VOLUME I
RESOURCE REPORTS

APPENDIX 6A
HDD BORING REPORT

Report of
Subsurface Exploration and Geotechnical Engineering Services

Proposed Tie-in-Facility, 6493 – Eastern Panhandle Expansion, Fulton County, Pennsylvania

Prepared for

EnSiteUSA
109 Fieldview Drive, P.O. Box 1007
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Prepared by

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February 1, 2017

PSI Project No. 0512713

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psi Information To Build On
Engineering • Consulting • Testing

Geotechnical

February 1, 2017

EnSiteUSA
109 Fieldview Drive
P.O. Box 1007
Versailles, KY 40383

Attention: Jacob Shams, P.E.

Reference: Report for Geotechnical Exploration and Assessment
Proposed Tie-in-Facility
6493 – Eastern Panhandle Expansion
Fulton County, Pennsylvania
PSI Project Number: 0512713

Dear Mr. Shams:

Thank you for choosing Professional Service Industries, Inc. (PSI) as your geotechnical consultant for the proposed Tie-in-Facility in Fulton County, Pennsylvania. This facility is planned as part of the Eastern Panhandle Expansion (Pipeline) Project.

As per your authorization, we have completed a subsurface exploration for this project. The findings of the exploration and our recommendations for the proposed development are discussed in the accompanying report. As requested, one electronic and three original hard copies of the report will be provided to you.

The soil samples obtained during this exploration will be retained in our laboratory for sixty days. Should there be any questions, please do not hesitate to contact our office. PSI would be pleased to continue providing geotechnical services throughout the implementation of the project, and we look forward to working with you and your organization on this and future projects.

Respectfully submitted,
Professional Service Industries, Inc.

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Project Manager

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Chief Engineer

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1 EXECUTIVE SUMMARY

PSI has completed a subsurface exploration and geotechnical evaluation for the proposed Tie-in-Facility project in Fulton County, Pennsylvania. One soil test boring was drilled within the footprint of the proposed facility. Competent residual soils underlain by Partially Weathered Rock was encountered within the boring.

The proposed Tie-in-Facility can be supported on shallow foundations bearing on the underlying competent residual soils, provided the recommendations in this report are followed. Shallow foundations can be proportioned using a net allowable bearing pressure of 2,000 pounds per square foot (psf) bearing on compacted structural fill.

The designer should not rely solely upon the executive summary and must read and evaluate the entire contents of this report, prior to utilizing our engineering recommendations in the preparation of design and construction documents.

2 PROJECT INFORMATION

2.1 PROPOSAL AND PROJECT AUTHORIZATION

This report presents the findings and recommendations related to the geotechnical exploration program performed by Professional Service Industries, Inc. (PSI) for the proposed Tie-in-Facility in Fulton County, Pennsylvania. These services were planned and performed in general accordance with scope and services outlined in PSI Proposal No 0512-187947, dated August 17, 2016.

2.2 PROJECT DESCRIPTION

Initial project information was provided by Mr. Jacob Shams with EnSiteUSA. We also reviewed the RFP document titled, “Facility Geotechnical Investigation, for Eastern Panhandle Expansion, Fulton County, PA” dated July 27, 2016. The project involves the construction of a Tie-in-Facility, which will support the pipes above grade. Based on the drawings provided to us, the pipes will be supported on isolated concrete and these columns will be approximately 2 to 3 feet, above the finished grade. We anticipate very minimal cut and fill grading activities of less than 1 foot.

As of the preparation of this report, no structural loading information was provided. However, we anticipate the maximum load on the column to be less than 50 kips.

If any of the noted information is incorrect or has changed, please inform PSI so that we may review the geotechnical data and amend the recommendations presented in this report, if deemed appropriate.

2.3 PURPOSE AND SCOPE OF WORK

The scope of services for this study included a site reconnaissance of the project area and the assessment of subsurface conditions through field exploration and laboratory testing. The study included an assessment of the site and subsurface conditions relative to the proposed development, engineering studies and the preparation of this report. The subsurface exploration was developed to provide the following:

- Geologic review of the project site.

- Subsurface conditions encountered including pertinent soil properties including water levels and drainage.

- Soil data review and analysis as it relates to the proposed site development.

- Civil site recommendations for site preparation, placement and compaction of fill.

- Structural recommendations to support foundation and construction.
- Comments relating to observed geotechnical conditions such as soft material or groundwater which could impact development.
- Determination of the Seismic Site Class and seismic design parameters per IBC 2009 based on the SPT N-values obtained during field exploration.

The scope of our services did not include an environmental assessment for determining the presence or absence of wetlands, or hazardous or toxic materials in the soil, bedrock, groundwater, or air, on or below or around this site. Any statement in this report or on the boring logs regarding odors, colors, unusual or unexpected items or conditions are strictly for the information of our client.

PSI did not provide nor was it requested to provide any service to investigate or detect the presence of moisture, mold or other biological contaminants in or around any structure, or any service that was designed or intended to prevent or lower the risk of the occurrence of the amplification of the same. Client acknowledges that mold is ubiquitous to the environment with mold amplification occurring when building materials are impacted by moisture. Client further acknowledges that site conditions are outside of PSI's control, and that mold amplification will likely occur, or continue to occur, in the presence of moisture. As such, PSI cannot and shall not be held responsible for the occurrence or recurrence of mold amplification.

2.4 SUBSURFACE EXPLORATION

PSI subcontracted Connelly Drilling Inc. to provide drilling services for the exploration program at the site. PSI in the proposal, recommended for two soil test borings to be drilled for the proposed facility. However, as per the client's request, only one soil test boring designated as GO-10 was drilled to a depth of 20 feet, below the existing grade.

Our drilling subcontractor used an ATV drill rig, equipped with an automatic hammer. Standard Penetration Tests were performed at selected depths within the boring regardless of the drilling methods as detailed in ASTM D1586. The penetration resistance, in conjunction with soil classifications, provides an indication of engineering characteristics of a soil.

Soil samples recovered during the drilling operations were transported to the PSI laboratory in Fairfax, Virginia for visual classification and further evaluation. Groundwater when encountered was noted. Descriptions of the soils encountered during our subsurface exploration are provided in the attached Boring Log. Groundwater conditions, penetration resistances, and other pertinent information are also included in the Boring Log in Appendix C.

The ground surface elevation at the boring location shown on the boring log was estimated from the google earth.

Drilling and soil sampling were conducted in accordance with the procedures generally recognized and accepted as standard methods of exploration of subsurface conditions related to earthwork and foundation engineering projects at this time.

The location of the boring is shown on the Boring Location Plan, in Appendix **B**. The findings of the PSI boring are presented on the Test Boring Log also included in Appendix **C**.

2.5 LABORATORY TESTING

A PSI geotechnical engineer visually-manually classified the soil samples obtained for this geotechnical report in general accordance with the Unified Soil Classification System (USCS) (ASTM D2487 and D2488). Selected samples were tested for natural water content (ASTM D2216). Atterberg limits tests (ASTM D4318), grain size analyses (ASTM D6913).

The laboratory test results are presented in Appendix **D**, as well as shown on the boring log.

3 SITE AND SUBSURFACE CONDITIONS

3.1 SITE LOCATION AND DESCRIPTION

The proposed project site is located on the east side of the intersection of Green Lane Road and Ravenwood Drive in Fulton County, Pennsylvania. Based on the google earth, the existing grade within the limits of the proposed Tie-in-Facility is relatively flat and varies from EL. 580 to EL. 582 feet. The surface cover within the limits of the proposed facility consists of grass. The location of the site is shown on the Boring Location Plan attached as **Appendix B**.

3.2 AREA GEOLOGY

The site is geologically located in the Piedmont Physiographic Province. A study of the area geology from the available literature shows that the site is underlain by McKenzie Formation (Maryland) or Willis Creek Formation (Pennsylvania) of Silurian age. The Formation in general consists of gray, thin-bedded shale, siltstone and argillaceous limestone.

3.3 SUBSURFACE CONDITIONS

The stratification of the soil conditions at the actual soil test boring location is described in this section. The log of the boring is provided in Appendix C.

<u>Surface Cover:</u> At the surface, test boring encountered approximately four inches of topsoil.

<u>Residual Soils:</u> Beneath the surface cover, residual soils, described as sandy silt or silty gravel were encountered to a depth of 5 feet, below the existing grade. The consistency of the sandy silt material can be described as stiff, while the relative density of the silty gravel material can be described as medium dense. Based on the Standard Penetration Testing (SPT), the N-values of these residual soils varied from 14 to 22 blows per foot (bpf), with moisture content values varying from 16 to 19 percent. Sieve analysis and Atterberg Limits were performed within the residual soils at a depth of 2.5 feet, below the existing grade. Based on the above tests, the soil was classified as silty gravel (GM) as per USCS and exhibited a liquid limit value of 39, with plasticity index value as 13. Approximately 19 percent of the test sample passed through the No. 200 Sieve (fines).

<u>Partially Weathered Rock:</u> Beneath the residual soils, Partially Weathered Rock (PWR) was encountered to the boring termination depth. The PWR can be described as very hard, clayey gravel material. The SPT-N values of this stratum varied from 50 blows for 5 inches of penetration to 50 blows for 2 inches of penetration, with natural moisture content values varying from 2 to 9 percent. Atterberg and Sieve analysis was performed within this stratum at a depth 8.5 feet below the existing grade. Based on the test

results, the sample was classified as silty gravel with sand (GM) as per USCS with a liquid limit value of 31 and plasticity index of 12. Approximately 15 percent of the test sample passed through the No. 200 Sieve (fines).

The above subsurface description is of a generalized nature provided to highlight the major soil strata encountered. The boring logs included in the appendices should be reviewed for specific information as to individual test boring locations. The stratification lines shown on the test boring logs represent the conditions only at the actual test boring locations. The stratification lines represent the approximate boundaries between subsurface materials and the actual transition may be gradual.

3.4 GROUNDWATER CONDITIONS

During and upon completion of drilling, no groundwater infiltration was observed in the test boring. Water level, if any, at the test boring location is shown on the respective log provided in Appendix **D**.

The groundwater observations presented in this report and the attached boring log reflect those observed at the time of our field activities. We recommend that the Contractor determine the actual groundwater levels at the time of construction to determine groundwater impact on the proposed construction procedure.

4 GEOTECHNICAL ASSESMENT AND RECOMMENDATIONS

The following recommendations are based on the information available on the proposed construction, the data obtained from the boring, and our experience with soils and subsurface conditions similar to those encountered at this site. Because the borings represent a very small statistical sampling of the subsurface materials, conditions encountered during construction may be substantially different from those encountered in our borings. In these instances, adjustments to the design and construction may be necessary depending on the actual conditions encountered.

As indicated earlier, very minimal cut and fill (less than a foot) is anticipated within the proposed construction limits of the Tie-in-Facility. Based on the review of the test boring, competent residual soils will likely be encountered at the design foundation bearing level of the isolated columns, assumed to be below the frost penetration depth, which is 36 inches below the existing grade. Since weathered rock was shallow, it is possible that very dense weathered rock, intact rock or boulders will be encountered in foundation excavations.

Groundwater was not encountered in the boring. However, groundwater seepage or surface runoff may be encountered within the excavation and in which case, sump pumps can be used for temporary dewatering.

4.1 SEISMIC CONSIDERATIONS

The project site is located within a municipality that employs the International Building Code (IBC), 2009 edition. As part of this code, the design of structures must consider dynamic forces resulting from seismic events. These forces are dependent upon the magnitude of the earthquake event as well as the properties of the soils that underlie the site.

Part of the IBC code procedure to evaluate seismic forces requires the evaluation of the Seismic Site Class, which categorizes the site based upon the characteristics of the subsurface profile within the upper 100 feet of the ground surface.

To define the Seismic Site Class for this project, and in accordance with your requested level of assessment, we have interpreted the results of our soil test borings drilled within the project site per Section 1613.5 of the code. Material properties were estimated below the depth of the borings based upon data available in published geologic reports as well as our experience with subsurface conditions in the general site area.

Based upon our assessment, it is our opinion that the subsurface conditions within the areas of the site planned for building construction are consistent with the characteristics of **Site Class C** as defined in Table 1613.5.2 of the building code.

