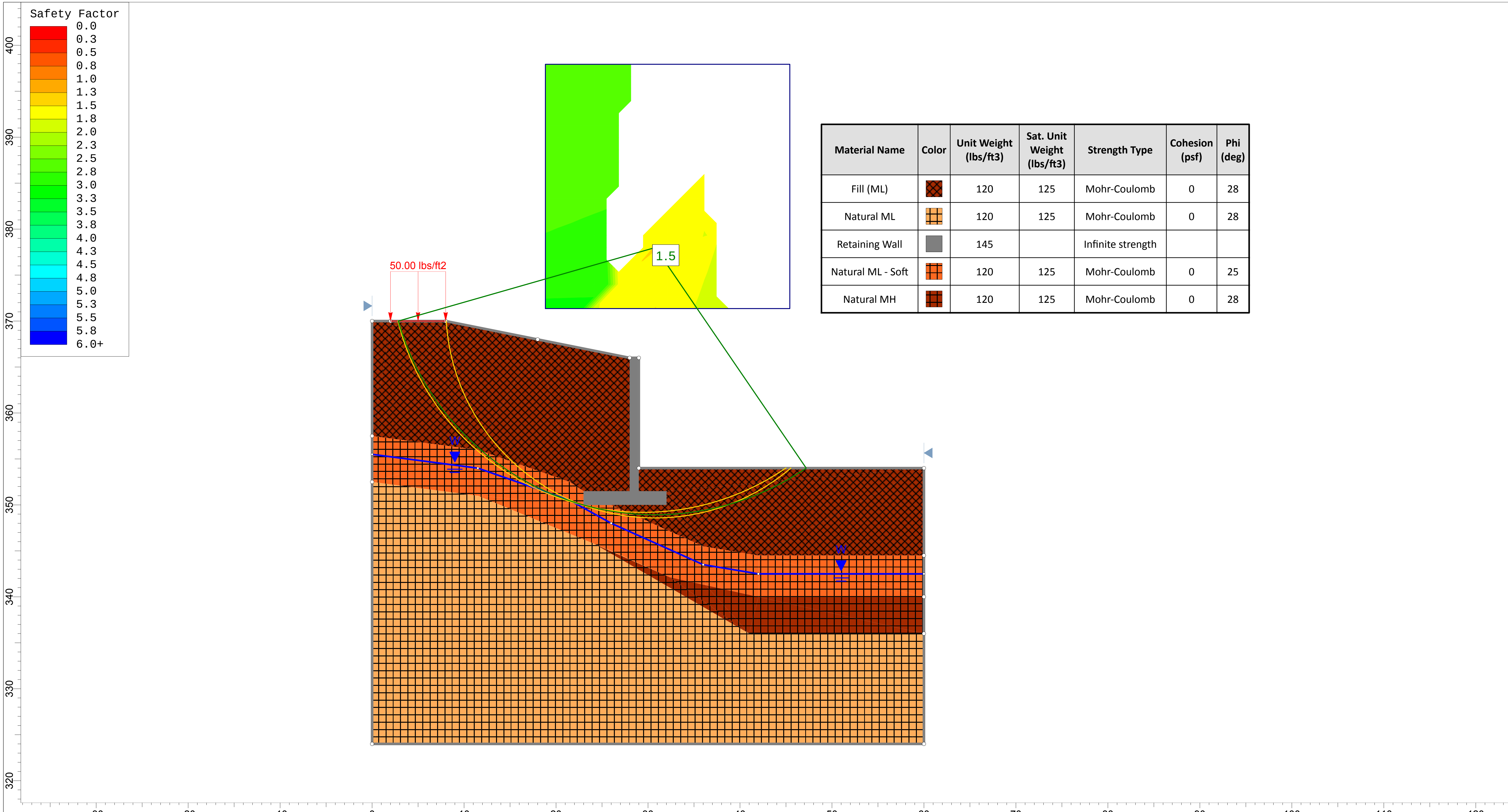
PROJECT NAME Hearst Park and Pool

PROJECT NUMBER 02.02340.02

PROJECT LOCATION Washington, DC

Sample ID	Depth (FT)	Liquid Limit	Plastic Limit	Plasticity Index	%<#200 Sieve	Water Content (%)	Proctor Method	Max Dry Density (pcf)	Optimum Moisture (%)	Oversize Fraction (%)	Sample Description/Classification
B-1-S-4	8.5 - 10.0					20.6					Brown,
B-10-S-3	5.0 - 6.5	34	20	14	63.4	14.7					Tan, Sandy Lean Clay (**CL**)
B-2-S-3	5.0 - 6.5					13.3					Brown,
B-5-S-5	13.5 - 15.0	32	19	13	50.3	19.3					Brown, Sandy Lean Clay (**CL**)
B-6-S-4	8.5 - 10.0	65	44	21	69.6	43.3					Brown, Sandy Elastic Silt (**MH**)
B-7-S-4	8.5 - 10.0	32	22	10	86.4	28.2					Brown, Lean Clay (**CL**)
B-8-S-5	13.5 - 15.0	38	25	13	39.2	26.6					Tan, Silty Sand With Gravel (**SM**)
B-9-S-5	13.5 - 15.0	60	34	26	64.2	42.7					Gray, Sandy Elastic Silt (**MH**)

