APPENDIX G2. GEOTECHNICAL DESIGN ANALYSIS

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GEOTECHNICAL DESIGN ANALYSIS

HUNTINGTON FLOOD DAMAGE REDUCTION STUDY, VA

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1. Study Area.

The area of study is located in Huntington and Huntington Station subdivisions, which is located in the north-east corner of Fairfax County, Virginia. The Huntington Community lies along the south bank of Cameron Run, just upstream of the confluence of Cameron Run and the Potomac River. The community is located within the 100-year flood plain, which is susceptible to riverine and tidal flooding. After the development and evaluation of the final concept plans during Phase 3, Fairfax County selected Plan 2c. The geotechnical study for the flood damage reduction project generally consists of subsurface investigation, testing, and the design of the levee embankments based on Plan 2c.

2. Geology.

The project site is located along the western edge of the Coastal Plain Province. This province is bordered to the west by the Piedmont Province and to the east by the Atlantic Ocean and Continental Shelf. The Coastal Plain sediments are made up of Quaternary, Tertiary, and Cretaceous deposits. These sediments lie on top of crystalline rock of the Piedmont Province which slopes seaward. Along the fall line, which forms the divide between the Piedmont and Coastal Plain, the deposits are only a few feet in thickness. The deposits form a thickening wedge to the east that reaches depths of up to 10,000 feet along the east coast in Maryland and Virginia. Regionally, the geology has been affected by sea level fall and rise as well as respective decrease and increase river flow.

The Potomac Group crops out in the immediate vicinity of the project site and consists of a complex series of ancient deltaic deposits varying from massive clays to interlayered sands, gravels, silts and clays. Meandering rivers and distributary channels deposited gravel, sand, silt, and clay during the early Cretaceous in a delta setting, with massive clays deposited along the leading edge of the delta. This formation is characterized by crossbedding, channel fills, rapid pinching and swelling of beds, and abrupt changes in size grading. In the area of the project site, clays of the Potomac Formation tend to be more montmorillonite rather than the mainly kaolinite and illite deposits which occur to the north. During the late Tertiary, base level changed and the area was subjected to erosion and fluvial deposition. Broad alluvial terraces were formed on the Potomac Formation in the late Tertiary with sand and quartz gravel. As terraces were formed and base levels changed, sliding and slumping of the Potomac clay occurred repeatedly until stability was reached. The most prevalent geologic processes taking place in the Coastal Plain today are creep, slumping, and sliding of Potomac Formation clays and slumping and sliding of Quaternary sediments.[1,2]

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[1] Obermeier, S. F., Swanson, P. G., Jones J. S. Jr., and Schnabel, J. J., 1984, Engineering Geology and design of Slopes for Cretaceous Potomac Deposits in Fairfax County, Virginia, and Vicinity: Geological Survey Bulletin 1556.

[2] Obermeier, S. F., and Langer, W. H., 1985, Relationships Between Geology and Engineering Characteristics of Soils and Weathered Rocks of Fairfax County and Vicinity, Virginia: Geological Survey Professional Paper 1344.

The complexities of the Potomac Formation bring different issues to light when considering them in an engineering project. Because of the large and abrupt variations in lithology of the Potomac Formation, composition and grain size of soils vary drastically over short horizontal distances.[1] Deposits may be lenticular and pinch out laterally, or may be truncated and cut by channel in-fills of vastly different character. The highly plastic montmorillonite clay is subject to high volume changes (shrink/swell), typically has low residual shear strength and can cause problems with stability in natural and engineered slopes. Shrink/swell is often a problem for shallow foundations and pavements.

3. Subsurface Exploration.

a. General. The subsurface exploration program was performed by the Baltimore District between October 2007 and May 2008. In October 2007, 4 test pits were excavation within the vicinity of the proposed alignment. Between March and May 2008, ten borings were drilled. In addition to the recent subsurface exploration performed by the Corps, supplementary subsurface data was obtained from other studies performed in the 1970s and early 1980s, which are described below.

b. Drilling. The drilling program performed by the Baltimore District consisted of a total of 10 borings. The holes were drilled along the proposed levee alignment for Concept Plan 2c, which parallels Cameron Run. Boring depths varied from approximately 30 feet to 60 feet. Drilling was accomplished by standard penetration test procedures using hollow-stem augers to advance the holes. Split spoons samples were generally obtained at 2.5-foot intervals, except for the top 12 feet where continuous sampling was performed. At certain locations, 3-inch diameter Shelby tubes were pressed to obtain undisturbed samples of fine-grained foundation clays. Several attempts were made to obtain Shelby tubes; however the Potomac clays are very stiff and can contain varying amounts of gravel, making it very difficult to obtain undisturbed samples. Only 2 Shebly tubes were obtained in DH-4B, located approximately 5 feet from DH-4. In addition, constant and falling head field permeability tests were performed in 2 borings to determine permeability values for the pervious sand stratum beneath the proposed embankment. Borings logs are presented in Sub-Attachment A, Drill Logs. Also, boring locations and soil profiles are shown on the enclosed Subsurface Boring Plans and Profile Figures.

c. Test Pits. In October 2007 prior to the drilling program, 4 test pits were excavated within the vicinity of the proposed alignment. Large bucket samples were obtained from the excavation for classification testing. Test pit logs are presented in Sub-Attachment A, Drill Logs.