The associated IBC probabilistic ground motion values for latitude 39.723099° and longitude -78.20648º obtained from the *Java Ground Motion Parameter Calculator – Version 5.1.0* on the USGS Earthquake Hazards Program – Seismic Design for Buildings web page (http://earthquake.usgs.gov/designmaps/us/application.php) are as follows:

Table 1: Seismic Design Parameters*

Period (seconds)	**Mapped MCE Spectral Response Acceleration** (g)**		**Site Coefficients**		**Adjusted MCE Spectral Response Acceleration (g)**		**Design Spectral Response Acceleration (g)**	
0.2	S_S	0.129	F_a	1.2	SM_S	0.155	SD_S	0.103
1.0	S_1	0.053	F_v	1.7	SM_1	0.091	SD_1	0.060

* 2% Probability of exceedance in 50 years.
** At B-C interface (i.e. top of bedrock).
MCE= Maximum Considered Earthquake

The Site Coefficients, F_a and F_v presented in the above table were also obtained from the USGS calculator, but can be interpolated from IBC Tables 1613.5.3(1) and 1613.5.3(2) as a function of the site classification and mapped spectral response acceleration at the short (S_S) and 1 second (S_1) periods.

For Seismic Design Category designations of C, D, E or F, which are contingent on the structure “Occupancy Category”, the Code also requires an assessment of liquefaction, slope stability and surface rupture due to faulting or lateral spreading. Detailed evaluations of these factors were beyond the scope of this study. However, the following table presents a qualitative assessment of these issues considering the site class, the subsurface soil properties, the groundwater elevation and probabilistic ground motions.

Table 2: Seismic Hazards

Hazard	**Relative Risk**	**Comments**
Liquefaction	Low	Site soils are dense, and the seismicity is low.
Slope Stability	Low	The site is relatively level and does not incorporate significant cut or fill slopes
Surface Rupture	Low	The site is not underlain by a mapped Holocene-aged fault

4.2 SITE PREPARATION AND EARTHWORK

We anticipate site preparation and earthwork for the proposed Tie-in-Facility to consist primarily of foundation excavation and backfilling.

- Utilities, if any, encountered within the proposed tower footprint should be removed or relocated. The utility excavations shall be backfilled and compacted as per the fill requirements provided in the subsequent paragraphs.

- All loose or wet soils or any debris encountered at the footing subgrade elevation shall be undercut and replaced with structural fill.

- Material satisfactory for structural fill may include clean soil or bankrun sand and gravel (SW, SP, SM, GW, and GM). CL, ML, GC, and SC material can be used in engineered fills, subject to the following limitations:

Maximum Dry Density (per ASTM D698)	≥ 105 pcf
Liquid Limit	≤ 40
Plasticity Index	≤ 20

  Organic soils and high plasticity clays and silts (CH, MH, OL, OH, PT) should not be used as engineered fill. The fill materials should be free from topsoil and debris, have less than 3 percent organics and should not contain rock fragments having a major dimension greater than 3 inches. The use of the excavated fill soils for controlled structural fill will be subject to approval of the Geotechnical Engineer of Record and moisture adjustments at the time of construction, and the plasticity and maximum dry density requirement specified in this section.

  The onsite existing fill material can be reused as a structural fill provided it meets the above indicated requirements.

- Fill placement should be in loose horizontal lifts no greater than 8 inches thick compacted uniformly with the proper equipment.

- Fill required to support the footings and the slab-on-grade should be compacted to at least 98 percent of the maximum dry density as per ASTM D698 (Standard Proctor) test method. The moisture content of the fill should be within plus or minus two (±2) percentage points of the optimum moisture content.

For proper site preparation, the earthwork should be performed under the observation of and to the satisfaction of the Geotechnical Engineer of Record or his authorized representative.

It will be important to maintain positive site drainage during construction. Stormwater runoff should be diverted around the excavated areas. The site should be graded at all times such that water is not allowed to pond. If any surface soils become wet due to rains, they should be removed or dried prior to further site work operations and/or fill placement.

4.3 FOUNDATION DISCUSSION

Our recommendations for subsurface preparation for foundation support are detailed in the following sections.

4.3.1 GENERAL SHALLOW FOUNDATION RECOMMENDATIONS

The isolated columns for the pipes of Tie-in-Facility can be supported on isolated spread foundations bearing on the underlying competent residual soils. The bottom of the column foundations should be below the frost penetration depth, which is assumed to be 36 inches, below the existing grade.

Spread foundations can be proportioned using a net allowable soil bearing pressure of 2,000 pounds per square foot (psf). Utilizing this allowable bearing pressure, we estimated the total settlement to be less than 1 inch with differential settlement being less than ½ inch over a horizontal distance of 25 feet. Column footings should have minimum widths of 24 inches, regardless of the actual bearing pressure.

Because of possible variations in subsurface conditions and related bearing capacity, all footing excavations and trenches should be observed and approved by the Geotechnical Engineer of Record or his qualified representative. Water and possibly some loose soil may collect in the footing excavations as a result of surface precipitation and near ground surface seepage. Therefore:

- Water, loose soil and soil softened by water should be removed from the bottom of the footing excavations before placing concrete.

- Footing excavations should not be left open for long periods. If the concrete cannot be placed due to inclement weather conditions or any other unforeseen circumstances, the bottom of the footing excavations and trenches should be protected by undercutting 3 inches and placing a 3-inch thick lean-mix concrete (2,000 psi) work mat immediately upon approval and before reinforcing steel is placed.

Where unsuitable bearing conditions are encountered as determined by the PSI Geotechnical Engineer or designated representative, these soils should be undercut and replaced with controlled structural fill. If backfilled up to the design bearing elevation, the over-excavation should extend laterally from all foundation edges a minimum of one half the depth of the undercut. The backfill should consist of the materials described earlier in this section. If the overexcavation is filled with concrete or flowable fill, the widening of the excavation will not be required.
Backfill around and above the footing should satisfy the controlled fill requirements described in Section 4.1 ‘Site Preparation and Earthwork’.

4.4 CONSTRUCTION DEWATERING

During our investigation, no groundwater was encountered in the test boring. As such, groundwater may not be encountered during the foundation excavation. However, additional water may be introduced into excavations due to surface runoff, temporary perched water and local precipitation during construction. Our past experience indicates that the foundation and subgrade bearing soils encountered on-site will soften considerably when exposed to free water. The contractor should keep excavations dry to prevent the softening of these materials. Methods such as sloping, ditching, and berming should be used to control surface water at the site.

Groundwater at this site can be handled by using sump pumps and pits may be utilized to direct and remove the water both during and after construction.

For the purposes of managing water that may enter an excavation, we recommend that collection pits with pumps be used to remove the water from the excavation. The sump pits should be backfilled with open graded stone (AASHTO #57 recommended) and should be surrounded by a properly graded filter medium. The purpose of the filter medium is to prevent clogging of the drainage system by the infiltration of fine-grained soils.

Pumping from the sump pits should be done with care to prevent the loss of soil fines, development of soil boils, or instability of slopes. We must emphasize that dewatering requirements will be dictated by groundwater conditions at the time of construction and may require more aggressive techniques than pumping from a sump pit. The contractor should use a technique or combination of techniques which achieve the desired results under actual field conditions.

5 CONSTRUCTION CONSIDERATIONS

To assess that the in-situ soil conditions or those conditions developed during the construction are as anticipated during the design stage, construction control, continuous observation and testing are recommended as follows:

- Structural fill placement, if any, should be monitored by a qualified soils technician working under the supervision of the geotechnical engineer of record.

- All footing excavations should be carried out under the observation of the geotechnical engineer of record or authorized representative.

5.1 EXCAVATION AND SAFETY

In Federal Register, Volume 54, No. 209 (October 1989), the United States Department of Labor, Occupational Safety and Health Administration (OSHA) amended its "Construction Standards for Excavations, 29 CFR, Part 1926, Subpart P". This document was issued to better allow for the safety of workers entering trenches or excavations. It is mandated by this federal regulation that excavations, whether they be utility trenches, basement excavations or footing excavations, be constructed in accordance with the new OSHA guidelines. It is our understanding that these regulations are being strictly enforced and if they are not closely followed, the owner and the Contractor could be liable for substantial penalties.

The Contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. The Contractor's "responsible person", as defined in 29 CFR Part 1926, should evaluate the soil exposed in the excavations as part of the Contractor's safety procedures. In no case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in all local, state, and federal safety regulations.

We are providing this information solely as a service to our client. PSI does not assume responsibility for construction site safety or the Contractor's or other parties' compliance with local, state, and federal safety or other regulations.

6 RECOMMENDED ADDITIONAL SERVICES

Additional foundation engineering, testing, and consulting services recommended for this project are summarized below:

- Footing Evaluations: It is recommended that footing for this project be evaluated by PSI. The purpose of these evaluations will be to verify that the design soil bearing pressure is available and that subgrade areas are properly prepared.

- Earthwork & Compaction Testing: It is recommended that an experienced engineering technician witness the required filling operations and take sufficient in-place density tests to verify that the specified degree of compaction has been achieved. Soil engineering judgments will be involved and should be made by the geotechnical engineer of record with information provided by the engineering technician.

- Soils Laboratory Testing: Testing to aid in the classification and verification of use of the on-site soils for structural fill and/or embankment material should be performed by PSI. Testing includes, but is not limited to, Atterberg Limits, Grain Size Analysis, California Bearing Ratio, Standard Moisture Density Relationship, and Moisture Content.

7 REPORT LIMITATIONS

The recommendations submitted in this report are based upon the available subsurface information obtained by PSI and design details furnished by **EnSiteUSA** for the proposed project. If there are any revisions to the plans for this project or if deviations from the subsurface conditions noted in this report are encountered during construction, PSI should be notified immediately to determine whether the recommendations provided herein must be changed. If PSI is not retained to perform these functions, we will not be responsible for the impact of those conditions on the geotechnical recommendations for the project.

PSI warrants that the findings, recommendations, specifications, or professional advice contained herein have been made in accordance with generally accepted professional geotechnical engineering practices in the local area at the date of this report. No other warranties are implied or expressed.

No entity can be as familiar with the design concepts inherent in these recommendations as PSI. Accordingly, only observations by PSI can permit PSI to finalize its recommendations and enhance the likelihood of the design concept being adequately considered during implementation of its recommendations.

After the plans and specifications are more complete, PSI should be retained and provided the opportunity to review the final design plans and specifications to check that our engineering recommendations have been properly incorporated into the design documents. At that time, it may be necessary to submit supplementary recommendations. This report has been prepared for the exclusive use of **EnSiteUSA** and its clients for the specific application to construction of the proposed **Tie-in-Facility Project,** located in Fulton County, Pennsylvania.

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APPENDIX A: IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL REPORT

Important Information about Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

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

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

Read the Full Report

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

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

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

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

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

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

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

Subsurface Conditions Can Change

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

Most Geotechnical Findings Are Professional Opinions

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

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual

subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

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

Do Not Redraw the Engineer's Logs

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

Give Contractors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

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

Geoenvironmental Concerns Are Not Covered

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

Obtain Professional Assistance To Deal with Mold

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

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

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

ASFE
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APPENDIX B – VICINITY MAP AND BORING LOCATION PLAN

LEGEND:

● B-1 - PROPOSED BORING

(10 FT) - BORING DEPTH

NOTES:
1. ALL BORINGS WILL BE ADVANCED WITH HOLLOW-STEM AUGERS.
2. SPT SAMPLING WILL BE PERFORMED IN ALL BORINGS.
3. BORING DEPTHS ARE AS SHOWN
4. BORING SPOILS WILL USED TO BACKFILL THE BORE HOLES.

psi Information To Build On
Engineering • Consulting • Testing

		REVISIONS
BORING LOCATION PLAN **PROPOSED TIE-IN FACILITY**		
FULTON COUNTY, PA		FEBRUARY 1, 2017
N.T.	N.T.S.	0512-713

APPENDIX C: BORING LOGS

DATE STARTED: 12/21/16
DATE COMPLETED: 12/21/16
COMPLETION DEPTH 20.0 ft
BENCHMARK: N/A
ELEVATION: 581 ft
LATITUDE: 39.723074°
LONGITUDE: 78.206551°
STATION: N/A **OFFSET:** N/A
REMARKS:

DRILL COMPANY: Connelly Drilling, Inc.
DRILLER: Josh Lewis **LOGGED BY:** Philip Daute
DRILL RIG: CME 550 ATV
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: 2-in SS1.874-in Core Standard
HAMMER TYPE: Automatic
EFFICIENCY N/A
REVIEWED BY: Lubomir Peytchev

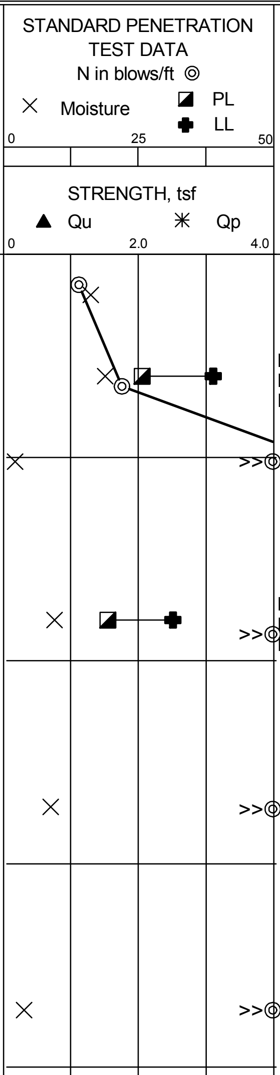
BORING GO-10

Water

▽	While Drilling	Dry feet
▼	Upon Completion	Dry feet
▼	Delay	N/A feet

BORING LOCATION:

Elevation (feet)	Depth, (feet)	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS) RQD & Recovery % (NX)	Moisture, %	Additional Remarks
	0			Approximately 4 inches of Topsoil	Top Soil			
580		1	18	Stiff, moist, light brown **sandy SILT (USCS ML)** some gravel, roots.	ML	3-6-8 N=14	16	
		2	14	Medium dense, moist, light brown **silty GRAVEL (USCS GM)** with sand, trace shale fragments. **Residuum**	GM	3-8-14 N=22	19	LL = 39 PL = 26 Fines=19.3%
575	5	3	2	Weathered Rock, SHALE and LIMESTONE, sampled as very hard, moist, red, dark brown **clayey GRAVEL (USCS GC) with sand**, Silurian [Bloomsburg and Mufflintown Formation]	WEATHERED SHALE	50/2"	2	
	10	4	10			7-50/5"	9	LL = 31 PL = 19 Fines=15.7%
570		5	3			50/3"	9	
565	15	6	2			50/2"	4	
	20			Bottom of test boring at 20 feet				



Professional Service Industries, Inc.
2930 Eskridge Rd
Fairfax, VA 22031
Telephone: (703) 698-9300

PROJECT NO.: 0512713-1
PROJECT: 6493-Eastern Panhandle Expansion
LOCATION: Potomac River Crossing
Washington County
Hancock, MD

The stratification lines represent approximate boundaries. The transition may be gradual.

