Attachment A

Geotechnical Engineering Report

October 24, 2018

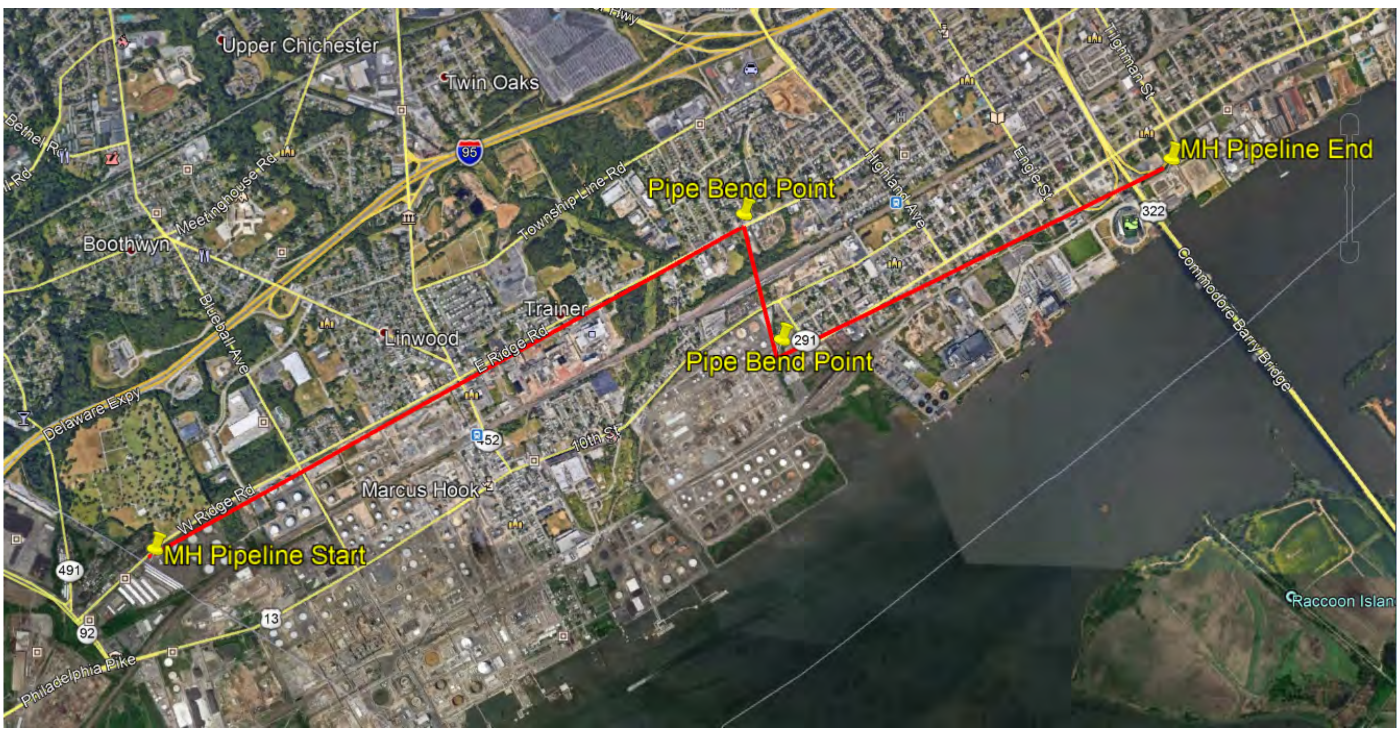
GEOTECHNICAL ENGINEERING REPORT

ADELPHIA GATEWAY, LLC PROPOSED 16” TILGHMAN LATERAL HDD

Delaware County, Pennsylvania

Project No. 18-00672-001

Submitted to:
Hunt, Guillot and Associates



Prepared For:

Hunt, Guillot and Associates, LLC
One Metroplex Drive, Suite 100
Birmingham, Alabama 35209

Prepared By:

Johnson, Mirmiran & Thompson, Inc.
40 Wight Avenue
Hunt Valley, Maryland 21030

October 24, 2018

Mr. Richard C. Healy, P.E.
Hunt, Guillot and Associates (HGA)
One Metroplex Drive, Suite 100
Birmingham, Alabama 35209

RE: **Geotechnical Engineering Report for Tilghman Lateral HDD**
Adelphia Gateway, LLC
Delaware County, Pennsylvania
JMT Job No. 18-00672-002

Dear Mr. Healy:

Johnson, Mirmiran & Thompson, Inc. (JMT) is pleased to submit the results of the geotechnical subsurface investigation and geotechnical engineering recommendations for the above referenced project. This report contains a discussion of our understanding of the project, the results of the subsurface investigation, and geotechnical recommendations for the design and construction of the 16” O.D. pipeline to be installed via Horizontal Directional Drilling (HDD) between the Marcus Hook Compressor Station and the PECO Tilghman Station in Delaware County, Pennsylvania.