d. Exploration by Others. As stated above, supplementary subsurface data was also obtained from other studies performed in the 1970s and early 1980s. This supplementary drilling data was obtained from the "Arlington Terrace Storm Drainage Study, dated April 1982." The study contained subsurface information on borings and samples taken in 1981 for the Cameron Run-Huntington Flood Control Project and

contain drilling logs of soil borings performed in 1972 for the Westgate Pump Power Project. The subsurface exploration performed in 1981 for the flood control project consisted of 11 borings, approximately 10 to 16 feet deep, drilled directly adjacent to the riverbank using a hand probe sampler. In addition, 9 soil samples were obtained from the river bottom for gradation analysis. In 1972, 4 borings (22' to 50' deep) were drilled in the vicinity of the proposed downstream levee tie-out. Soil descriptions, borings logs and laboratory test data from these other subsurface explorations are presented in Sub-Attachment C, Supplemental Subsurface Exploration Data.

4. Laboratory Testing.

All disturbed soil samples from the recent drill holes were tested at the Baltimore District Material & Instrumentation Unit. The samples were visually classified, and representative samples were tested for Atterberg limits, moisture contents, and mechanical analyses. Direct shear and unconfined compression testing were performed on material taken from the undisturbed Shelby tube samples. Mechanical analyses, unit weight, moisture content, and Atterberg limits were determined for each of the undisturbed samples. The direct shear testing was performed at the Penniman and Browne, Inc. Laboratory. Two direct shear tests were performed on one of the Shelby tube samples. The first direct shear test was performed on undisturbed samples taken from the tube, and the second set of direct shear tests was performed on previously sheared samples to determine the residual shear strength of the material. Additional discussion on the materials strength properties is provided below. Physical property soil test results are presented in Sub-Attachment B, Laboratory Test Results.

5. Existing Foundation Condition.

a. Foundation Conditions beneath Levee Centerline. Subsurface exploration indicates the existence of three general foundation overburden zones within the top 60 feet along the centerline of the levee embankment alignment. The top stratum consists of an upper blanket zone of silt, clay and silty or clayey sand. This blanket material varies in thickness from 5 to 16 feet, with an average thickness of 8 feet. In general, the blow counts/foot (N) for the upper blanket material ranged from 3 to 20, with an average N value of 6 for most of the material. Beneath this blanket stratum is a pervious stratum of silty sand and gravel varying from 2 to 13 feet thick: however, at drill holes DH-1 and 4, the poorly graded sand and gravel stratum was not encountered and only a 2-foot thick zone of silty gravel was encountered in DH-2. Mechanical sieve analysis performed on soil sample of sand and gravel material showed that the material contained 3 to 12 percent fines (material passing the 200 sieve). The blow counts (N) for the pervious sand and gravel stratum ranged from 10 to 48, with an average N value of 15 to 20 for most of the material. Beneath the top two strata is thick deposit of very hard, lean to fat clay, which continued to the bottom of the borings. The clay deposit had varying amounts of sand and gravel. In some borings, seams of clayey, silty sand were encountered within the clay deposit. The blow counts (N) for the clay deposit ranged from 14 to 50+, with an average N value of 20 to 30 for most of the material. Groundwater was generally encountered approximately 4 to 12 feet below the ground

surface, with the average depth of groundwater at about 5 feet. At the upstream and downstream ends of the alignment, a thin layer (less than 2' thick) of soft clay and organic silt was encountered between 2 to 8-feet below the ground surface. This soft material had blow counts (N) ranging from 2 to weight of hammer.

The test pits, which were excavated to a depth of 6-feet, encountered materials similar to those encountered in the borings. The material consisted of sandy clay (CH) or clayey sand (SC) both containing gravel. At the upstream end of the alignment, TP-1 encountered a 1-foot thick layer of elastic silt, having an organic content of 8.3%. This is very similar to the thin elastic silt encountered in DH-2. In TP-1, groundwater was observed slowly entering the test pit at the bottom of the excavation.

b. Foundation Conditions along Riverbank and Riverbed. As stated above, 11 borings were drilled along the river channel in 1981, which was being considered for a potential alignment of a levee embankment. The 1982 Preliminary Soils Investigation report stated the following findings: "Boring depths ranged from 10-15.5 feet and were generally stopped at the depths by gravels or other obstacles. Seven of the 11 borings show an upper layer of poorly graded loose sands at depths of 2 to 5 feet. The other 4 borings have surface layers containing soft silty clay or soft organic silt. All boring show substantial subsurface strata of soft and very soft fibrous peat or organic silts. The total depth and thickness of the organic layers generally increased with distance downstream along the [alignment]." Since the underlying strata along the river contain a very substantial, highly compressible deposit of soft peat and organic silt, significant long term and differential settlement would present problems for a levee embankment. For this 1982 study, samples were also taken from the river bed to classify the soil type for potential used as levee embankment material. Within the upstream and downstream limits of the proposed levee embankment alignment, the channel sediments consist of a poorly graded fine and medium sands. Farther downstream of the proposed levee embankment, the channel sediments consist primarily of silt and clay.

Soil profiles are shown on the enclosed Subsurface Boring Plans and Profile Figures.