GENERAL NOTES

SAMPLE IDENTIFICATION

The Unified Soil Classification System (USCS), AASHTO 1988 and ASTM designations D2487 and D-2488 are used to identify the encountered materials unless otherwise noted. Coarse-grained soils are defined as having more than 50% of their dry weight retained on a #200 sieve (0.075mm); they are described as: boulders, cobbles, gravel or sand. Fine-grained soils have less than 50% of their dry weight retained on a #200 sieve; they are defined as silts or clay depending on their Atterberg Limit attributes. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size.

DRILLING AND SAMPLING SYMBOLS

SFA: Solid Flight Auger - typically 4" diameter flights, except where noted.
HSA: Hollow Stem Auger - typically 3¼" or 4¼ I.D. openings, except where noted.
M.R.: Mud Rotary - Uses a rotary head with Bentonite or Polymer Slurry
R.C.: Diamond Bit Core Sampler
H.A.: Hand Auger
P.A.: Power Auger - Handheld motorized auger

☒ SS: Split-Spoon - 1 3/8" I.D., 2" O.D., except where noted.
■ ST: Shelby Tube - 3" O.D., except where noted.
▯ RC: Rock Core
⬇ TC: Texas Cone
✋ BS: Bulk Sample
▦ PM: Pressuremeter
CPT-U: Cone Penetrometer Testing with Pore-Pressure Readings

SOIL PROPERTY SYMBOLS

N: Standard "N" penetration: Blows per foot of a 140 pound hammer falling 30 inches on a 2-inch O.D. Split-Spoon.
N_{60}: A "N" penetration value corrected to an equivalent 60% hammer energy transfer efficiency (ETR)
Q_u: Unconfined compressive strength, TSF
Q_p: Pocket penetrometer value, unconfined compressive strength, TSF
w%: Moisture/water content, %
LL: Liquid Limit, %
PL: Plastic Limit, %
PI: Plasticity Index = (LL-PL),%
DD: Dry unit weight, pcf
▼,▽,▼: Apparent groundwater level at time noted

RELATIVE DENSITY OF COARSE-GRAINED SOILS

Relative Density	N - Blows/foot
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	50 - 80
Extremely Dense	80+

ANGULARITY OF COARSE-GRAINED PARTICLES

Description	Criteria
Angular:	Particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular:	Particles are similar to angular description, but have rounded edges
Subrounded:	Particles have nearly plane sides, but have well-rounded corners and edges
Rounded:	Particles have smoothly curved sides and no edges

GRAIN-SIZE TERMINOLOGY

Component	Size Range
Boulders:	Over 300 mm (>12 in.)
Cobbles:	75 mm to 300 mm (3 in. to 12 in.)
Coarse-Grained Gravel:	19 mm to 75 mm (¾ in. to 3 in.)
Fine-Grained Gravel:	4.75 mm to 19 mm (No.4 to ¾ in.)
Coarse-Grained Sand:	2 mm to 4.75 mm (No.10 to No.4)
Medium-Grained Sand:	0.42 mm to 2 mm (No.40 to No.10)
Fine-Grained Sand:	0.075 mm to 0.42 mm (No. 200 to No.40)
Silt:	0.005 mm to 0.075 mm
Clay:	<0.005 mm

PARTICLE SHAPE

Description	Criteria
Flat:	Particles with width/thickness ratio > 3
Elongated:	Particles with length/width ratio > 3
Flat & Elongated:	Particles meet criteria for both flat and elongated

RELATIVE PROPORTIONS OF FINES

Descriptive Term	% Dry Weight
Trace:	< 5%
With:	5% to 12%
Modifier:	>12%

GENERAL NOTES

(Continued)

CONSISTENCY OF FINE-GRAINED SOILS

Q_U - TSF	N - Blows/foot	Consistency
0 - 0.25	0 - 2	Very Soft
0.25 - 0.50	2 - 4	Soft
0.50 - 1.00	4 - 8	Firm (Medium Stiff)
1.00 - 2.00	8 - 15	Stiff
2.00 - 4.00	15 - 30	Very Stiff
4.00 - 8.00	30 - 50	Hard
8.00+	50+	Very Hard

MOISTURE CONDITION DESCRIPTION

Description	Criteria
Dry:	Absence of moisture, dusty, dry to the touch
Moist:	Damp but no visible water
Wet:	Visible free water, usually soil is below water table

RELATIVE PROPORTIONS OF SAND AND GRAVEL

Descriptive Term	% Dry Weight
Trace:	< 15%
With:	15% to 30%
Modifier:	>30%

STRUCTURE DESCRIPTION

Description	Criteria
Stratified:	Alternating layers of varying material or color with layers at least ¼-inch (6 mm) thick
Laminated:	Alternating layers of varying material or color with layers less than ¼-inch (6 mm) thick
Fissured:	Breaks along definite planes of fracture with little resistance to fracturing
Slickensided:	Fracture planes appear polished or glossy, sometimes striated

Description	Criteria
Blocky:	Cohesive soil that can be broken down into small angular lumps which resist further breakdown
Lensed:	Inclusion of small pockets of different soils
Layer:	Inclusion greater than 3 inches thick (75 mm)
Seam:	Inclusion 1/8-inch to 3 inches (3 to 75 mm) thick extending through the sample
Parting:	Inclusion less than 1/8-inch (3 mm) thick

SCALE OF RELATIVE ROCK HARDNESS

Q_U - TSF	Consistency
2.5 - 10	Extremely Soft
10 - 50	Very Soft
50 - 250	Soft
250 - 525	Medium Hard
525 - 1,050	Moderately Hard
1,050 - 2,600	Hard
>2,600	Very Hard

ROCK BEDDING THICKNESSES

Description	Criteria
Very Thick Bedded	Greater than 3-foot (>1.0 m)
Thick Bedded	1-foot to 3-foot (0.3 m to 1.0 m)
Medium Bedded	4-inch to 1-foot (0.1 m to 0.3 m)
Thin Bedded	1¼-inch to 4-inch (30 mm to 100 mm)
Very Thin Bedded	½-inch to 1¼-inch (10 mm to 30 mm)
Thickly Laminated	1/8-inch to ½-inch (3 mm to 10 mm)
Thinly Laminated	1/8-inch or less "paper thin" (<3 mm)

ROCK VOIDS

Voids	Void Diameter
Pit	<6 mm (<0.25 in)
Vug	6 mm to 50 mm (0.25 in to 2 in)
Cavity	50 mm to 600 mm (2 in to 24 in)
Cave	>600 mm (>24 in)

GRAIN-SIZED TERMINOLOGY

(Typically Sedimentary Rock)

Component	Size Range
Very Coarse Grained	>4.76 mm
Coarse Grained	2.0 mm - 4.76 mm
Medium Grained	0.42 mm - 2.0 mm
Fine Grained	0.075 mm - 0.42 mm
Very Fine Grained	<0.075 mm

ROCK QUALITY DESCRIPTION

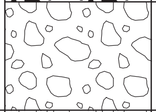
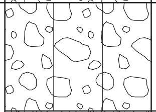
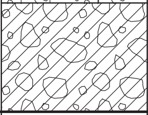
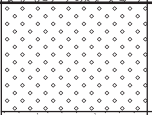
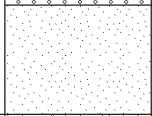
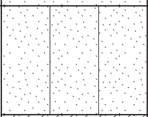
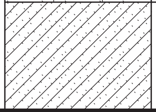
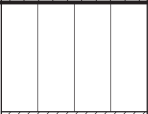
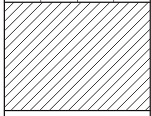
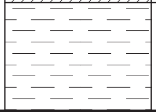
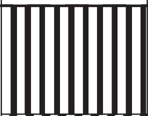
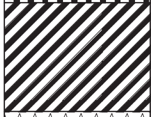
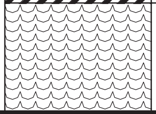
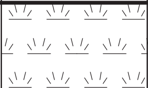
Rock Mass Description	RQD Value
Excellent	90 -100
Good	75 - 90
Fair	50 - 75
Poor	25 -50
Very Poor	Less than 25

DEGREE OF WEATHERING

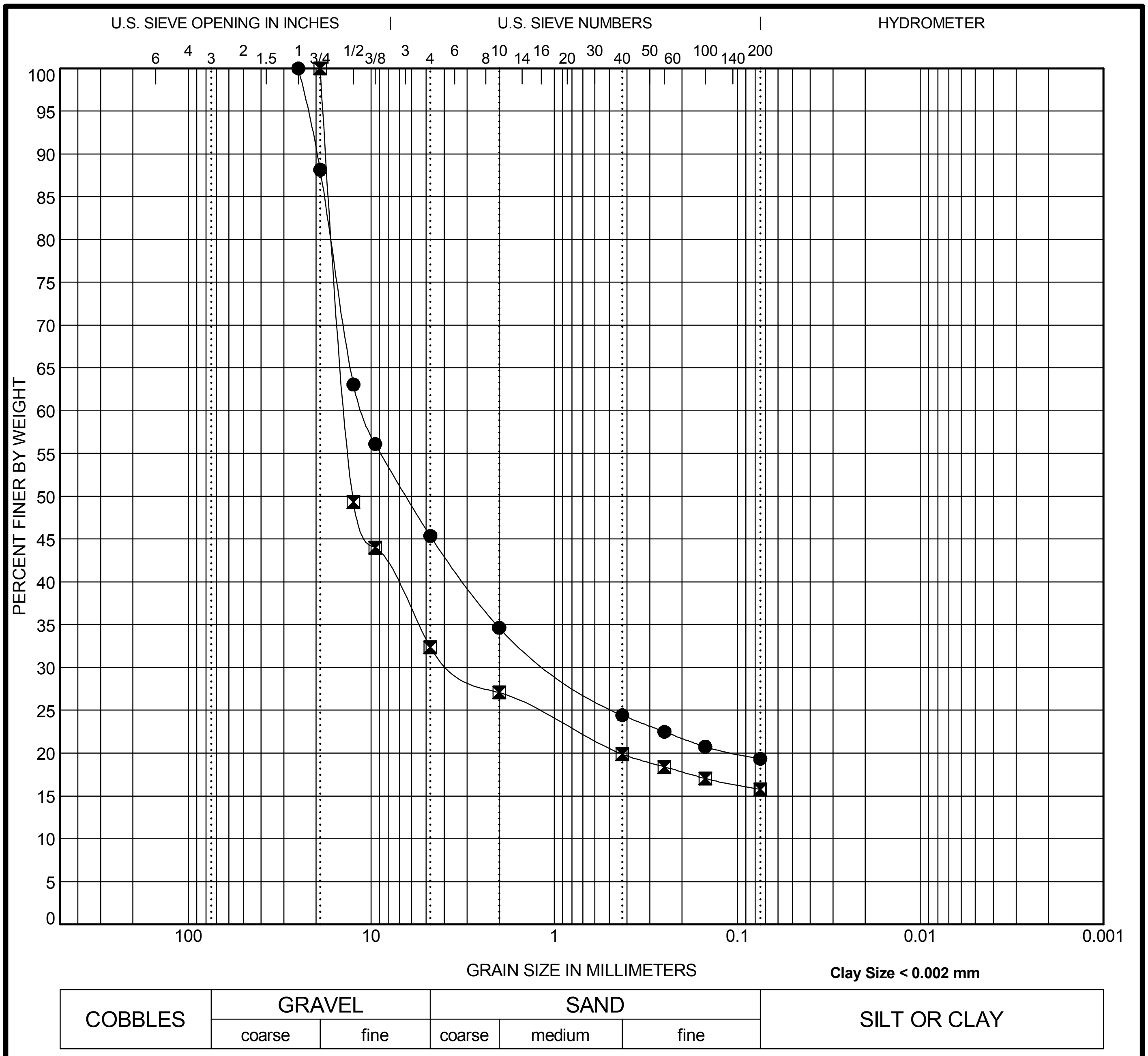
Slightly Weathered:	Rock generally fresh, joints stained and discoloration extends into rock up to 25 mm (1 in), open joints may contain clay, core rings under hammer impact.
Weathered:	Rock mass is decomposed 50% or less, significant portions of the rock show discoloration and weathering effects, cores cannot be broken by hand or scraped by knife.
Highly Weathered:	Rock mass is more than 50% decomposed, complete discoloration of rock fabric, core may be extremely broken and gives clunk sound when struck by hammer, may be shaved with a knife.