APPENDIX D GLOBAL STABILITY ANALYSIS SECTIONS AND DATA



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Fill (ML)		120	125	Mohr-Coulomb	0	28
Natural ML		120	125	Mohr-Coulomb	0	28
Retaining Wall		145		Infinite strength		
Natural ML - Soft		120	125	Mohr-Coulomb	0	25
Natural MH		120	125	Mohr-Coulomb	0	28

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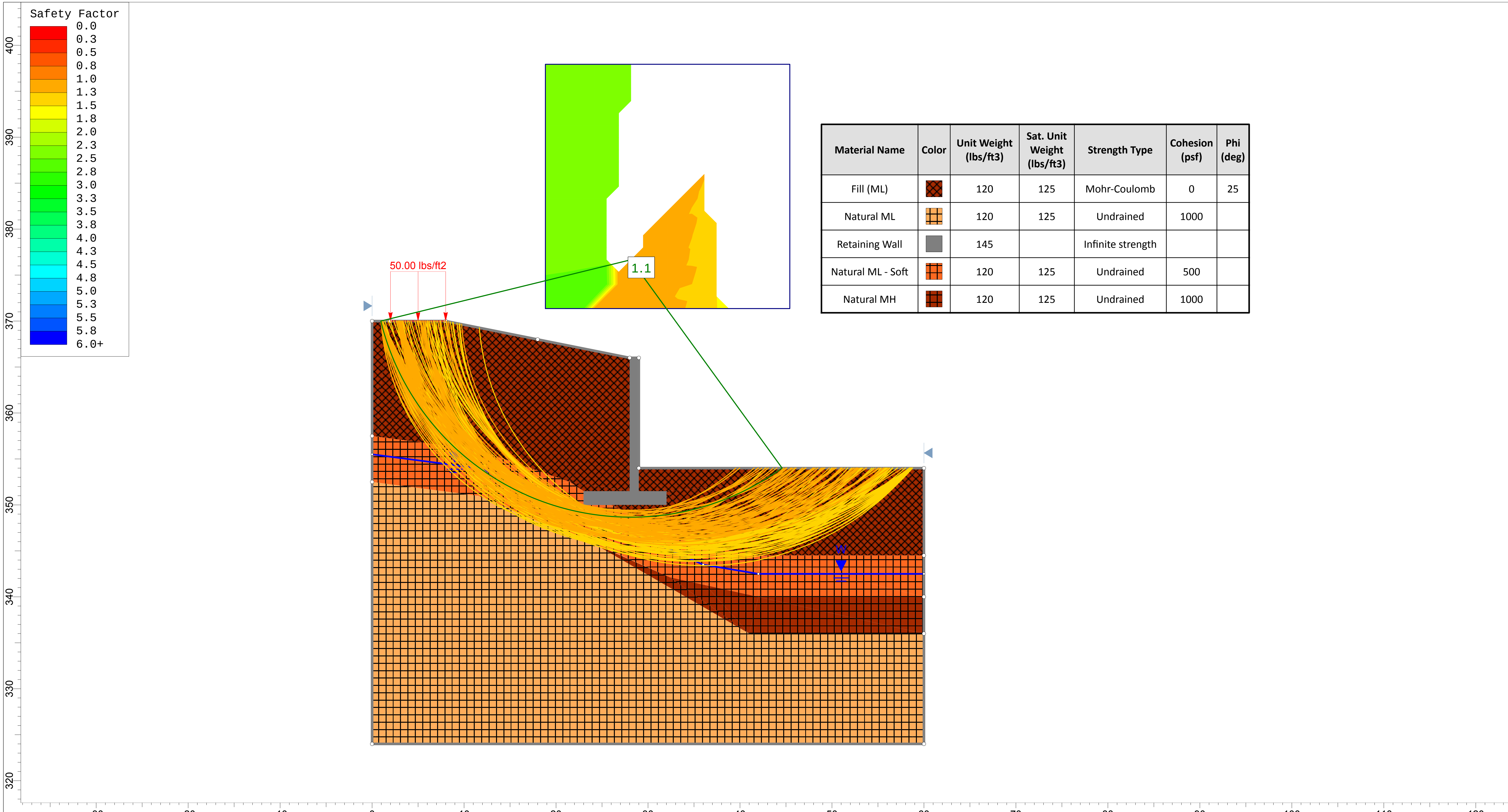
SLIDEINTERPRET 7.017