6. Alternative Evaluation and Plan Selection.

Several alternatives were developed and considered to provide adequate flood damage reduction measures for the Huntington Community. The plan formulation process was divided into 3 phases, with each phase developing, evaluating, and eliminating alternatives in order to identify the most cost-effective flood damage reduction plan for the County to implement. A complete description of the plan formulation for this project is presented in the main text. The geotechnical analysis evaluated those plans that would directly impact the levee alignment and embankment design. During Phase 1, two levee alignments were considered. The first levee alignment located the levee embankment as close to Cameron Run as possible. The second levee alignment located the levee farther away from the river bank to minimize impacts to the adjacent wetlands. Based on geotechnical concerns, the levee alignment located along the riverbank was not desirable because of the extremely poor foundation condition encountered along the riverbank. As

described above, the underlying strata along the riverbank contain a highly compressible deposit of soft peat and organic silt. This condition would present problems with the stability and settlement of the embankment slope. Due to the potential for significant long term and differential settlement of the soft peat and organic silt layer, overbuilding the levee to accommodate for any settlement would most likely be required. Furthermore, dewatering and the soft foundation issues would make construction of the levee embankment along the riverbank extremely difficult and costly.

The second levee alignment is located farther away from the river bank, which provides many benefits. This alignment minimizes impacts to the adjacent wetlands. The foundation conditions are more suitable for constructing an embankment. In additional, the total levee height is smaller since the alignment is located where the ground surface is at a higher elevation than along the riverbank. The design and analysis of the levee embankment and foundation were based on the second alignment and is presented in the following paragraphs.

7. Levee Design

a. General. The design of the levee embankment is based on the selected Plan 2c, which will consist of an earth levee embankment and drainage structures. The levee design was performed in accordance with guidance contained in EM 1110-2-1913, Design and Construction of Levees (30 April 2000). Detailed descriptions and results for the various geotechnical analyses are provided in the Sub-Appendices.

b. Levee Section: A typical cross section of the levee embankment is shown on Figure 4. The proposed embankment has a 10-foot wide crest and 1 vertical on 2.5 horizontal (1.0V:2.5H) side slopes. A 6-foot deep trapezoidal impervious cutoff/inspection trench will be located beneath the levee along the alignment centerline. The embankment will be constructed primarily using select fill material from a borrow source obtained by the contractor. The select fill zone, shown on Figure 4, will consist of impervious soils available in the area. The select fill (impervious earth) materials would consist of clays, silty or sandy clays, clayey sands and gravels that contain at least 25% by weight passing the No. 200 mesh sieve and have a plasticity index greater than 5 but less than 30.

A combination blanket and toe drain will be placed along the landside toe of the levee embankment. A typical blanket and toe drain detail is shown on Figure 4. The combination blanket and toe drain will intercept seepage through the semi-pervious foundation blanket layer and will reduce potential uplift pressures along the base of the levee. The blanket drain will also provide a means of collecting any possible internal seepage and provide an exit for the collected seepage at the landside levee toe. The blanket drain will be 18-inches thick and will consist of fine aggregate similar to an ASTM C 33 Fine Aggregate. A trapezoidal toe drain will be placed to a depth of about 5 feet below the existing ground surface at the levee toe at elevation 0.0. An 8-inch perforated plastic pipe will be installed in the toe drain to collect and convey the seepage flow beyond the levee toe. The toe drain material will consist of coarse, high

permeability gravel similar to an AAHSTO No. 57 coarse aggregate. The gravel toe drain will be surrounded by geotextile filter to prevent contamination of the gravel by the fine grained foundation soils. The fine and coarse aggregates for the drains will be obtained from off-site commercial sources.

At certain location, ramps will be constructed to provide access onto and over the levee for maintenance and recreation activities. The ramps will be constructed with random fill, consisting of a range of materials (gravels, sands, silts, clays or combination). The surface of the ramps and levee crest will be paved.

c. Design Parameters for Foundation Soils. Geotechnical design parameters used in the various analyses were based on blow count data, laboratory test results, and from observations made during the drilling operation. Various studies and reports were used to assist in determining soil parameters. These reports provide correlations between strength properties of soils based on soil index properties and in-situ testing. The table below shows the typical ranges of effective friction angles used in the stability analyses:

Zone/Stratum	Blow Counts (N)	Effective Angle of Internal Friction, ϕ',
Clayey Silty Sand, Sandy Clay Blanket	3 to 20, avg. range = 6 to 10	28° to 32°, avg.= 30°
Sand & Gravel Aquifer Layer	10 to 48, avg. range =15 to 20	32° to 36°, avg.= 34°
Lower lean to fat Clay	14 to 50+, avg. range = 20 to 30	See below lab test results

The lower clay deposit encountered is part of the Potomac Formation; the clays are very stiff and are highly overconsolidated. Shear strength testing generally shows the Potomac clays to have high effective peak strengths; however, the residual shear strength can be significantly less than the peak strength due to saturation of the clay, shearing along previously sheared, or the soil being highly fractured. For this project, 2 Shelby tubes were also obtained from the lower clay deposit in order to performed unconfined compression and direct shear testing. One unconfined compression test and 2 direct shear tests were performed. The first direct shear test was performed on an undisturbed sample, and the second test was performed on previously sheared samples from the first test. The samples were resheared along the same failure plane to determine the residual shear strength of the clay material. The test values (peak and residual) obtained for both tests are consistent with published data for the Potomac clay material. Below are the results of the testing.