It has been a pleasure to be of service to HGA. If you have any questions or need further information, please do not hesitate to contact us at this office.

Very truly yours,

JOHNSON, MIRMIRAN & THOMPSON, INC.

Michael E. Leffler, P.E.
Vice President

40 Wight Avenue, Hunt Valley, MD 21030 (410) 329-3100 www.jmt.com

ADELPHIA GATEWAY, LLC
TILGHMAN LATERAL HDD
DELAWARE COUNTY, PENNSYLVANIA

Table of Contents

1.0 INTRODUCTION, OBJECTIVE AND SCOPE

This report was prepared for the Hunt, Guillot and Associates (HGA) in accordance with our agreement. Johnson, Mirmiran & Thompson, Inc. (JMT) has completed the Geotechnical Engineering Report for the design and construction of the Proposed Adelphia Gateway, LLC 16-inch Tilghman Lateral HDD project consisting of a pipeline to be installed via Horizontal Directional Drilling in Delaware County, Pennsylvania.

1.1 SITE DESCRIPTION AND PROPOSED CONSTRUCTION

The project site is to be located in Lower Chichester Township, Trainer Borough, and Chester City in Delaware County, Pennsylvania. The pipeline is proposed to start at the Marcus Hook Compressor Station on the southwest side of the project, run northeast along West and East Ridge Road, turn to the southeast for approximately 0.45 miles and then proceed northeast again along State Highway 291/West 2nd Street until it terminates at the PECO Tilghman Station adjacent to the railroad on the east side of USR 322. The pipeline will be located in an urban area complete with houses, factories, businesses and even a couple nearby superfund sites. As is to be expected, numerous underground utilities are present in this area. In addition, navigating the permitting concerns of all of the municipalities with jurisdiction over the project may present challenges to the project much as we encountered challenges gaining permits to perform exploratory boreholes along the length of the project.

The proposed project consists of the construction of a 16-inch O.D. pipeline that will be approximately 4.4 miles in length and be located approximately 20-feet below the pavement, railroad, or stream. It is proposed to install the vast majority of the pipe via Horizontal Directional Drilling in order to provide a minimum of disruption to the existing facilities. Nine entry points are proposed, and they are to primarily be located in the existing highway right-of-way. In four locations near the pipe bend points and near the end of the project, trenching will be needed. Specifically, trenched locations will be needed between HDD-4 and HDD-5, HDD-5 and HDD-6, HDD-7 and HDD-8, and HDD-8 and HDD-9.

1.2 SCOPE OF WORK

The scope of JMT's services on this project consists of exploring the subsurface conditions using soil borings and geotechnical laboratory testing, evaluating the subsurface conditions encountered, developing geotechnical recommendations, and submitting our findings in this report.

1.3 PURPOSE OF REPORT

The purpose of this report is to provide the results of the general subsurface investigation along the alignment of the proposed pipeline and geotechnical engineering recommendations to aid in the design and construction of the nine HDD sections of pipeline and the four open trench sections of pipeline. The geotechnical engineering recommendations presented in this report are based on JMT's geotechnical engineering analysis of the subsurface conditions indicated by the test borings.

2.0 SUBSURFACE INVESTIGATION

2.1 GEOLOGIC SETTING

The Marcus Hook Project site is located in Delaware County, in southeast Pennsylvania. The proposed 22,500-foot pipeline begins on Ridge Road, about 200 feet from the Pennsylvania/Delaware state line. The ridge for which the road is named is the surface expression of the Fall Line which is the boundary between the Piedmont to the west and the Atlantic Coastal Plane to the east. Ridge Road parallels the “Northeast Corridor” which is about 1000 feet to the southeast. The Northeast Corridor is the AMTRAK and CSXT electrified rail line that connects the metropolitan areas of Washington D.C. and Boston to Maine. Both Ridge Road and the rail lines parallel the Delaware River which is the major topographic feature in the area. About half of the total length of the pipeline will occupy the right of way of Ridge Road. The pipeline then turns southeast, crosses under the rail lines and turns northeast to parallel the rail lines on their southeast side for about 7,800 feet to its terminus.