SOIL CLASSIFICATION CHART

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		(LITTLE OR NO FINES)		GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
COARSE GRAINED SOILS	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)		GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS	CLEAN SANDS		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE		(LITTLE OR NO FINES)		SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	SANDS WITH FINES		SM	SILTY SANDS, SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)		SC	CLAYEY SANDS, SAND - CLAY MIXTURES
				ML	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS OR CLAYEY SILTS WITH SLIGHT PLASTICITY
FINE GRAINED SOILS	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
				OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE				MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

APPENDIX D: LABORATORY TESTING RESULTS



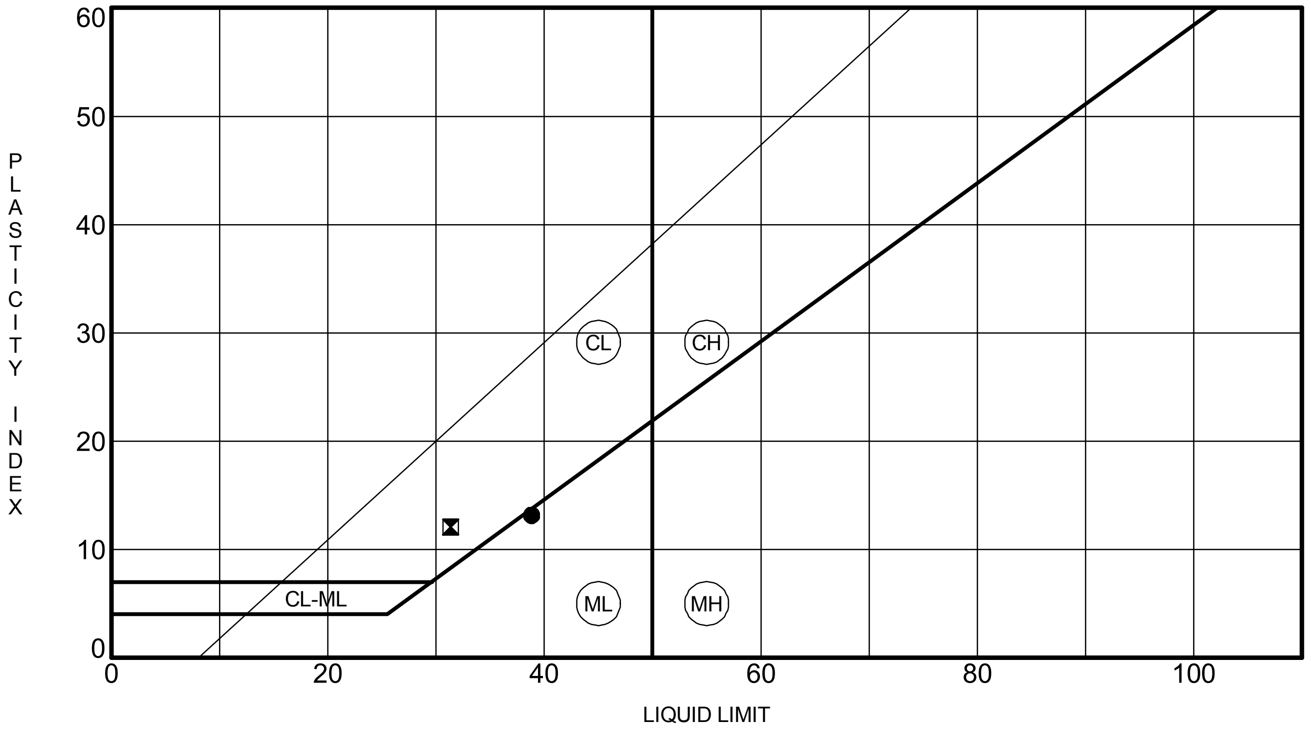
Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	**GO-10**	**3.0**	silty GRAVEL with sand	**39**	**26**	**13**		
☒	**GO-10**	**9.0**	clayey GRAVEL with sand	**31**	**19**	**12**		

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**GO-10**	**3.0**	**25**	**11.076**	**0.99**		**54.6**	**26.0**	**19.3**	
☒	**GO-10**	**9.0**	**19**	**13.653**	**3.225**		**67.6**	**16.6**	**15.7**	

Professional Service Industries, Inc.
2930 Eskridge Rd
Fairfax, VA 22031
Telephone: (703) 698-9300
Fax: (703) 560-7931

GRAIN SIZE DISTRIBUTION

Project: 6493-Eastern Panhandle Expansion
PSI Job No.: 0512713-1
Location: Potomac River Crossing
Washington County



	Boring	Depth (ft)	LL	PL	PI	Fines	Classification (*Visual)
●	**GO-10**	**3.0**	**39**	**26**	**13**	**19.3**	silty GRAVEL with sand
☒	**GO-10**	**9.0**	**31**	**19**	**12**	**15.7**	clayey GRAVEL with sand

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Fax: (703) 560-7931

ATTERBERG LIMIT RESULTS

PSI Job No.: 0512713-1
Project: 6493-Eastern Panhandle Expansion
Location: Potomac River Crossing
Washington County

psi Information To Build On
Engineering • Consulting • Testing

July 12, 2016
Revised August 15, 2016

EnSite USA, Inc.
109 Fieldview Drive
Versailles, KY, 40383
Attn: Grace Northcutt, P.E.

Re: Draft Report for Geotechnical Subsurface Exploration & Engineering Services
6493 – Eastern Panhandle Expansion
I-68 Crossing, Preliminary Investigation
Washington County, Maryland
PSI Project Number 0512719-1

Dear Ms. Northcutt:

Thank you for choosing Professional Service Industries, Inc. (PSI) as your consultant for the referenced project. Authorization to perform services was provided through PSI Proposal No. 0512-182516 dated June 21, 2016. The proposal was executed by Ms. Northcutt, P.E. representing EnSite USA, Inc.

This letter report presents the results of borings performed by PSI at five locations along the proposed HDD alignment at I-68 Crossing. Approximate boring locations are presented in the Appendix, Sections 1A: Site Plan and 1B: Boring Location Plans.

Scope of Services

PSI's services consisted of field exploration, laboratory testing, and preparation of a geotechnical engineering report for the proposed HDD location. Field work included drilling 2 test borings (Borings GO-4, and GO-5) utilizing hollow-stem (HSA) auger drilling, wash rotary drilling, and rock coring in conformance with ASTM standards.

Laboratory testing included unit weight, moisture content, Atterberg limits, grain size distribution tests, pH and resistivity testing, unconfined compressive strength and slake durability testing. All tests were performed in with ASTM standards.

Summary of Field Exploration and Laboratory Testing

The borings were completed with a track-mounted drill rig with hollow-stem augers in conformance with ASTM standards. Standard Penetration Testing (SPT) and split-spoon sampling of overburden soils was performed at 2.5 foot intervals for the first 10 feet and at 5-foot intervals thereafter to the termination depths to evaluate the strength and relative consistency of the soils encountered. Below auger refusal depth, rock coring was performed using NQ coring equipment. All recovered soil and rock samples were visually classified by a PSI geotechnical engineer and a graphical log developed for each boring. Boring depths and depths at which auger refusal were encountered are summarized in Table 1 below.

Table 1 – Summary of Boring Depths

Boring	Approximate Termination Depth (feet)	Ground Surface Elevation (feet, NAVD)	Approximate Depth/Elevation of Top of Weathered Rock	Approximate Depth/Elevation of Auger Refusal
GO-4	60	435	7 feet, EL ±428 MSL	10 feet, EL ±425 MSL
GO-5	70	447	8.5 feet, EL ±438.5 MSL	13.5 feet, EL ±433.5 MSL

The Boring Logs included in the Appendix approximate depths and visual descriptions of overburden soil and underlying rock materials encountered, soil SPT test results, rock core recovery and quality designation (RQD) values, and measurements of groundwater where encountered. The total length of recovered rock core, divided by the length of the run, is referred to as rock core recovery, and is expressed as a percentage. The Rock Quality Designation (RQD) is a measure of the rock mass quality, and is defined as the total length of sound, intact rock core pieces 4 inches or more in length, divided by the length of the rock core run, also expressed as a percentage. The rock core recovery and RQD values are indicated on the Boring Logs included with this report.

Geotechnical Investigation Results

A brief summary of subsurface stratigraphies as encountered at the various borings are presented as follows:

Surficial Materials: Approximately 3 inches of surficial topsoil were encountered at the ground surface of Borings GO-4 and GO-5.

Alluvium with shallow tilled surface (10 to 12 inches) with thickness from 2 feet to 5 feet consisting of soft to medium stiff lean clay (Unified Classification CL) and loose to dense clayey sand (SC)

Residuum: Residual soil classified as clayey SAND (SC) was encountered to depths ranging from approximately 7 to 8.5 feet below existing surface grades at both borings. The residual soil was between 3.5 to 5 feet thick at the boring locations. Standard Penetration Test (SPT) N-values in this layer ranged from approximately 17 to 34 blows per foot (BPF).

Weathered Rock: Typically consisting of weathered shale, weathered rock was encountered at both boring locations. The weathered rock samples consisted of soft shale with Limestone floaters. SPT N-values were typically in excess of 50 blows per foot. Auger refusal was encountered within the weathered rock at depths ranging from approximately 7 to 8.5 feet below existing grades.

Bedrock: Bedrock materials encountered below the auger refusal depths consisted primarily of cyclic sequences of Limestone and shale, with occasional layers of sandy shale and sandstone. Voids were not encountered in the borings. Core recoveries ranged from 33 to 100 percent. RQD values ranged from 0 to 97 percent.

The above subsurface descriptions are of a generalized nature provided to highlight the major strata encountered. The boring logs included in the Appendix should be reviewed for specific information as to individual boring locations. The stratification lines shown on the boring logs represent the conditions only at the actual boring locations. The stratification lines represent the approximate boundaries between subsurface materials and the actual transition may be gradual.