Sample	Depth	Material Class.	q_u (tsf)	Dry Density	Water content	PI	PL
DH-4B Shelby-1	22.0’ – 24.0’	Fat Clay (CH)	1.48	95.5 pcf	25.3	26	24

Sample	Material Class.	Dry Density	Water content	PI	PL	Direct Shear Test	ϕ’
DH-4B Shelby-2 28.0’ – 30.0’	Fat Clay (CH)	95.5 pcf	28.2	31	28	Undistrubed	30°
						Resheared	14°

As stated above, constant and falling head field permeability tests were performed in 2 wells. A detailed description of the test procedures and results are provided in Sub-Attachment D, Hydraulic Conductivity Field Testing. The well screens were installed in the previous foundation zone, which consisted of poorly graded sand with gravel and silt such as SP and SP-SM classified in accordance with the Unified Soil Classification System. In general, the testing yielded the following:

Well	Material/Zone	Hydraulic conductivity, k
DH-3	Poorly graded Sand w/ gravel (SP)	2×10^{-2} cm/sec (falling)
DH-3	Poorly graded Sand w/ gravel (SP)	4.0×10^{-3} cm/sec (constant)
DH-6	Poorly graded Sand w/ gravel & silt (SP-SM)	3.0×10^{-3} cm/sec (constant)

Based on field testing, visual classification, and mechanical sieve analyses of selective soil samples, a range of hydraulic conductivity values were selected for the various foundation strata encountered. The table below shows the typical ranges of values used in the seepage analysis:

Zone/Stratum	Hydraulic conductivity, k
Clayey Silty Sand, Sandy Clay Blanket	1×10^{-4} to 1×10^{-6} cm/sec
Sand & Gravel Aquifer Layer	1×10^{-3} to 2×10^{-2} cm/sec
Lower lean to fat Clay	1×10^{-6} to 1×10^{-8} cm/sec

d. <u>Design Parameters for Embankment Material.</u> The proposed design for the levee assumes a homogenous embankment, constructed primarily of select earth material from a borrow source obtained by the contractor. The select earth material should generally consist of impervious silty or sandy clays, clayey silts or clayey sands and gravels that contain at least 25% by weight passing the No. 200 mesh sieve and have a plasticity index greater than 5 but less than 30. At this phase of the project design, assumptions were made on the embankment’s shear strength and hydraulic conductivity parameters. The design parameters were based on engineering judgment and experience with the design and analysis of embankments with similar dimensions and function. Generally, the effective shear strength (ϕ’) for a compacted embankment material would range between 28° and 36°. A value of 34° and no cohesion (c=0) was used in the stability analysis. A conservative hydraulic conductivity range of 1×10^{-5} to 1×10^{-6} cm/sec was used in the seepage analysis. Design parameters used in the seepage and stability analyses for the drainage materials were within the normal range of strength (ϕ’= 30° to 34°) and hydraulic conductively (k= 1×10^{-1} to 1×10^{-3} cm/sec) normally obtained for processed aggregate materials.

e. Settlement Analysis: For the proposed alignment, the foundation conditions are considered suitable for embankment construction with very little to no consolidation of the foundation or long term settlement of the levee embankment. The deposit of Potomac clay, which is approximately 15 feet below the ground, is very hard and overconsolidated. The clay deposit is considered to have been preloaded since the natural water contents are closer to the plastic limit than to the liquid limit. During drilling it was extremely difficult to push Shelby tubes and to obtain undisturbed samples for testing. Based on published data, consolidometer test on unweathered samples of Potomac clay material commonly have preconsolidation values 10 to 20 tsf in excess of existing overburden. Based on these conditions, the addition load (0.9 tsf) from the levee embankment will not induce consolidation of the Potomac clay material. The foundation exploration, however, did encounter small isolated pockets of soft organic silt and clay material at the upstream and downstream ends of the project. These soft zones are 1 to 2 feet thick, within 7 feet of the ground surface, and with blow counts of weight of hammer to 2. It is anticipated that these isolated soft zones will fully consolidate during the construction of the embankment and cause no long term settlement of the embankment. Therefore, long term settlement should not occur and overbuilding the embankment to accommodate for settlement is not considered necessary.

f. Seepage Analysis: The embankment was evaluated for seepage through and beneath the embankment. Since the duration of a flood event at or above the design water surface is very brief, it is unlikely that a condition of full saturation of the embankment will occur. However, the seepage analysis did assume a full steady state condition, causing saturation of the levee embankment. Any seepage through the embankment will be collected by the sand blanket drain and discharged at the toe of the levee, which will prevent saturation of the landside portion of the levee embankment. Because of the pervious aquifer and varying thickness of blanket, analysis of underseepage was performed. Preliminary seepage analysis was performed using techniques presented in EM 1110-2-1913, Design and Construction of Levees (30 April 2000), Appendix B, “Mathematical Analysis of Underseepage and Substratum Pressures. This analysis indicated that the uplift pressures and exit gradients at the landside toe of the levee are within acceptable ranges, except at along one reach where the exit gradients were slightly higher. During the design, it was determined that additional area was required for ponding interior drainage run-off. In order to provide space for ponding, the area landward of the levee toe will be excavated to elevation 6.0. The excavation depth varies from 0 to 3 feet and starts about 50 feet from the levee toe and extends to 250 feet away from the levee toe. Because of the pervious sand and gravel stratum below the thin blanket layer, a toe drain was included to relieve excess hydrostatic pressures that could develop beneath the blanket layer. The toe drain also collects seepage and relieves seepage pressures that could develop farther landward in the vicinity of the ponding area. Additional underseepage analysis was performing in order to gain an understanding of the flow rates and the magnitudes of hydrostatic uplift pressures acting beneath the blanket layer and the levee foundation. A finite element seepage program, SEEP/W 2007, developed by GEO-Slope International was used to analyze seepage conditions. The hydraulic conductivity values for the foundation materials were varied to determine a range of possible results. In addition, the sand blanket drain and gravel toe drain beneath