The site is within the transition zone between the Atlantic Coastal Plain Physiographic Province, and the Piedmont Physiographic Province. This zone is characterized by the presence of relatively thin coastal plain unconsolidated sediments overlying the igneous and metamorphic rocks of the Piedmont. The top of rock drops sharply towards the southeast.

Topography is low relief and surface drainage is southeasterly towards Naaman Creek, Marcus Hook Creek, and Stoney Creek which all outlet into the Delaware River.

The Coastal Plain sediments on site are mapped as the Quaternary Age Trenton Gravel which is a product of Delaware River deposits. It is described in the literature as gray or pale-reddish-brown very gravelly sand interstratified with cross bedded sand and clay-silt beds. It also includes areas of Holocene alluvium and swamp deposits. The Trenton overlies crystalline rock of lower Paleozoic age which outcrops to the west. Two rock formations are mapped within the project limits. The *Geologic Map of Pennsylvania 1980* shows the portion of the project west of the intersection of Ridge Road and Summit Street to be underlain by anorthosite. The later mapping, shown in the *Bedrock Geologic Map of the Pennsylvania Portion of the Marcus Hook Quadrangle Delaware County, Pennsylvania 2005* describes the rock as the Ardentown Granite Suite. The primary difference in minerology between granite and anorthosite is that the granite would contain more quartz and would be slightly harder to drill. To the east of Summit Street, which is between borings B-3.1and B-3.2, The rock type is mapped as the Wissahickon Formation on the 1980 map and as Chester Park Gneiss on the 2005 map. Wissahickon is an older mapping unit which includes both gneiss and schist. The later mapping which defines the area as gneiss is confirmed by the cores recovered from borings B-5.2 and B-8.2. The hardness and strength of gneiss is generally considered to be equivalent to those of granite. The transition from residual soil to gneissic rock is frequently gradational. The borings show the presence of silty sand material having high STP blow counts which are shown on the logs as 50/<6”. This material is described as decomposed rock and usually becomes denser with depth and will contain gravel size rock fragments.

2.2 SOIL BORINGS AND LABORATORY TESTING

The subsurface investigation consisting of a total of 20 test borings was performed by our sub-consultant American Geotech Incorporated (AGI) in two series, the first one on July 31 through August 21, 2018; and the second one on September 19 through October 2, 2018. The borings were located along the alignment of the proposed HDD-pipe installations. Four of the borings had to be moved from their original prescribed locations in order to avoid either utilities or private property that was not accessible. These borings are B-2.1, B-2.2, B-5.2, and B-9.1. Their new locations are presented on the sheets in the back of Appendix A.

The borings were either drilled through or nearby a roadway surface. The pavement that was cored (see the photo below) was asphalt, except in the cases of borings B-3.3 and B-6.1 where there is 10" of concrete and 3" to 3.5" of asphalt in the pavement buildup. The borings were drilled using a SIMCO HS 2800 truck-mounted rig.

Borings B-8.3, and B-9.1 were not drilled until the electric and natural gas utility subsidiary of Exelon Corporation named PECO first cleared their pipelines by utilizing the vacuum excavation method to locate them. Vacuum Excavation, as pioneered by the SoftDig® Company, consists of advancing a lance into the soil. The soil is loosened by a compressed air jet and simultaneously sucked up the lance by a vacuum. The lance can be advanced with very little down force, and no cutting tools are employed. Underground utilities, including cables, conduits, and pipes are all hard enough to stop the advancement of the lance without damage.

Representative soil samples were obtained in the soil borings by means of the split-barrel sampling procedure (see the photo below) in general accordance with ASTM D 1586. In the split-barrel sampling procedure, a 2-inch O.D. split-barrel sampler is driven into the soil a distance of 24 inches by means of 140-pound hammer falling 30 inches. The number of hammer blows required to drive the sampler through a 12-inch interval is termed the Standard Penetration Test (SPT) N value. It is calculated by adding the blow counts for the second and third 6-inch intervals of each sample which can be found on the Boring Logs. The SPT was performed on the SIMCO HS 2800 rig using a rope and cathead. A representative portion of each SPT sample was placed in a glass jar to preserve its in-situ moisture content, and appropriately marked. Samples were taken at depths of 3-5 feet, and every 5 feet thereafter.