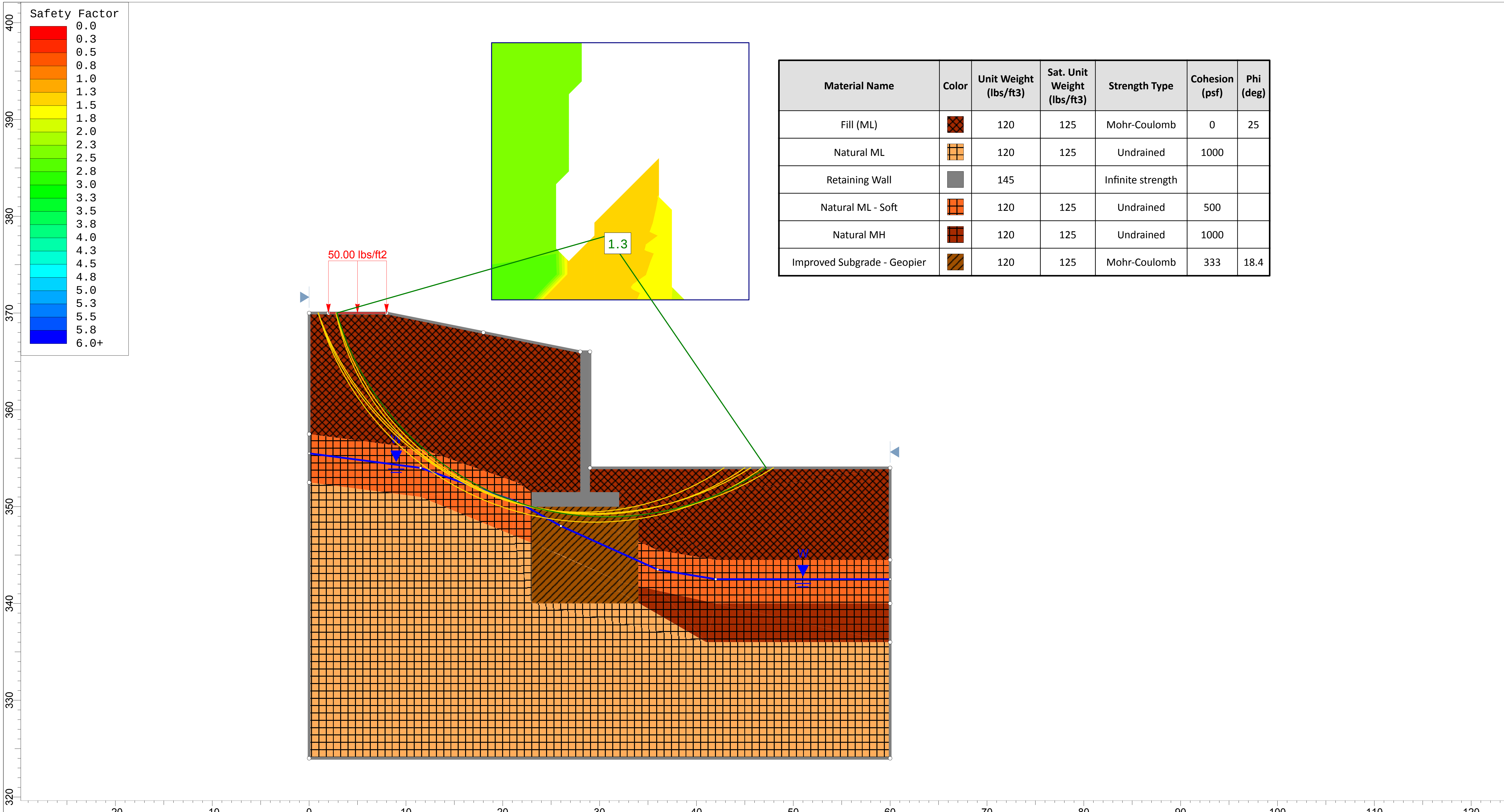
Project	Hearst Park and Pool
Analysis Description	Slope Stability Analysis (Section A-A') - Drained Condition
Drawn By	S. Ghany
Scale	1:120
Company	DMY Engineering Consultants,
Date	11/16/2017
File Name	Inc. Section A-A'.slim



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Fill (ML)		120	125	Mohr-Coulomb	0	25
Natural ML		120	125	Undrained	1000	
Retaining Wall		145		Infinite strength		
Natural ML - Soft		120	125	Undrained	500	
Natural MH		120	125	Undrained	1000	

Project	Hearst Park and Pool		
Analysis Description	Slope Stability Analysis (Section A-A') - Existing Undrained Condition		
Drawn By	S. Ghany	*Scale*	1:120
Company	DMY Engineering Consultants,		
Date	11/16/2017	*File Name*	Inc. Section A-A'.slim

SLIDEINTERPRET 7.017



Material Name	Color	Unit Weight (lbs/ft3)	Sat. Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)
Fill (ML)		120	125	Mohr-Coulomb	0	25
Natural ML		120	125	Undrained	1000	
Retaining Wall		145		Infinite strength		
Natural ML - Soft		120	125	Undrained	500	
Natural MH		120	125	Undrained	1000	
Improved Subgrade - Geopier		120	125	Mohr-Coulomb	333	18.4

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SLIDEINTERPRET 7.017

Project	Hearst Park and Pool		
Analysis Description	Slope Stability Analysis (Section A-A') - Undrained with Rammed Aggregate Piers		
Drawn By	S. Ghany	*Scale* 1:114	*Company* DMY Engineering Consultants,
Date	11/16/2017		*File Name* Inc. Section A-A'.slim