Table 2 – Classification and Strength of Overburden Soil Test Results

Boring	Sample No.	Sample Depth (feet)	USCS Classification [1]	Moisture Content (%)	Atterberg Limits			Grain-Size Distribution		
					Liquid Limit	Plastic Limit	Plasticity Index	Gravel (%)	Sand (%)	Fines (%)
GO-4	S-1	0.0 – 1.5	CL	19	35	21	14	0.6	21.5	77.9
GO-4	S-2	2.5 – 4.0	---	9	---			---		
GO-4	S-3	5.0 – 6.5	SC	9	29	17	12	28.1	51.8	20.1
GO-4	S-4	7.0 – 7.3	---	2	---			---		
GO-5	S-1	0.0 – 1.5	SC	22	45	26	19	12.3	58.3	29.4
GO-5	S-2	2.5 – 4.0	---	41	---			---		
GO-5	S-3	5.0 – 6.5	SC	21	41	25	16	14.8	61.2	23.9
GO-5	S-4	8.5 – 8.9	---	4	---			---		
GO-5	S-5	9.5 – 9.8	---	5	---			---		
GO-5	S-6	13.5 – 13.8	---	3	---			---		

(1) For USCS Soil Classification definitions, refer to the General Notes in Attachment

Table 3 – Elevation, Rock Recovery and RQD Test Results

Boring	Depth (feet)	Elevation (feet)	Run Length (feet)	Recovery (%)	RQD (%)
GO-4	7.0 – 10.0	428 - 425	3	37	0
GO-4	10.0 – 15.0	425 - 420	5	83	10
GO-4	15.0 – 20.0	420 – 415	5	100	66
GO-4	20.0 – 25.0	415 – 410	5	100	80
GO-4	25.0 – 30.0	410 – 405	5	100	80
GO-4	30.0 – 35.0	405 – 400	5	100	75
GO-4	35.0 – 40.0	400 – 395	5	97	66
GO-4	40.0 – 45.0	395 – 390	5	100	80
GO-4	45.0 – 50.0	390 – 385	5	83	0

Boring	Depth (feet)	Elevation (feet)	Run Length (feet)	Recovery (%)	RQD (%)
GO-4	50.0 – 55.0	385 - 380	5	100	86
GO-4	55.0 – 60.0	380 – 375	5	98	86
GO-5	13.5 – 15.0	433.5 – 432.0	1.5	100	61
GO-5	15.0 – 20.0	432 - 427	5	100	80
GO-5	20.0 – 25.0	427 – 422	5	100	68
GO-5	25.0 – 30.0	422 – 417	5	100	95
GO-5	30.0 – 35.0	417 - 412	5	100	57
GO-5	35.0 – 40.0	412 - 407	5	100	93
GO-5	40.0 – 45.0	407 - 402	5	100	83
GO-5	45.0 – 50.0	402 - 397	5	100	97
GO-5	50.0 – 55.0	397 - 392	5	100	67
GO-5	55.0 – 60.0	392 - 387	5	100	90
GO-5	60.0 – 65.0	387 - 382	5	100	93
GO-5	65.0 – 70.0	382 - 377	5	100	93

Table 4 – Rock Unconfined Compressive Strength Test Results

Boring	Approximate Sample Depth (feet)	Rock Classification	Unit Weight (pcf)	Unconfined Compressive Strength	
				(psi)	(tsf)
GO-4	36.0-36.5	Shale	173.96	2140	154.1
GO-4	57.0-57.5	Shale	170.49	2100	151.2
GO-5	21.5-22.0	Shale	170.61	4680	337.0
GO-5	45.5-46.0	Shale	169.67	1500	108.0
GO-5	55.6-56.0	Shale	163.74	3230	232.6

Table 5 – Slake Durability Test Results

Boring	Approximate Sample Depth (feet)	Rock Classification	Slake Durability Index First Cycle (%)	Slake Durability Index Second Cycle (%)
GO-4	38.5 – 39.0	Shale	99.4	98.4
GO-4	54.0 – 54.5	Shale	99.5	98.9
GO-5	22.0 – 22.6	Shale	99.3	98.8
GO-5	44.0 – 44.5	Shale	99.7	99.5
GO-5	64.5 – 65.0	Shale	99.5	99.0

One (1) representative soil sample was selected by PSI for soil resistivity testing. Table 6 below presents a summary of the test results. A detailed report is included in the Appendix.

Table 6 - Soil Resistivity Test Results

Boring No.	GO-4
Depth	1.0’ – 5.0’
pH - AASHTO T289	5.8
Soil Resistivity – AASHTO T-288	8950 Ohm-cm

Should there be any questions, please do not hesitate to contact our office at (703) 698-9300. PSI would be pleased to continue providing geotechnical services throughout the implementation of the project, and we look forward to working with you on this and future projects.

Respectfully submitted,
PROFESSIONAL SERVICE INDUSTRIES, INC.

Lubomir D. Peytchev

Lubomir D. Peytchev, P.E.
Senior Geotechnical Engineer

Naseer Nayeem, P.E.
Vice President/Principal Consultant

Appendix:

Figure 1A Site Vicinity Map and Figure 1B – Boring Location Plan
Boring Logs and General Notes
Cross Section Showing the General Stratigraphy
Laboratory Test Results
Slake Durability Test Results
Soil Resistivity Test Results
Important Information About Your Geotechnical Report

Figure 1A Site Vicinity Map and Figure 1B – Boring Location Plan

Map Source: Google Maps

Information To Build On
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FIGURE 1A: Site Vicinity Map

Columbia HDD I-68 Crossing

Washington County, MD		June 29, 2016
G.C.	Not Drawn To Scale	0512-719

LEGEND:

⌖ B-1 - PROPOSED BORING

(10 FT) - BORING DEPTH

NOTES:

1. ALL BORINGS WILL BE ADVANCED WITH HOLLOW-STEM AUGERS.
2. SPT SAMPLING WILL BE PERFORMED IN ALL BORINGS.
3. BORING DEPTHS ARE AS SHOWN
4. BORING SPOILS WILL USED TO BACKFILL THE BORE HOLES.

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FIGURE 1B: BORING LOCATION PLAN
ENSITE USA- CPG PIPELINE

WASHINGTON COUNTY, MD
JUNE 29, 2016
G.C.
N.T.S.
0512-713

Boring Logs and General Notes

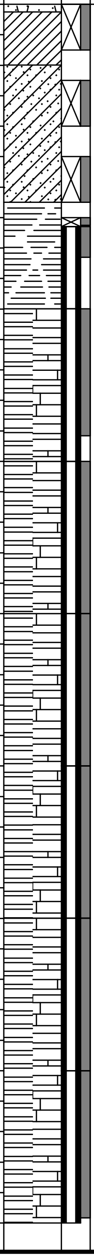
DATE STARTED: 6/23/16
DATE COMPLETED: 6/23/16
COMPLETION DEPTH 60.0 ft
BENCHMARK: N/A
ELEVATION: 435 ft
LATITUDE: 39.7099278°
LONGITUDE: 78.2084611°
STATION: N/A **OFFSET:** N/A
REMARKS:

DRILL COMPANY: Connelly Drilling, Inc.
DRILLER: Tom Chew **LOGGED BY:** J. Thonnfend
DRILL RIG: Diedrich D-50
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: 2-in SS1.874-in Core Standard
HAMMER TYPE: Automatic
EFFICIENCY N/A
REVIEWED BY: Lubomir Peytchev

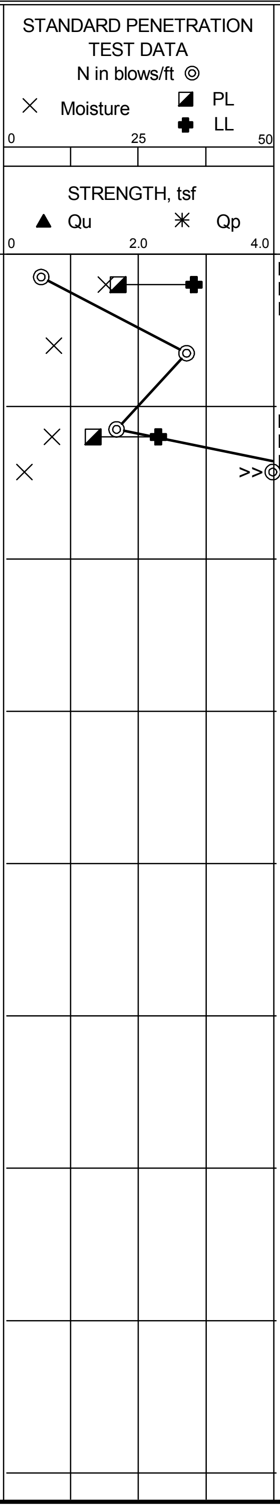
BORING GO-4

Water		
▽ While Drilling	Dry	feet
▼ Upon Completion	Dry	feet
▽ Delay	N/A	feet

BORING LOCATION:



Elevation (feet)	Depth, (feet)	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS) RQD & Recovery % (NX)	Moisture, %	Additional Remarks
	0			Approximately 3 inches of Topsoil	Top Soil			
		1	18	Soft to medium stiff, moist, brown, sandy **lean CLAY (USCS CL)** some gravel, roots. **Alluvium**	CL	1-3-4 N=7	19	LL = 35 PL = 21 Fines=77.9%
		2	18	Dense to medium dense, moist, brown, olive brown, dark brown **clayey SAND (USCS SC)** some gravel, trace shale fragments. **Residuum**	SC	11-18-16 N=34	9	
430	5	3	18			4-10-11 N=21	9	LL = 29 PL = 17 Fines=20.1%
		4	3	Weathered Rock, gray, dark gray, soft SHALE and hard LIMESTONE, Silurian [Wills Creek Shale and Bloomsburg Formation]	WEATHERED SHALE	50/3"	4	
		5	12			RQD=0 Rec=37%		
425	10							
		6	50	Interbedded, slightly weathered, medium bedded to thin bedded, gray, dark gray , fine grained to medium grained, soft SHALE and hard LIMESTONE, dip of 35 degrees, (RQD = 10 %) Silurian [Wills Creek Shale and Bloomsburg Formation]	Shale and Limestone	RQD=10 Rec=83%		
420	15							
		7	60	Interbedded, slightly weathered, medium bedded to thin bedded, gray, dark gray , fine grained to medium grained, soft SHALE and hard LIMESTONE, dip of 50 degrees, (RQD = 66 %) Silurian [Wills Creek Shale and Bloomsburg Formation]	Shale and Limestone	RQD=66 Rec=100%		
415	20							
		8	60	Interbedded, slightly weathered, medium bedded to thin bedded, gray, dark gray , fine grained to medium grained, soft SHALE and hard LIMESTONE, dip of 50 degrees, (RQD = 80 %) Silurian [Wills Creek Shale and Bloomsburg Formation]	Shale and Limestone	RQD=80 Rec=100%		
410	25							
		9	60			RQD=80 Rec=100%		
405	30							
		10	60	Interbedded, slightly weathered, medium bedded to thin bedded, red, trace white and gray , fine grained to medium grained, hard LIMESTONE and soft SHALE, dip of 40 degrees, (RQD of 75 % and 66 %) Silurian [Wills Creek Shale and Bloomsburg Formation]	Shale and Limestone	RQD=75 Rec=100%		
400	35							
		11	58			RQD=66 Rec=97%		
395	40							



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Professional Service Industries, Inc.
2930 Eskridge Rd
Fairfax, VA 22031
Telephone: (703) 698-9300

PROJECT NO.: 0512719-1
PROJECT: 6493-Eastern Panhandle Expansion
LOCATION: I-68 HDD Crossing
Washington County
Hancock, MD

The stratification lines represent approximate boundaries. The transition may be gradual.

BORING GO-4

DATE STARTED: 6/23/16
DATE COMPLETED: 6/23/16
COMPLETION DEPTH 60.0 ft
BENCHMARK: N/A
ELEVATION: 435 ft
LATITUDE: 39.7099278°
LONGITUDE: 78.2084611°
STATION: N/A **OFFSET:** N/A
REMARKS:

DRILL COMPANY: Connelly Drilling, Inc.
DRILLER: Tom Chew **LOGGED BY:** J. Thonnfend
DRILL RIG: Diedrich D-50
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: 2-in SS1.874-in Core Standard
HAMMER TYPE: Automatic
EFFICIENCY N/A
REVIEWED BY: Lubomir Peytchev

Water		
▽	While Drilling	Dry feet
▼	Upon Completion	Dry feet
▼	Delay	N/A feet

BORING LOCATION:

Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS) RQD & Recovery % (NX)	Moisture, %	STANDARD PENETRATION TEST DATA N in blows/ft ◎ × Moisture ◪ PL ✚ LL 0 25 50 STRENGTH, tsf ▲ Qu ✳ Qp 0 2.0 4.0	Additional Remarks
	40			12	60	Interbedded, slightly weathered, medium bedded to thin bedded, red, trace white and gray , fine grained to medium grained, hard LIMESTONE and soft SHALE, dip of 40 degrees, (RQD = 80 %) Silurian [Wills Creek Shale and Bloomsburg Formation]	Shale and Limestone	RQD=80 Rec=100%			
390	45			13	50	Interbedded, slightly weathered, medium bedded to thin bedded, gray, dark gray , fine grained to medium grained, soft SHALE and hard LIMESTONE, dip of 50 degrees, (RQD = 0 %) Silurian [Wills Creek Shale and Bloomsburg Formation]	Shale and Limestone	RQD=0 Rec=83%			
385	50			14	60	Interbedded, slightly weathered, medium bedded to thin bedded, gray, dark gray , fine grained to medium grained, soft SHALE and hard LIMESTONE, dip of 45 degrees, (RQD = 86 %) Silurian [Wills Creek Shale and Bloomsburg Formation]	Shale and Limestone	RQD=86 Rec=100%			
380	55			15	59			RQD=86 Rec=98%			
375	60					Bottom of test boring at 60 feet					

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LOCATION: I-68 HDD Crossing
Washington County
Hancock, MD

The stratification lines represent approximate boundaries. The transition may be gradual.