the levee embankment were also included in the typical section being analyzed. The analysis showed that the excess hydrostatic pressures would dissipate through the blanket layer and be controlled by the drains. The results from the seepage analysis are provided in Sub-Attachment E, Seepage Aanlysis.

g. Stability Analysis: For small levee embankments less than about 10 to 15 feet high, formal slope stability analyses are generally not performed. The maximum height of the proposed Huntington levee would be about 15 feet on the riverside and 13 feet on the landside; however, a majority of the landside levee embankment is less than than10 feet high due to the additional fill placed along the toe to provide cover for the pump station conduit. Nevertheless, a stability analysis was performed using the same typical levee section and foundation profile that was developed for the seepage analysis. The slope stability seepage program, SLOPE/W 2007, developed by GEO-Slope International was used to analyze stability of the embankment and foundation. The slope stability analyses were performed for the intermediate river stage, sudden drawdown, steady seepage, and end-of-construction conditions. The minimum factors of safety are 1.4 for intermediate river stage and steady seepage, 1.3 for end of construction, and 1.0 for sudden drawdown. Results from the stability analysis showed that the proposed embankment and foundation meet the factor-of-safety requirements as required in EM 1110-2-1913, Design and Construction of Levees (30 April 2000). The results from the stability analysis are provided in Sub-Attachment F, Stability Analysis.

f. Filter Design: The internal drains will collect and control seepage. In order to meet this objective, the drainage material must be designed to retain the protected materials (impervious embankment fill and foundation) and permit the movement of water through the drain and out to the embankment toe. The sand drain for the proposed embankment should be designed to meet the filter design requirements as presented in the Natural Resources Conservation Service (NRCS) Chapter 26 “Gradation Design of Sand and Gravel Filters”. During the development of plans and specifications, the filter design should be accomplished to meet filter criteria for both the embankment and foundation material adjacent to the sand drain. Attached in Sub-Attachment G is a copy of the NRCS filter criteria.

8. Sources of Construction Materials.

The design for the proposed embankment materials should be readily available in the immediate vicinity of the project. The materials required to construct the various structures consist of the following:

- impervious earth and random fills
- processed sand and gravel materials (aggregates)
- riprap
- topsoil
- pavement and sidewalk materials (concrete, asphalt, etc.)
- concrete
- and various geosynthetics and plastic pipe

In the next design phase, consideration could be given to perform an extensive borrow source investigation program, where several borrow areas would be identified for obtaining the various types of earth materials. However, the prospect of developing a government project owned borrow source for the various materials becomes complex due to environmental, cultural, archeological, and real estate agreement issues. Because of these factors and the small quantities that would be required, it is considered reasonable to assume that all the materials would be obtained from commercially available sources. By utilizing existing operating commercial sources, the need and cost for potential studies, investigations, mitigation, permitting, and real estate acquisition or leasing was eliminated. Therefore, the levee embankment will be constructed using materials from borrow sources provided by the contractor, commercial sources, and required excavation, which is the most effective approach. The contractor should be required to provide all necessary permits to ensure acceptability of the site(s).

Descriptions for some of the various materials that will be required are provided below:

- Impervious Earth Fill: Impervious earth fill is required for constructing the select fill zone of the levee embankment. The impervious embankment materials would consist of clays, silty or sandy clays, clayey sands and gravels that contain at least 25% by weight passing the No. 200 mesh sieve and have a plasticity index greater than 5 but less than 30.

- Random Fill: Random fill material will be required for the ramps and to cover the box culvert for the pump station. The random fill can consist of a wide range of materials such as clays, sands, gravels or a combination. The material excavated to create the ponding areas could also be possibly used for random fill.

- Aggregates: Various types of processed aggregates would be used in the project. Aggregates would be used for the sand blanket drain, toe drain, and subbase course beneath pavements. The specifications for the various aggregates should be written to assure that they meet filter requirements, and are aggregates listed in Virginia Department of Transportation, Road and Bridge Specification, or meet ASTM or AAHSTO standards. These materials would be readily available from local commercial suppliers. Below is a table showing various types of aggregates.

Aggregate Type	Specification
Fine (Drainage Fill) Aggregate	ASTM C 33 Fine Aggregate
Coarse (Drainage Fill) Aggregate	AAHSTO Coarse Aggregate #57
Road subbase material	Dense Graded Aggregate 21A or 21B.*

* Virginia Department of Transportation, Road and Bridge Specification

- Riprap. Riprap will mostly be required for drainage channels to prevent erosion. The riprap or stone material would be available from local stone

quarries. Riprap would most likely be specified to meet Virginia Department of Transportation, Road and Bridge Specification's Riprap Class I, II , or AI.