When bedrock was encountered, in some cases the boring was terminated, and in other cases it was sampled through coring (see the photo below). Specifically, rock was cored in borings B-1.1, B-1.3, B-2.1, B-5.2, and B-8.2. The rock was sampled in general accordance with ASTM D2113-08 using an NQ rock core barrel. The percentage of sample recovery was recorded on the boring log along with the visual description. The Rock Quality Designation (RQD) was determined in accordance with ASTM D6032-02, and it was also recorded on the boring log. Borings B-1.2, B-3.1, B-3.2, B-4.1, B-6.1, B-7.1, and B-8.1 were terminated upon encountering apparent bedrock surface; and borings B-2.2, B-3.3, B-4.2, B-5.1, B-6.2, B-6.3, B-8.3, and B-9.1 were terminated before encountering bedrock surface. It should be noted that where the borings were terminated prior to contacting bedrock surface, it was because these borings were drilled at the beginning or end of an HDD section where the pipeline is shallower in depth.

All the samples were transported to AGI's headquarters for further visual examination by an experienced Geotechnical Engineer. Representative soil samples were selected for laboratory testing. The laboratory tests conducted on these samples included tests to classify the soil strata and to determine the engineering parameters that are required to perform analyses. The tests included:

- Natural moisture content in accordance with ASTM D2216
- Particle size analysis in accordance with ASTM D422
- Atterberg Limits tests in accordance with ASTM D4318
- Unconfined Compression test in accordance with ASTM D2166

The Unified Soil Classification System (USCS) was used to analyze the samples. A USCS chart and Logs of the test borings are included in Appendix B, and the laboratory test data is included in Appendix C.

In addition to the split spoon samples, six shelby tube samples were taken in the following borings adjacent to the Sunoco facility in Lower Chichester Township: B-1.1, B-1.2, B-2.1, B-2.2, B-3.1, and B-3.2. These samples were not taken for the usual intended purpose of performing soil strength tests on undisturbed samples, but rather for acquiring larger samples on which to perform resistivity, corrosive, heavy metal, volatile organic compounds, and semi-volatile organic compounds testing if so desired.

Asphalt pavement being cored in boring B-2.1 prior to drilling.

Drive Rod being lowered down through the hollow stem augers in order to obtain a standard penetration test sample in boring B-5.2.

Opening the core barrel to see the rock core sample in boring B-2.1.

3.0 SUBSURFACE CONDITIONS

3.1 SUBSOIL CONDITIONS/STRATIGRAPHY

The subsurface conditions at the site were evaluated by using the data from the 20 soil test borings completed for the proposed pipeline. The boring location plan view indicating the approximate location of the test borings and logs of the test borings are included in Appendices A and B, respectively. The stratification lines shown on the test boring logs represent approximate transitions between material types. In-situ strata changes could occur gradually or at slightly different levels. Also, the boring logs depict conditions at each particular boring location at the time of drilling indicated. Some conditions, particularly groundwater conditions, could vary from the conditions encountered in each particular boring on the day of drilling. Furthermore, this report does not reflect any variations which may occur between the borings. The nature and extent of the variations between borings may not become evident until the time of construction.

The surface materials consisted of grass at the tops of borings B-1.2, B-1.3, B-4.1, B-4.3, B-5.1, B-6.3, and B-7.1; and gravel at the tops of borings B-1.1, B-6.2, B-8.1, B-8.2, B-8.3, and B-9.1. Borings B-2.1, B-2.2, and B-3.1 each went through 8” of asphalt, and boring B-3.2 went through 6.5” of asphalt. Boring B-5.2 went through cracked asphalt in the parking lot. Borings B-3.3 and B-6.1 were drilled through a combination concrete/asphalt pavement buildup as described above in Section 2.2.

Both cohesive and granular strata were encountered in the borings. The strata tended to increase in density or hardness with increase in depth based on the blow counts.