DATE STARTED: 6/22/16
DATE COMPLETED: 6/22/16
COMPLETION DEPTH 70.0 ft
BENCHMARK: N/A
ELEVATION: 447 ft
LATITUDE: 39.7117556°
LONGITUDE: 78.2086167°
STATION: N/A OFFSET: N/A
REMARKS:

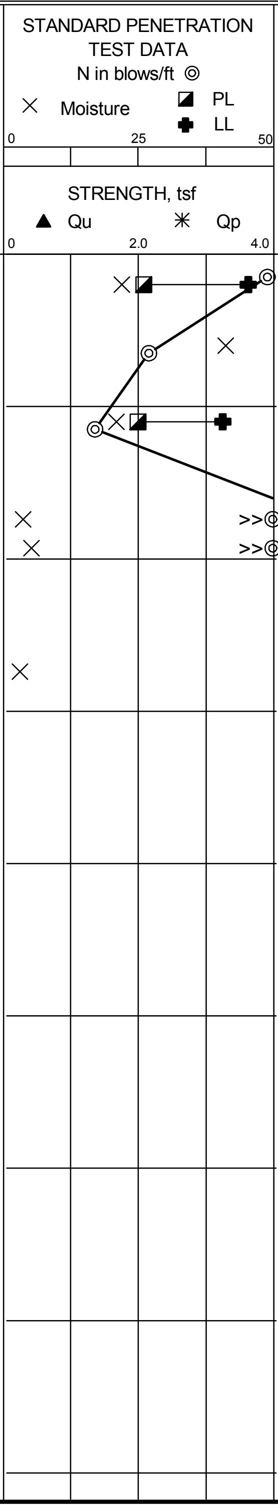
DRILL COMPANY: Connelly Drilling, Inc.
DRILLER: Tom Chew LOGGED BY: J. Thonnfend
DRILL RIG: Diedrich D-50
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: 2-in SS1.874-in Core Standard
HAMMER TYPE: Automatic
EFFICIENCY N/A
REVIEWED BY: Lubomir Peytchev

BORING GO-5

Water			
▽	While Drilling	Dry	feet
▼	Upon Completion	Dry	feet
▽	Delay	N/A	feet

BORING LOCATION:

Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS) RQD & Recovery % (NX)	Moisture, %	Additional Remarks
445	0			1	6	Approximately 3 inches of Topsoil	Top Soil	1-31-18 N=49	22	LL = 45 PL = 26 Fines=29.4%
						Dense to medium dense, moist, brown, sandy **clayey SAND (USCS SC)** some gravel, roots. **Alluvium**	SC			
				2	4			17-18-9 N=27	41	
440	5			3	18	Medium dense, moist, brown, olive brown, dark brown **clayey SAND (USCS SC)** some gravel, traceshale fragments. **Residuum**	SC	3-7-10 N=17	21	LL = 41 PL = 25 Fines=23.9%
				4	5	Weathered Rock, gray, dark gray, soft SHALE and hard LIMESTONE, Silurian [Wills Creek Shale and Bloomsburg Formation]	WEATHERED SHALE	50/5"	4	
435	10			5	3			50/3"	5	
430	15			6	18	Interbedded, slightly weathered, medium bedded to thin bedded, gray, dark gray , fine grained to medium grained, soft SHALE and hard LIMESTONE, dip of 55 degrees, (RQD = 80 %) Silurian [Wills Creek Shale and Bloomsburg Formation]	Shale and Limestone	RQD=61 Rec=100%	3	
				7	60			RQD=80 Rec=100%		
425	20			8	60	Interbedded, slightly weathered, medium bedded to thin bedded, gray, dark gray , fine grained to medium grained, soft SHALE and hard LIMESTONE, dip of 48 degrees, (RQD = 68 %) Silurian [Wills Creek Shale and Bloomsburg Formation]	Shale and Limestone	RQD=68 Rec=100%		
420	25			9	60	Interbedded, slightly weathered, medium bedded to thin bedded, gray, dark gray , fine grained to medium grained, soft SHALE and hard LIMESTONE, dip of 50 degrees, (RQD = 95 %) Silurian [Wills Creek Shale and Bloomsburg Formation]	Shale and Limestone	RQD=95 Rec=100%		
415	30			10	60	Interbedded, slightly weathered, medium bedded to thin bedded, gray, dark gray , fine grained to medium grained, soft SHALE and hard LIMESTONE, dip of 50 degrees, (RQD = 57 %) Silurian [Wills Creek Shale and Bloomsburg Formation]	Shale and Limestone	RQD=57 Rec=100%		
410	35			11	60	Interbedded, slightly weathered, medium bedded to thin bedded, gray, dark gray , fine grained to medium grained, soft SHALE and hard LIMESTONE, dip of 50 degrees, (RQD = 93 %) Silurian [Wills Creek Shale and Bloomsburg Formation]	Shale and Limestone	RQD=93 Rec=100%		
	40									



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PROJECT NO.: 0512719-1
PROJECT: 6493-Eastern Panhandle Expansion
LOCATION: I-68 HDD Crossing
Washington County
Hancock, MD

The stratification lines represent approximate boundaries. The transition may be gradual.

DATE STARTED: 6/22/16
DATE COMPLETED: 6/22/16
COMPLETION DEPTH 70.0 ft
BENCHMARK: N/A
ELEVATION: 447 ft
LATITUDE: 39.7117556°
LONGITUDE: 78.2086167°
STATION: N/A **OFFSET:** N/A
REMARKS:

DRILL COMPANY: Connelly Drilling, Inc.
DRILLER: Tom Chew **LOGGED BY:** J. Thonnfend
DRILL RIG: Diedrich D-50
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: 2-in SS1.874-in Core Standard
HAMMER TYPE: Automatic
EFFICIENCY N/A
REVIEWED BY: Lubomir Peytchev

BORING GO-5

Water			
▽	While Drilling	Dry	feet
▼	Upon Completion	Dry	feet
▼	Delay	N/A	feet

BORING LOCATION:

Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS) RQD & Recovery % (NX)	Moisture, %	STANDARD PENETRATION TEST DATA N in blows/ft ◎ × Moisture ◪ PL ✚ LL 0 25 50 STRENGTH, tsf ▲ Qu ✳ Qp 0 2.0 4.0	Additional Remarks
405	40			12	60	Interbedded, slightly weathered, medium bedded to thin bedded, gray, dark gray , fine grained to medium grained, soft SHALE and hard LIMESTONE, dip of 43 degrees, (RQD = 83 %) Silurian [Wills Creek Shale and Bloomsburg Formation]	Shale and Limestone	RQD=83 Rec=100%			
400	45			13	60	Interbedded, slightly weathered, medium bedded to thin bedded, gray, dark gray , fine grained to medium grained, soft SHALE and hard LIMESTONE, dip of 38 degrees, (RQD = 97 %) Silurian [Wills Creek Shale and Bloomsburg Formation]	Shale and Limestone	RQD=97 Rec=100%			
395	50			14	60	Interbedded, slightly weathered, medium bedded to thin bedded, gray, dark gray , fine grained to medium grained, soft SHALE and hard LIMESTONE, dip of 40 degrees, (RQD = 67 %) Silurian [Wills Creek Shale and Bloomsburg Formation]	Shale and Limestone	RQD=67 Rec=100%			
390	55			15	60	Interbedded, slightly weathered, medium bedded to thin bedded, gray, dark gray , fine grained to medium grained, soft SHALE and hard LIMESTONE, dip of 50 degrees, (RQD = 90 %) Silurian [Wills Creek Shale and Bloomsburg Formation]	Shale and Limestone	RQD=90 Rec=100%			
385	60			16	60	Interbedded, slightly weathered, medium bedded to thin bedded, gray, dark gray , fine grained to medium grained, soft SHALE and hard LIMESTONE, dip of 53 degrees, (RQD = 93 %) Silurian [Wills Creek Shale and Bloomsburg Formation]	Shale and Limestone	RQD=93 Rec=100%			
380	65			17	60			RQD=93 Rec=100%			
	70					Bottom of test boring at 70 feet					

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LOCATION: I-68 HDD Crossing
Washington County
Hancock, MD

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GENERAL NOTES

SAMPLE IDENTIFICATION

The Unified Soil Classification System (USCS), AASHTO 1988 and ASTM designations D2487 and D-2488 are used to identify the encountered materials unless otherwise noted. Coarse-grained soils are defined as having more than 50% of their dry weight retained on a #200 sieve (0.075mm); they are described as: boulders, cobbles, gravel or sand. Fine-grained soils have less than 50% of their dry weight retained on a #200 sieve; they are defined as silts or clay depending on their Atterberg Limit attributes. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size.

DRILLING AND SAMPLING SYMBOLS

SFA: Solid Flight Auger - typically 4" diameter flights, except where noted.
HSA: Hollow Stem Auger - typically 3¼" or 4¼ I.D. openings, except where noted.
M.R.: Mud Rotary - Uses a rotary head with Bentonite or Polymer Slurry
R.C.: Diamond Bit Core Sampler
H.A.: Hand Auger
P.A.: Power Auger - Handheld motorized auger

☒ SS: Split-Spoon - 1 3/8" I.D., 2" O.D., except where noted.
■ ST: Shelby Tube - 3" O.D., except where noted.
▯ RC: Rock Core
⬇ TC: Texas Cone
✋ BS: Bulk Sample
⁙ PM: Pressuremeter
CPT-U: Cone Penetrometer Testing with Pore-Pressure Readings

SOIL PROPERTY SYMBOLS

N: Standard "N" penetration: Blows per foot of a 140 pound hammer falling 30 inches on a 2-inch O.D. Split-Spoon.
N_{60}: A "N" penetration value corrected to an equivalent 60% hammer energy transfer efficiency (ETR)
Q_u: Unconfined compressive strength, TSF
Q_p: Pocket penetrometer value, unconfined compressive strength, TSF
w%: Moisture/water content, %
LL: Liquid Limit, %
PL: Plastic Limit, %
PI: Plasticity Index = (LL-PL),%
DD: Dry unit weight, pcf
▼,▽,▼: Apparent groundwater level at time noted

RELATIVE DENSITY OF COARSE-GRAINED SOILS

Relative Density	N - Blows/foot
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	50 - 80
Extremely Dense	80+

ANGULARITY OF COARSE-GRAINED PARTICLES

Description	Criteria
Angular:	Particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular:	Particles are similar to angular description, but have rounded edges
Subrounded:	Particles have nearly plane sides, but have well-rounded corners and edges
Rounded:	Particles have smoothly curved sides and no edges

GRAIN-SIZE TERMINOLOGY

Component	Size Range
Boulders:	Over 300 mm (>12 in.)
Cobbles:	75 mm to 300 mm (3 in. to 12 in.)
Coarse-Grained Gravel:	19 mm to 75 mm (¾ in. to 3 in.)
Fine-Grained Gravel:	4.75 mm to 19 mm (No.4 to ¾ in.)
Coarse-Grained Sand:	2 mm to 4.75 mm (No.10 to No.4)
Medium-Grained Sand:	0.42 mm to 2 mm (No.40 to No.10)
Fine-Grained Sand:	0.075 mm to 0.42 mm (No. 200 to No.40)
Silt:	0.005 mm to 0.075 mm
Clay:	<0.005 mm

PARTICLE SHAPE

Description	Criteria
Flat:	Particles with width/thickness ratio > 3
Elongated:	Particles with length/width ratio > 3
Flat & Elongated:	Particles meet criteria for both flat and elongated

RELATIVE PROPORTIONS OF FINES

Descriptive Term	% Dry Weight
Trace:	< 5%
With:	5% to 12%
Modifier:	>12%

GENERAL NOTES

(Continued)

CONSISTENCY OF FINE-GRAINED SOILS

Q_U - TSF	N - Blows/foot	Consistency
0 - 0.25	0 - 2	Very Soft
0.25 - 0.50	2 - 4	Soft
0.50 - 1.00	4 - 8	Firm (Medium Stiff)
1.00 - 2.00	8 - 15	Stiff
2.00 - 4.00	15 - 30	Very Stiff
4.00 - 8.00	30 - 50	Hard
8.00+	50+	Very Hard

MOISTURE CONDITION DESCRIPTION

Description	**Criteria**
Dry:	Absence of moisture, dusty, dry to the touch
Moist:	Damp but no visible water
Wet:	Visible free water, usually soil is below water table

RELATIVE PROPORTIONS OF SAND AND GRAVEL

Descriptive Term	**% Dry Weight**
Trace:	< 15%
With:	15% to 30%
Modifier:	>30%

STRUCTURE DESCRIPTION

Description	**Criteria**
Stratified:	Alternating layers of varying material or color with layers at least ¼-inch (6 mm) thick
Laminated:	Alternating layers of varying material or color with layers less than ¼-inch (6 mm) thick
Fissured:	Breaks along definite planes of fracture with little resistance to fracturing
Slickensided:	Fracture planes appear polished or glossy, sometimes striated