9. General Construction Considerations.

To construct the various features, the usual type of construction equipment will be used, such as dozers, excavators, cranes, compactors, hauling trucks, and other miscellaneous equipment. Materials would be transported to and from the site using normal size on-road hauling (dump) trucks and concrete trucks. Careful consideration of the operations and methods used to construct the various features in the vicinity of utilities and adjacent properties and buildings will be necessary. At certain locations, during placement of fill in low areas, some type of temporary protection such as flood-barriers berms may be required in order to provide the required protection from flooding during a high creek event. In addition, temporary shoring and bracing may be required for some of the structural excavation.

As described above, isolated soft areas were encountered in some of the borings and test pits. If these areas or zones are near the surface and are encountered during the excavation, it may be necessary to remove these compressible and weak materials depending on the thickness and extent of these zones. Another option may be to temporary surcharge the area to induce consolidation of the material.

Access along certain reaches will require phasing and planning of the construction work. This is especially important for the upstream tie-out reach adjacent to the homes at the toe of the proposed levee. The proposed ponding areas can be used as the staging areas and stockpiling of materials adjacent to the site.

10. Next Phase of Design

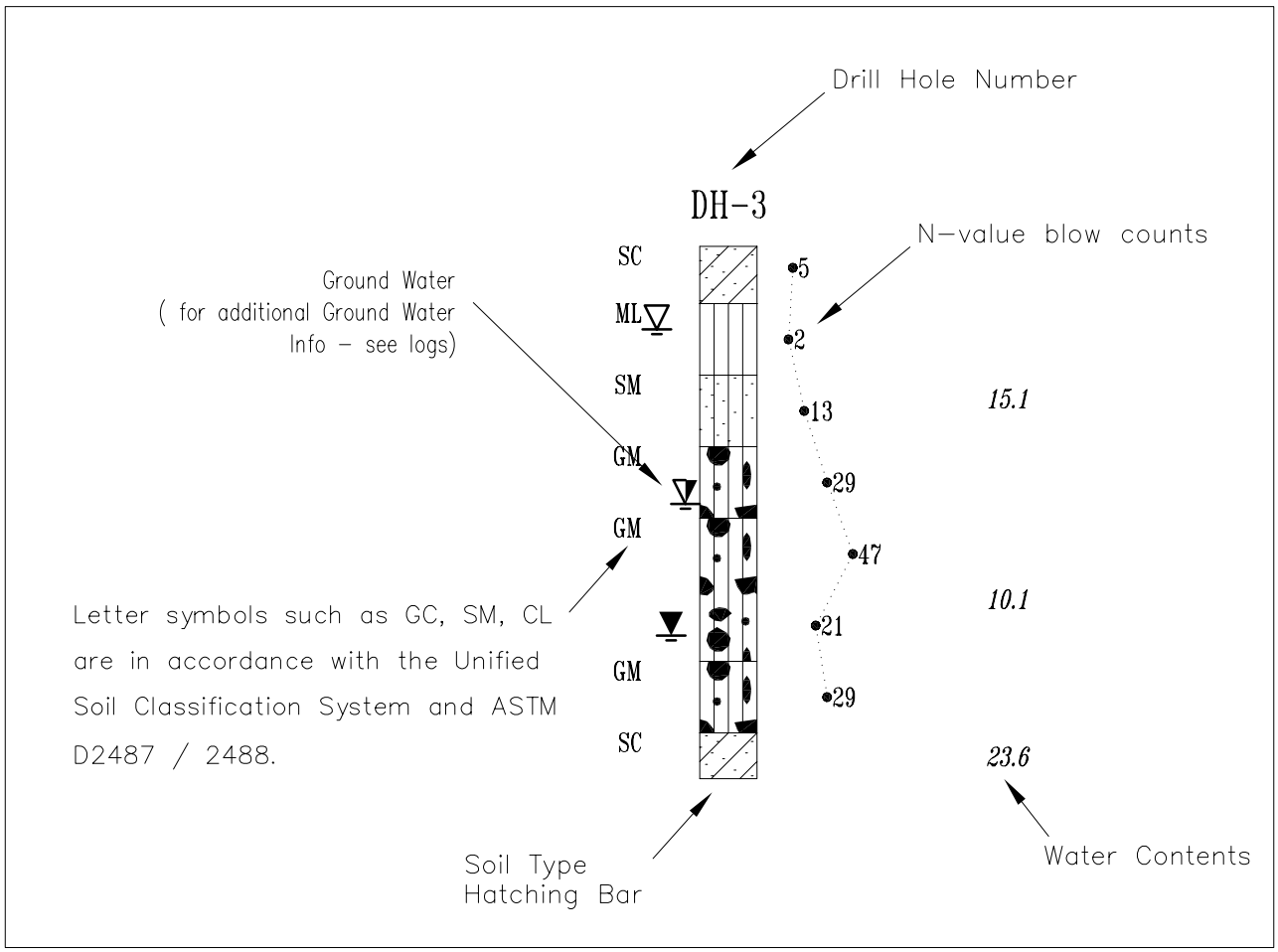
Based on the subsurface investigation and the preliminary design for this study, additional subsurface investigation, testing, and design work will be necessary. Additional drilling and testing will be required in order to provide the necessary data to complete the design for the pump station. Prior to developing construction plans and specifications, additional geotechnical design will consist of performing the appropriate foundation design and analyses for the following features:

- Pump Station
- Drainage Structures
- Filter Design of sand drain and geotextile
- Foundation Design for the Interior Drainage features (manholes, inlets, conduit/piping, swales, etc.)
- Pavement design
- And possibly some additional subsurface investigation (testing pits) to identify any potential soft areas beneath the proposed embankment, especially along the riverside toe.