The borings encountered the following set of strata/subsurface conditions:

Stratum A: Cohesive Soils: Three substrata consisting primarily of Lean Clay (CL) were differentiated based on their degree of hardness: one being soft to medium stiff; the second one being stiff to very stiff; and the third being very stiff (this stratum was only encountered in boring B-8.2). Two samples were classified as Fat Clay (CH) in boring B-4.2. In borings B-1.1, B-5.1 and B-5.2, there were a few samples that were classified as either Silty Sand (SM) or Elastic Silt (MH) that we included in the cohesive strata based on their relatively high degrees of plasticity as determined by the plasticity index tests. In the case of boring B-1.1, sample S-3, the soil at this depth had a Liquid Limit of 69.7, and a Plasticity Index of 25.2. Similarly, for boring B-5.1, sample S-2, the Liquid Limit was 58.0 and the Plasticity Index was 26.4; and for boring B-5.2, sample S-2, the Liquid Limit was 58.7 and the Plasticity Index was 23.7.

Six unconfined compression tests were run to more accurately gauge the soil strength. They were run on essentially intact SPT samples from borings B-4.1, B-4.2, B-6.1, b-6.3, B-8.2, and B-9.1. The strengths determined by the testing are as follows:

1. 5 samples were tested from the stiff to very stiff Lean Clay substratum. The average unconfined compressive strength from these samples was 23.5 psi at an average strain of 6.9%.

2. Only one sample, sample S-2 from boring B-8.2, was tested from the very stiff Lean Clay substratum. It had an unconfined compressive strength of 39.2 psi at a strain of 14.3%.

Stratum B: Granular Soils: Three substrata consisted of Clayey Sand or Clayey Sand with gravel (SC) and Silty Sand (SM). These three substrata were differentiated based on their degree of compactness: one being loose; the second one being medium dense to dense; and the third one being very dense. There was also a very loose Silt (ML) substratum which was only encountered in two borings: borings B-2.2 and B-7.1. In addition, there was a loose to medium dense Silt or Sandy Silt, (ML) and Silty Sand (SM) substratum that was encountered in numerous borings.

The very dense Clayey Sand, Clayey Sand with Gravel, and Silty Sand with gravel (SC or SM) substratum was encountered immediately above bedrock in the borings in which bedrock either was encountered or was believed to be encountered, and it may be residual soil that weathered out of bedrock over time.

Three unconfined compression tests were run on samples that had enough cohesive material to be able to be tested. These were again essentially intact SPT samples, and they came from borings B-6.2, B-7.1, and B-8.1. The strengths are as follows:

1. Only one sample, sample S-2 from boring B-7.1, was tested from the very loose Silt substratum. It had an unconfined compressive strength of 7.75 psi at a strain of 13.9%.
2. 2 samples were tested from the loose to medium dense Silt or Sandy Silt, and Silty Sand substratum. The average unconfined compressive strength from these samples was 28.4 psi at an average strain of 8.55%.

Stratum C: Cobbles: Zones of cobbles were encountered in borings B-1.3, B-6.2, and B-8.2, and may be expected to pop up in other places along the pipeline alignment.

Stratum D: Weathered Rock: Weathered rock was encountered in boring B-5.2 in the top two feet above bedrock.

Stratum E: Bedrock: Boring refusal was encountered in 12 of the 20 borings. As noted above in Section 2.2, rock was cored in borings B-1.1, B-1.3, B-2.1, B-5.2, and B-8.2 after augur refusal was encountered. In borings B-1.2, B-3.1, B-3.2, B-4.1, B-6.1, B-7.1, and B-8.1, augur refusal was encountered on what is believed to be bedrock surface, and the borings were terminated at that point.

In borings B-1.1, B-1.3, B-2.1, B-5.2, and B-8.2 in which bedrock was cored, a consistent pattern emerged in which igneous rock consisting of Granitic Pegmatite (Xpg) was encountered near the beginning of the pipeline in borings B-1.1, B-1.3, and B-2.1; and metamorphic Amphibolite from Hornblende Gneiss (hg) was encountered in borings B-5.2 and B-8.2 to the northeast. Bedrock surface appears to be fairly consistent in depth at approximately 30 feet deep with bedrock surface sloping gently in elevation from southwest to northeast. The rock core samples had an average recovery percentage of 93.6, and an average RQD of 66.5% in the Granitic Pegmatite and 53.5% in the Amphibolite from Hornblende Gneiss.