Description	**Criteria**
Blocky:	Cohesive soil that can be broken down into small angular lumps which resist further breakdown
Lensed:	Inclusion of small pockets of different soils
Layer:	Inclusion greater than 3 inches thick (75 mm)
Seam:	Inclusion 1/8-inch to 3 inches (3 to 75 mm) thick extending through the sample
Parting:	Inclusion less than 1/8-inch (3 mm) thick

SCALE OF RELATIVE ROCK HARDNESS

Q_U - TSF	Consistency
2.5 - 10	Extremely Soft
10 - 50	Very Soft
50 - 250	Soft
250 - 525	Medium Hard
525 - 1,050	Moderately Hard
1,050 - 2,600	Hard
>2,600	Very Hard

ROCK BEDDING THICKNESSES

Description	**Criteria**
Very Thick Bedded	Greater than 3-foot (>1.0 m)
Thick Bedded	1-foot to 3-foot (0.3 m to 1.0 m)
Medium Bedded	4-inch to 1-foot (0.1 m to 0.3 m)
Thin Bedded	1¼-inch to 4-inch (30 mm to 100 mm)
Very Thin Bedded	½-inch to 1¼-inch (10 mm to 30 mm)
Thickly Laminated	1/8-inch to ½-inch (3 mm to 10 mm)
Thinly Laminated	1/8-inch or less "paper thin" (<3 mm)

ROCK VOIDS

Voids	**Void Diameter**
Pit	<6 mm (<0.25 in)
Vug	6 mm to 50 mm (0.25 in to 2 in)
Cavity	50 mm to 600 mm (2 in to 24 in)
Cave	>600 mm (>24 in)

GRAIN-SIZED TERMINOLOGY

(Typically Sedimentary Rock)

Component	**Size Range**
Very Coarse Grained	>4.76 mm
Coarse Grained	2.0 mm - 4.76 mm
Medium Grained	0.42 mm - 2.0 mm
Fine Grained	0.075 mm - 0.42 mm
Very Fine Grained	<0.075 mm

ROCK QUALITY DESCRIPTION

Rock Mass Description	**RQD Value**
Excellent	90 -100
Good	75 - 90
Fair	50 - 75
Poor	25 -50
Very Poor	Less than 25

DEGREE OF WEATHERING

Slightly Weathered:	Rock generally fresh, joints stained and discoloration extends into rock up to 25 mm (1 in), open joints may contain clay, core rings under hammer impact.
Weathered:	Rock mass is decomposed 50% or less, significant portions of the rock show discoloration and weathering effects, cores cannot be broken by hand or scraped by knife.
Highly Weathered:	Rock mass is more than 50% decomposed, complete discoloration of rock fabric, core may be extremely broken and gives clunk sound when struck by hammer, may be shaved with a knife.

SOIL CLASSIFICATION CHART

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	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, SILTY CLAYS, LEAN 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 SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

MATERIAL AND SAMPLE SYMBOLS LIST

MATERIAL AND SAMPLE SYMBOLS LIST

Pavement/Soils

- ▨ TOPS-TOPSOIL
- ▨ SC/CH
- ▨ CH/CL
- ▨ CH/MH
- ▨ CH/SC
- ▨ CL/ML
- ▨ CL/SC
- ▨ CL/CH
- ▨ GP/GW
- ▨ CRA Crushed Aggregate
- ▨ GW/GP
- ▨ ML/MH
- ▨ GC/SC
- ▨ OH/OL
- ▨ GP/SP
- ▨ OL/OH
- ▨ PT Peat
- ▨ OH Organic
- ▨ SC/CL
- ▨ OL Organic
- ▨ SC/GC
- ▨ SC-SM
- ▨ SP/SW
- ▨ SM/GM
- ▨ SM/MH
- ▨ SM/ML
- ▨ SM/SC
- ▨ SP/GP
- ▨ SW/SP

Sedimentary Rocks

- ▨ BLD-Boulder Bed
- ▨ DLS Dolostone
- ▨ LST-DLS-Interbedded Limestone/Dolostone

Igneous Rocks

- ▨ CHT Charnocktite

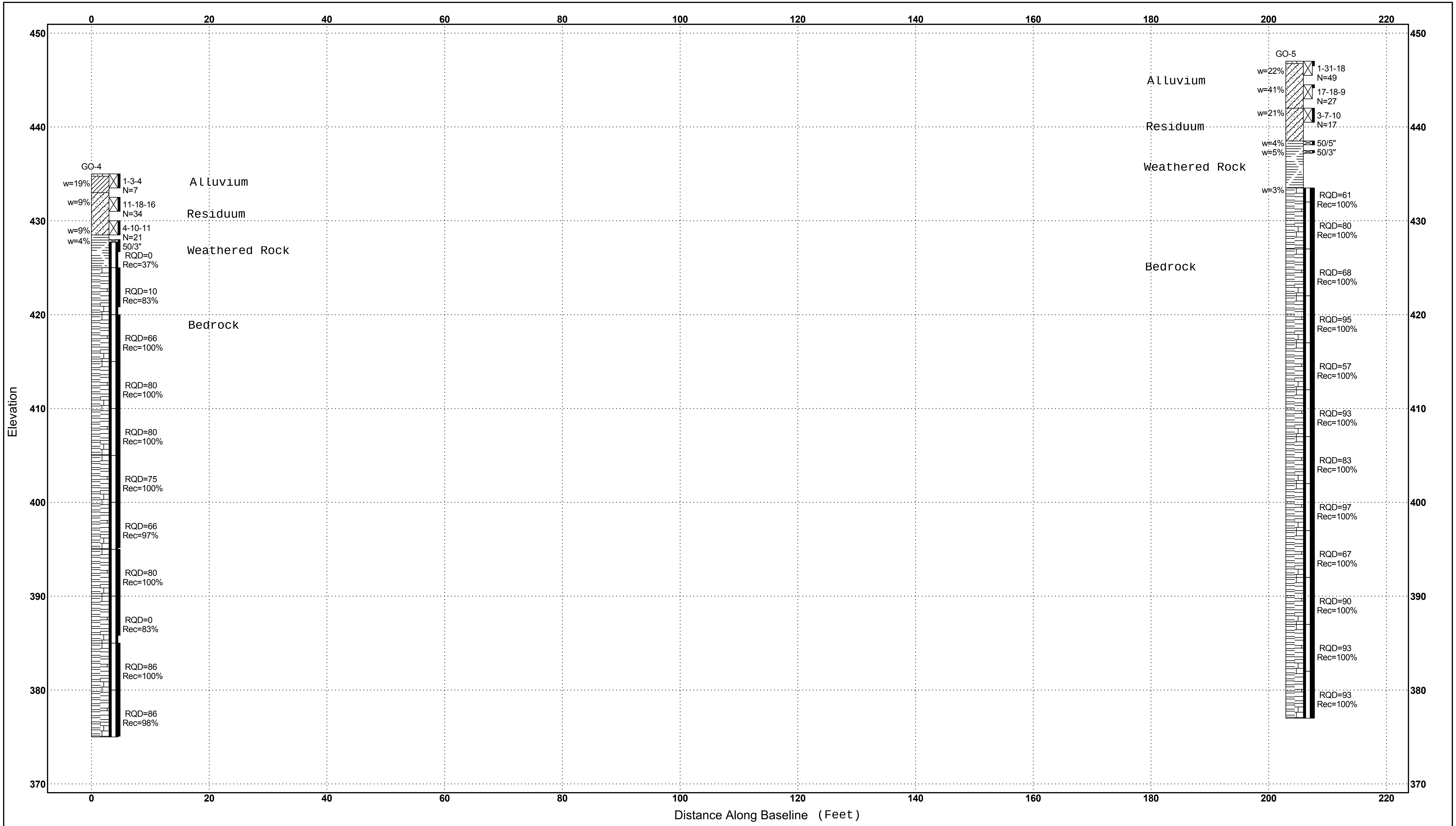
Metamorphic Rocks

- ▨ MSLS Metasiltstone
- ▨ MSST Metasandstone
- ▨ QZT - Quartzite
- ▨ SPS Soapstone
- ▨ MBST Metabasalt
- ▨ MBL Marble

Sampling

Cross Section Showing the General Stratigraphy

EnSite USA Project Number 6493
Eastern Panhandle Expansion, I-68 Crossing,
PSI Project Number 0512719-1
July 12, 2016



Professional Service Industries, Inc.
2930 Eskridge Rd
Fairfax, VA 22031

Profile (GO-4 and GO-5)

6493-Eastern Panhandle Expansion
PSI Project Number: 0512719-1

I-68 HDD Crossing
Washington County
Hancock, MD

Laboratory Test Results

Laboratory Summary Sheet

Sheet 1 of 1

Borehole	Approx. Depth	Liquid Limit	Plastic Limit	Plasticity Index	Qu (tsf)	%<#200 Sieve	Est. Specific Gravity	Water Content (%)	Dry Density (pcf)	Satur- ation (%)	Void Ratio
GO-4	1	35	21	14		77.9%		19			
GO-4	3							9			
GO-4	6	29	17	12		20.1%		9			
GO-4	7.15							4			
GO-5	1	45	26	19		29.4%		22			
GO-5	3							41			
GO-5	5.5	41	25	16		23.9%		21			
GO-5	8.7							4			
GO-5	9.65							5			
GO-5	13.7							3			

psi

Professional Service Industries
2930 Eskridge Rd
Fairfax, VA 22031
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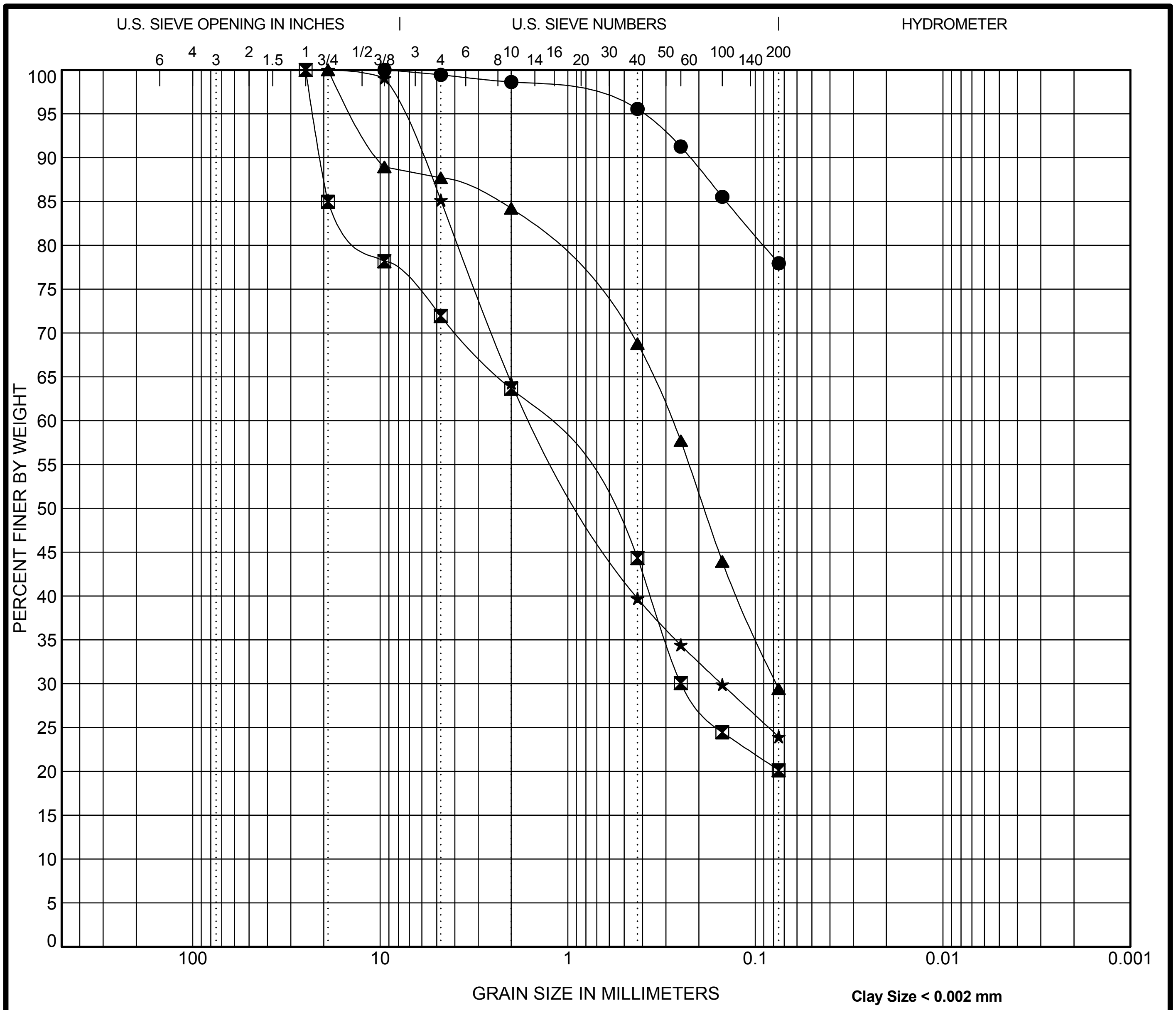
Fax: (703) 560-7931