- Modification to the proposed design and embankment slopes depending on the actual properties of the materials that will be used to construct the embankment.

Prior to completing the construction of the project, O&M manuals should be completed for all features of the system (embankments, drainage structure, pump station, etc).

FIGURES

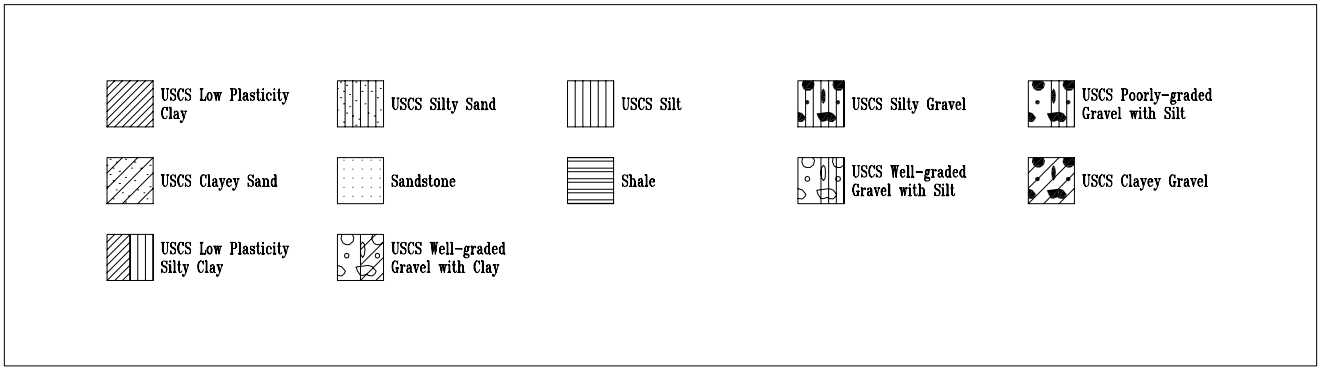


BORING LOG LEGEND

BORINGS BY OTHERS (APRIL 1982)

BORINGS BY CORPS (MARCH 2008)

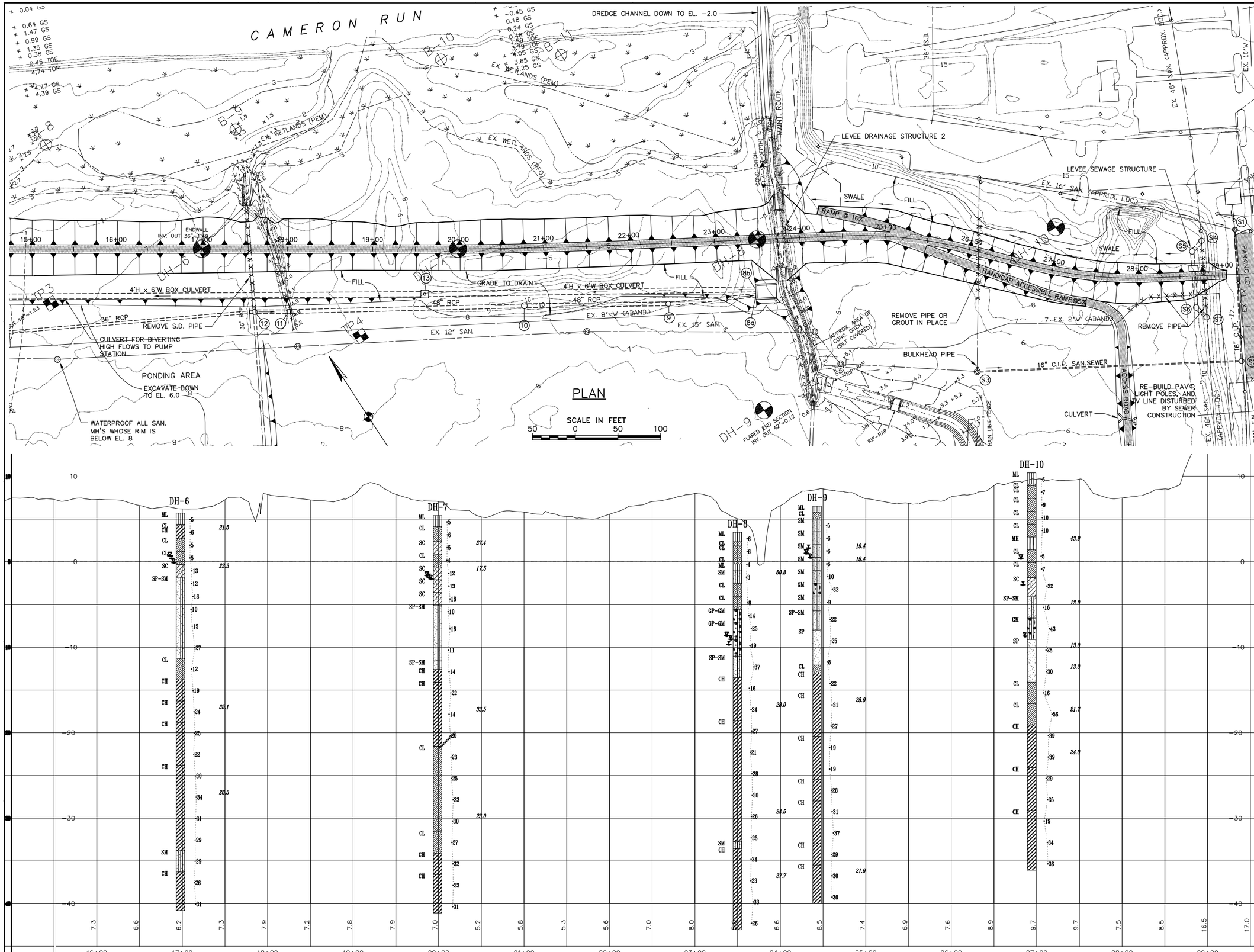
TEST PIT BY CORPS (OCT 2007)