3.2 GROUNDWATER CONDITIONS

Groundwater was measured during drilling. Groundwater was encountered in all the borings except B-1.3, B-2.2, and B-5.2. Typically, groundwater was encountered at approximately 10 feet deep. Only under the alignment for HDD-3 was it found to be significantly different where it was observed to be at 23 feet deep in boring B-3.1, and 28 feet deep in boring B-3.2.

It is anticipated that the HDD installed pipelines will be below groundwater level for the majority of their lengths. In addition, some of the portions of the pipeline that are to be installed in trenches may encounter groundwater during construction as well depending on how deep these portions of pipeline are to be installed and potentially the time of year as well.

The recorded water levels, or absence of water, reflect the conditions at the time of this investigation only. Fluctuations in the location of hydrostatic groundwater level and perched water levels can occur as a result of seasonal variations in evaporation, precipitation, surface water run-off, leaking utilities and other factors.

3.3 LABORATORY TEST PROGRAM

3.3.1 Moisture and Classification Testing

All the samples were visually classified in the laboratory by a Geotechnical Engineer to corroborate and/or modify the field classifications. The classifications were based on texture and plasticity in general accordance with the Unified Soil Classification System (USCS) ASTM D-2488. The USCS group symbol for each soil type is indicated for each stratum in the ASTM Classification column on the boring logs. Selected samples were tested for their natural water content, grain size distribution including percentage fines, and Atterberg Limits. All tests were conducted in accordance with ASTM procedures. Results of the laboratory tests are included in Appendix C.

In all, 42 jar samples which were considered representative of the different soil strata encountered along the proposed pipeline alignment were selected for testing, and all of them were tested for moisture content. 40 of them were tested for grain size distribution including the hydrometer analysis, and 20 of them were tested for plasticity. Based on these results, and by visually classifying the samples above or below the tested samples as to whether they were similar or not to the tested ones, the soil profiles presented in Appendix F were developed.

4.0 GEOTECHNICAL ENGINEERING RECOMMENDATIONS

4.1 ABOVE GROUND OBSTRUCTIONS

It is recommended that the Contractor have the proper equipment on hand to install the pipe in conditions where there are overhead wires running along the side of the road and crossing the road in numerous locations, and where there are buildings and fences that are very close to the side of the road in some locations. This equipment includes both the equipment to be used for the trenched sections of the pipe and the equipment to be used for the HDD sections of the pipe. The Contractor should carefully reconnaissance the pipeline route to make sure their equipment is suitable given all of these above ground obstructions that may affect the available headroom and space to operate. In some cases, it may be necessary to move the HDD entry or exit points to avoid overhead obstructions.

4.2 BELOW GROUND OBSTRUCTIONS

Numerous underground utilities will need to be cleared prior to installing the pipe in this urban area. Especially in the area around the Sunoco facility adjacent to HDD-2 where arrangements will need to be made in order to construct this portion of the pipe. We had a difficult time gaining their approval to drill exploratory borings B-2.1 and B-2.2, and ultimately had to move these borings from their originally prescribed locations. See Appendix I for some helpful information on how to go about gaining Sunoco's approval for the installation of this pipeline. In particular, the problem area starts just to the southwest of Blueball Avenue and proceeds along West Ridge Road up to Hewes Avenue. In this location there are either 8 or 9 Sunoco pipelines crossing West Ridge Road (see the Google Map image in Appendix I). One solution to install the pipeline in this zone would be to utilize the vacuum excavation method (see Section 2.2 above) to locate their pipelines.

Other underground obstructions that may be encountered are either buried historic concrete pads or footings, or cobble zones. A concrete pad was encountered in boring B-3.3 at about 3 feet deep, and it is possible for similar occurrences to crop up anywhere on the project site relatively close to the surface. Cobble zones were encountered in borings B-1.3, B-6.2, and B-8.2. Again, similar such occurrences could crop up anywhere on the project site. Both buried concrete pads as well as cobble zones could prove to be an obstacle for the construction of the trenched sections of the pipe as well as the HDD sections, and the right equipment to deal with such obstacles should be on hand at all times.

4.3 HDD SECTION OF PIPE

Granular soils with large sand contents will be encountered throughout much of the pipeline alignment. Such conditions can lead to difficulty in terms of the equipment getting frozen or stuck in the sand, and in terms of a void forming from running sand that can lead to settlement on the ground surface. Therefore, the Contractor who is selected should have experience operating in sandy and below groundwater conditions; and they should have the right equipment, tooling, and materials to handle/prevent any such difficulties from arising.

The subsurface strata are varied along the pipeline alignment including cohesive strata of varying degrees of hardness from soft to very stiff, and granular strata of varying degrees of compactness from very loose to very dense. (See Section 3.1 above for average unconfined compressive strength values for two of the cohesive strata and two of the granular strata.) At locations where softer materials are encountered immediately above harder materials, caution should be used so that the HDD pipes do not deflect off of the harder materials and cause mis-alignment of the HDD and possibly breaks at the joints in the HDD pipes. The pipeline may also hit potential underground concrete pads and cobble zones, and it will penetrate bedrock in places. In addition, the pipeline will be installed both above and below the groundwater table. Thus, the pipe will be installed in a varying set of subsurface conditions, and the Contractor must be prepared with the proper equipment to deal with all of them. In terms of the distribution of cohesive and granular strata, pipelines HDD-1 and HDD-6 should encounter both types of soil. Pipelines HDD-2, HDD-3, HDD-4, HDD-7, and HDD-8 should encounter primarily granular strata; and pipelines HDD-5 and HDD-9 should encounter primarily cohesive strata.

4.4 HORIZONTAL DIRECTIONAL DRILLING THROUGH BEDROCK

Based on the borings, it is anticipated that the HDD installed pipelines should not penetrate bedrock surface throughout most of the proposed alignment. However, even where the pipe will not be installed in bedrock, a good bit of it will be installed in the very dense stratum immediately above bedrock. Also, it may be the case that the pipe winds up penetrating bedrock in places in-between the boring coverage in which bedrock surface rises above the top of rock line developed from the test boring data (e.g. in the long stretch between borings B-3.1 and B-3.2). Furthermore, It should also be noted that in the borings where bedrock surface was not encountered (borings B-2.2, B-3.3, B-4.2, B-5.1, B-6.2, B-6.3, B-8.3, and B-9.1,) estimations were made as to the elevation of the top of rock in these locations based on the top of rock surface established by interpolating between the nearby borings. On this basis, the top of rock line shown in the soil profiles in

Appendix F was developed. But it is only an estimation, and bedrock may in fact be encountered either higher or lower. In the cases of the profiles for HDD-4 and HDD-5, it is unclear based on the nearby borings where the top of rock line is. However, we project that top of rock rises in elevation towards the end of HDD-4 and the beginning of HDD-5 so as to mimic the borings in which bedrock was encountered. In the cases of the borings in which bedrock was encountered, the very dense stratum of SC or SM was encountered immediately above bedrock and it averaged approximately 6' in thickness. So too, in the cases of boring B-4.2 at the end of HDD-4, and boring B-5.1 at the beginning of HDD-5, this stratum was also encountered in the bottom portions of the borings. Therefore, it is likely that bedrock lies not far below the bottoms of these borings. However, this is only a projection, and therefore we did not show the top of rock line in the second half of the soil profile for HDD-4 and in the first half of the soil profile for HDD-5.

Where Bedrock will be breached, it is expected to be hard in nature and hard to drill. Therefore, the right drill rig/tooling will be needed to drill through this rock.

Locations in which the HDD installed pipe appears to either be in bedrock or very close to bedrock surface include the following:

1. HDD-1: along the base run from the toe of the descent to approximate station 7+00; along the base run from approx. station 15+00 to the toe of the rise; up the bottom half of the rise
2. HDD-2: down the bottom half of the descent; along the base run from the toe of the descent to approximate station 21+00
3. HDD-4: at the toe of the descent; along the base run from approx. station 57+50 to the toe of the rise; up the bottom half of the rise
4. HDD-5: down the lower portion of the descent; along the entire base run of the pipe
5. HDD-6: at the toe of the descent
6. HDD-8: along the base run of the pipe which appears to be between 2 to 4.5 feet above bedrock surface up to approx. station 14+00

4.5 TRENCHED SECTION OF PIPE

Support of excavation consisting of shoring or other actable methods deemed appropriate by the Contractor will be needed for part of the trenched sections where space is limited by the surrounding site features above ground. In some places, it may be feasible to excavate without shoring. However, in the cases of the trench between HDD-7 and HDD-8, and the trench between HDD-8 and HDD-9, the excavation will be in granular soil below the groundwater table and the backslope angle will not be able to be very steep Therefore, the excavation limits could become large depending on the depth of the pipe along these two trenched sections, and shoring may be needed in these cases as well.