GEOLOGIC PROFILE LEGEND

US Army Corps of Engineers Baltimore District	HUNTINGTON SUBSURFACE LEGENDS	DATE: NOV 2008
		SCALE: Not To Scale
		SHEET

FIG 1



US Army Corps of Engineers
Baltimore District

SYMBOL	REV.	DATE	DESCRIPTION	BY

U.S. ARMY ENGINEER DISTRICT, BALTIMORE
CORPS OF ENGINEERS
BALTIMORE, MARYLAND

PLATE | DRAWING NUMBER XXXXX | FILE NAME 456B104

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HUNTINGTON
FAIRFAX COUNTY, VIRGINIA
FLOOD DAMAGE REDUCTION STUDY

SUBSURFACE BORING PLAN & PROFILE
SHEET 2 OF 2

Sheet Number:
FIG 3

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SYMBOL	REV.	DATE	DESCRIPTION	BY

U.S. ARMY ENGINEER DISTRICT, BALTIMORE
CORPS OF ENGINEERS
BALTIMORE, MARYLAND

PLATE | DRAWING NUMBER XXXXX | FILE NAME 456B103

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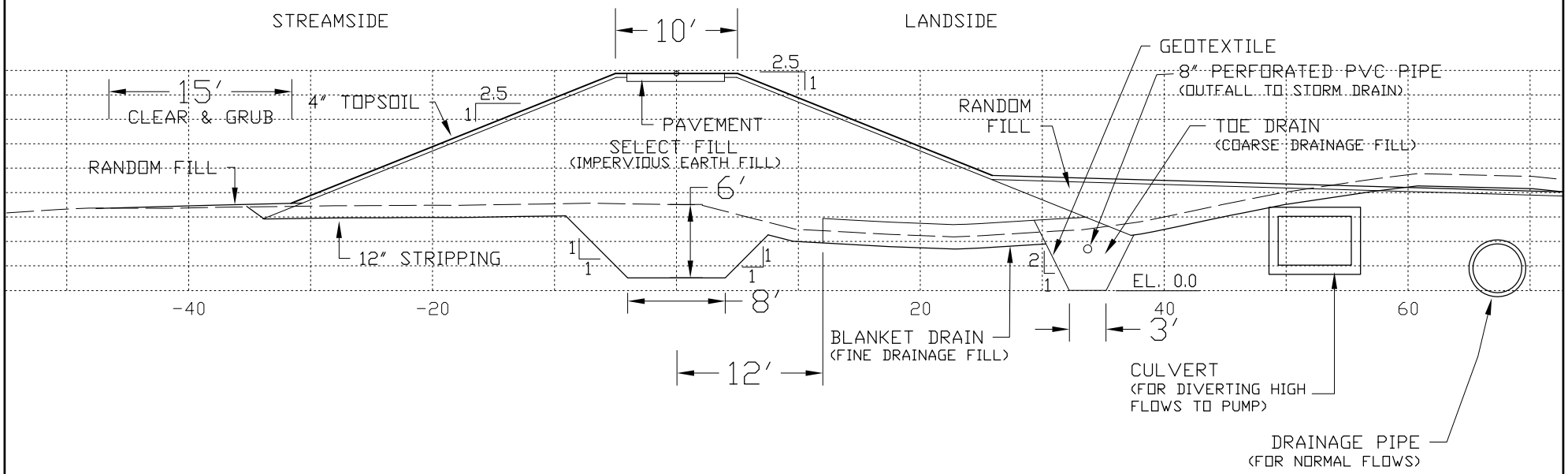
HUNTINGTON
FAIRFAX COUNTY, VIRGINIA
FLOOD DAMAGE REDUCTION STUDY

SUBSURFACE BORING PLAN & PROFILE
SHEET 1 OF 2

Sheet Number:
FIG 2



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Baltimore District

HUNTINGTON
FLOOD DAMAGE REDUCTION STUDY
FAIRFAX COUNTY, VIRGINIA

DATE: OCT 2008
SCALE: NOT TO SCALE
FIGURE 4