As previously noted, the depth to groundwater should be expected to fluctuate and be higher at the time of construction than it was at the time of drilling. Also, due to the interlayered cohesive and granular soils, a perched or trapped groundwater table may be encountered. Therefore, depending on the season, and depending on the depth of the pipe, water may be encountered in the excavation. The highest water levels were recorded in the following borings: B-1.2 at 9' deep, B-2.1 at 7' deep, B-6.2 at 10' deep, B-8.2 at 8' deep, B-8.3 at 8' deep, and B-9.1 at 8' deep. The excavation will need to be dewatered if water is encountered in the base of the excavation.

The trenched section of the pipeline between HDD-5 and HDD-6 is expected to be in soft cohesive soil, and the base of the excavation of the trenched section between HDD-7 and HDD-8 may be in very loose silt soil.

Once these trenches are excavated, should the soil in fact be soft, very loose, or wet in nature, then it is recommended that the trench be excavated to a minimum depth of 12 to 24 inches beneath the pipe invert, and that it be backfilled and properly compacted with granular material. The granular material should meet PennDOT requirements for pipe bedding which normally consists of either natural sand, gravel, or sand and gravel from a borrow pit, or similar material from an aggregate producer or supplier. In fact, the base of the excavation of every trenched section of pipe should be inspected for soft, very loose, or wet soils, and any such occurrence should receive the same treatment of overexcavation and replacement with compacted granular material.

4.6 SEISMIC CLASSIFICATION

Section 1613.5.2 in the 2015 IBC references Chapter 20 of ASCE 7, which presents Soil Site Class Definitions in Table 20.3-1 based on various criteria, which include Average Standard Penetration resistance (N_{bar}), Average Shear Wave Velocity, (V_{bar}), and Average Undrained Shear Strength (Su_{bar}). The table provides correlations for Soil Site Classes "C", "D", and "E" with various ranges of Standard Penetration Tests (N_{bar}), Shear Wave Velocity (V_{bar}), and Undrained Shear Strength (Su_{bar}) to be calculated for the top 100 feet of the subsurface materials at a site in accordance with the procedures described in Chapter 20. In addition, the table presents criteria related to various soil properties for Site Classes "E" and "F". Site Classes "A" and "B" are for bedrock, and they are correlated with ranges of Shear Wave Velocity (V_{bar}).

Table 20.3-1 and the procedures outlined in Chapter 20 of ASCE 7 have been used to evaluate the Soil Site Class for this project site. Based on the test boring results, the average N value, N_{bar}, for the proposed site is greater than 50 bpf. This average N value includes extrapolated data down to a depth of 100 feet. Based on this N_{bar} value, the project site should be classified as Class "C". It should be noted that in the case of this project site, even though the surface soils on average are approximately 37 feet deep, and are underlain by hard, metamorphic and igneous rock, Site Classifications "A" and "B" for rock are not permitted per paragraph 20.1 in which it is stated that if 10 ft or more of soil underlie the bottom of the foundation, then Site Classes A and B shall not be assigned. In this case, there is greater than 10 feet of soil underlying the HDD pipelines throughout much of the total alignment length.

5.0 CLOSING

This report has been prepared to aid in the evaluation of this site and to assist the Design Team with the proposed Adelphia Gateway, 16” Tilghman Lateral HDD project located in Delaware County, Pennsylvania. The report scope is limited to recommendations pertaining to the specific project and the location described. The project description represents our current understanding of the significant aspects of the proposed pipeline installation that require geotechnical consideration.

The analysis and recommendations contained in this report are based upon the data obtained from the test borings performed at the locations indicated on the boring location plan. The nature and extent of the variations between borings may not become evident until the course of construction. If subsurface conditions different from those described are noted during construction, then recommendations in this report must be re-evaluated.

Plans and specifications should be established to account for possible additional costs that may be required for construction of foundations and/or excavations as recommended in this report. Additional costs may be incurred for various reasons, including extra foundation depth, dewatering, etc.