Summary of Laboratory Results

PSI Job No.: 0512719-1
Project: 6493-Eastern Panhandle Expansion
Location: I-68 HDD Crossing
Washington County
Hancock, MD



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	**GO-4**	**1.0**	Lean CLAY (USCS CL)	**35**	**21**	**14**		
⊠	**GO-4**	**6.0**	Clayey SAND (USCS SC) with gravel	**29**	**17**	**12**		
▲	**GO-5**	**1.0**	Clayey SAND (USCS SC)	**45**	**26**	**19**		
★	**GO-5**	**5.5**	Clayey SAND (USCS SC)	**41**	**25**	**16**		

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**GO-4**	**1.0**	**9.5**				**0.6**	**21.5**	**77.9**	
⊠	**GO-4**	**6.0**	**25**	**1.495**	**0.249**		**28.1**	**51.8**	**20.1**	
▲	**GO-5**	**1.0**	**19**	**0.279**	**0.077**		**12.3**	**58.3**	**29.4**	
★	**GO-5**	**5.5**	**19**	**1.527**	**0.152**		**14.8**	**61.2**	**23.9**	

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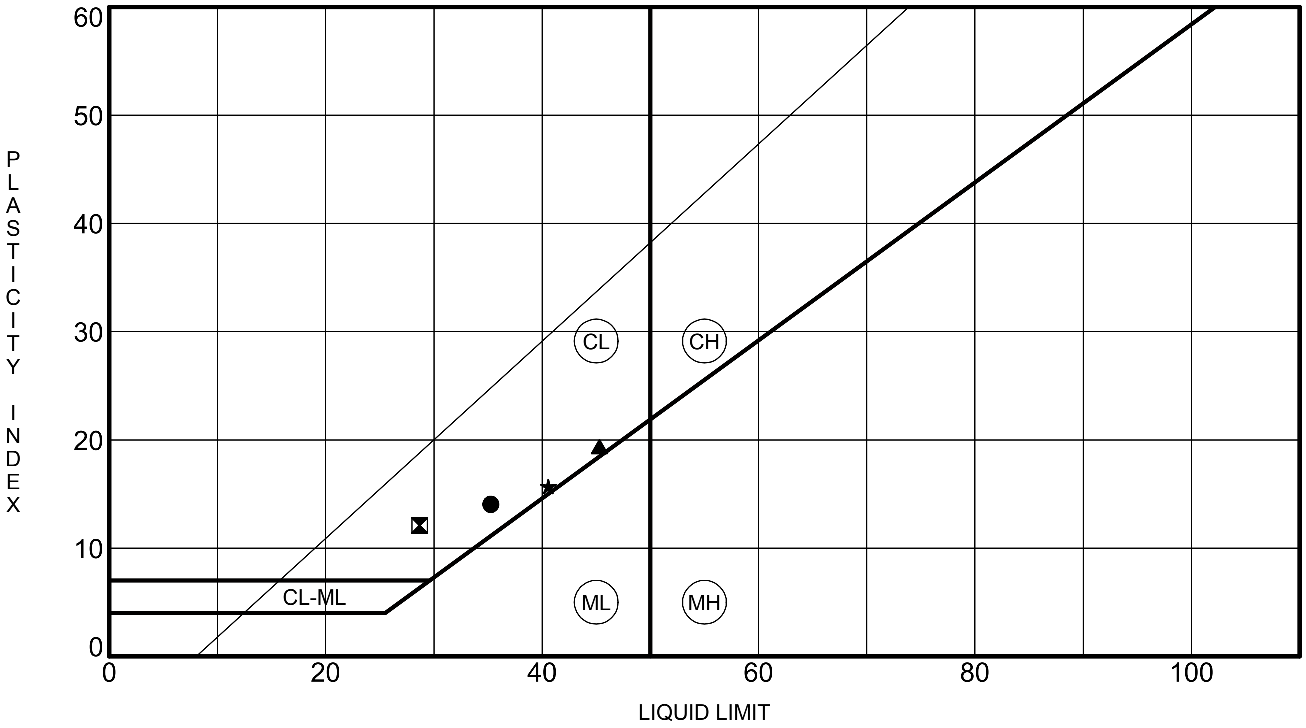
GRAIN SIZE DISTRIBUTION

Project: 6493-Eastern Panhandle Expansion
PSI Job No.: 0512719-1
Location: I-68 HDD Crossing
Washington County



	Boring	Depth (ft)	LL	PL	PI	Fines	Classification (*Visual)
●	**GO-4**	**1.0**	**35**	**21**	**14**	**77.9**	`Lean CLAY (USCS CL)`
⧗	**GO-4**	**6.0**	**29**	**17**	**12**	**20.1**	`Lean CLAY (USCS CL)`
▲	**GO-5**	**1.0**	**45**	**26**	**19**	**29.4**	`Lean CLAY (USCS CL)`
★	**GO-5**	**5.5**	**41**	**25**	**16**	**23.9**	`Lean CLAY (USCS CL)`

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ATTERBERG LIMIT RESULTS

PSI Job No.: 0512719-1
Project: 6493-Eastern Panhandle Expansion
Location: I-68 HDD Crossing
Washington County

Slake Durability Test Results

Slake Durability of Shales and Similar Weak Rocks - ASTM D4644

Client	Professional Service Industries, Inc. (PSI)	Boring	GO-4
Client Project	Proposed Eastern Panhandle Expansion I-68 Crossing	Depth	38.0'-39.0'
Project No.	37538	Sample	RC-11
	6493 Eastern Panhandle Expansion, I-68 HDD Crossing	Lab Sample No.	37538002

Visual Description: Dark Reddish Gray Siltstone

Initial Water Content

Drum ID	A
Drum + Wet Shale, gm	1772.14
Drum + Dry Shale, gm	1767.93
Drum Wt., gm	1244.86
Water Content, %	1%
Initial Dry Shale Weight, gm	523.07
Water Temerature Before Cycle 1, *C	21.4
Water Temerature After Cycle 1, *C	21.6
Average Temp during Cycle 1, *C	21.5
Drum + Dry Shale after Cycle 1, gm	1764.73
Dry Shale after Cycle 1	519.87
Slake Durability Index (First cycle)	**99.4%**
Water Temerature Before Cycle 2, *C	21
Water Temerature After Cycle 2, *C	21.6
Average Temp during Cycle 2, *C	21.3
Drum + Dry Shale after Cycle 2, gm	1759.78
Dry Shale after Cycle 2	514.92
Slake Durability Index (Second cycle)	**98.4%**

Type II—Retained specimen consist of large and small fragments.

Initial Photograph

Final Photograph

Input Validation: tmp Reviewed By: SVG Date Tested: 6/28/2016

Slake Durability of Shales and Similar Weak Rocks - ASTM D4644

Client	Professional Service Industries, Inc. (PSI)	Boring	GO-4
Client Project	Proposed Eastern Panhandle Expansion I-68 Crossing	Depth	54.0'-55.0'
Project No.	37538	Sample	RC-14
	6493 Eastern Panhandle Expansion, I-68 HDD Crossing	Lab Sample No.	37538003

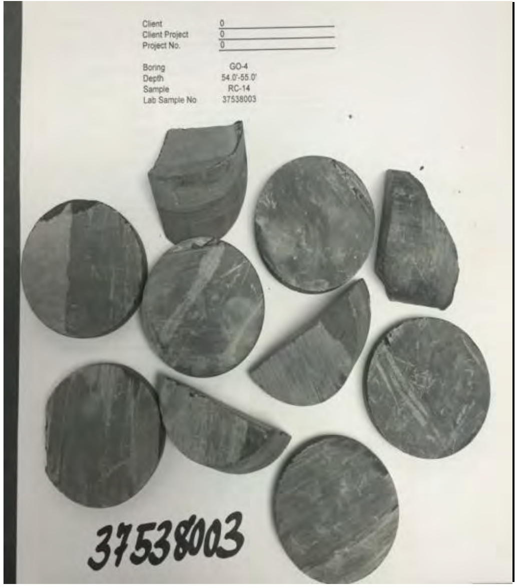
Visual Description: Gray Shale

Initial Water Content

Drum ID	B
Drum + Wet Shale, gm	1769.99
Drum + Dry Shale, gm	1767.23
Drum Wt., gm	1244.29
Water Content, %	1%
Initial Dry Shale Weight, gm	522.94
Water Temerature Before Cycle 1, *C	21.6
Water Temerature After Cycle 1, *C	21.7
Average Temp during Cycle 1, *C	21.65
Drum + Dry Shale after Cycle 1, gm	1764.46
Dry Shale after Cycle 1	520.17
Slake Durability Index (First cycle)	**99.5%**
Water Temerature Before Cycle 2, *C	21.6
Water Temerature After Cycle 2, *C	21.9
Average Temp during Cycle 2, *C	21.75
Drum + Dry Shale after Cycle 2, gm	1761.47
Dry Shale after Cycle 2	517.18
Slake Durability Index (Second cycle)	**98.9%**

Type II—Retained specimen consist of large and small fragments.

Initial Photograph



Final Photograph

Input Validation: tmp Reviewed By: SVG Date Tested: 6/28/2016

Slake Durability of Shales and Similar Weak Rocks - ASTM D4644

Client	Professional Service Industries,	Boring	GO-5
Client Project	Proposed Eastern Panhandle Ex	Depth	22.0'-23.0'
Project No.	37538	Sample	RC-9
		Lab Sample No.	37538004

6493 Eastern Panhandle Expansion, I-68 HDD Crossing

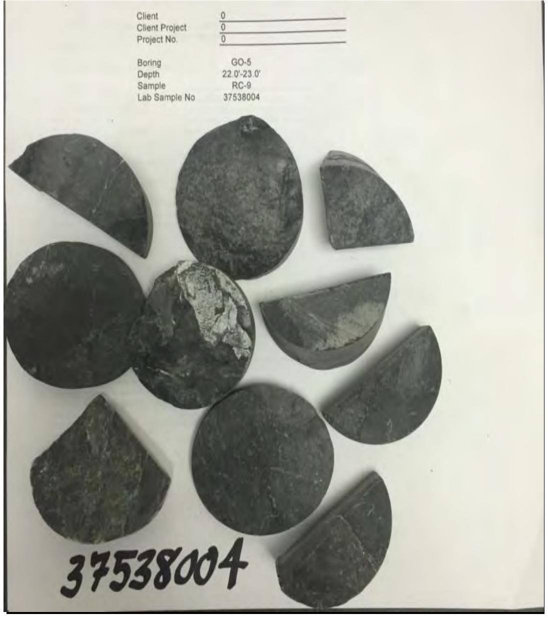
Visual Description: Gray Shale

Initial Water Content

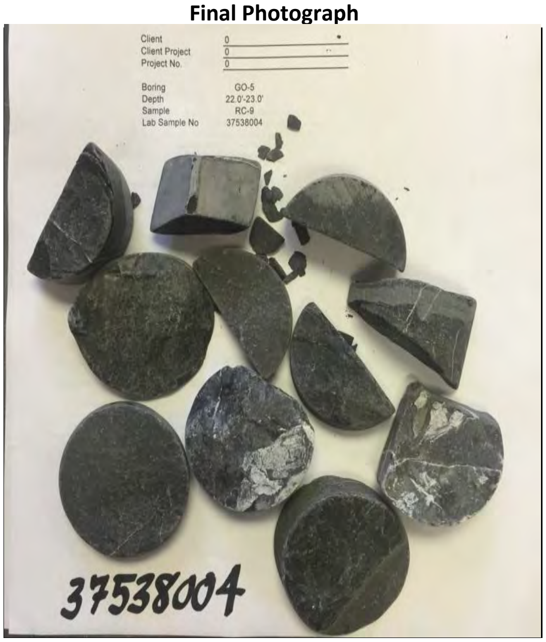
Drum ID	D
Drum + Wet Shale, gm	1779.82
Drum + Dry Shale, gm	1777.53
Drum Wt., gm	1260.98
Water Content, %	0.44%
Initial Dry Shale Weight, gm	516.55
Water Temerature Before Cycle 1, *C	21.2
Water Temerature After Cycle 1, *C	21.5
Average Temp during Cycle 1, *C	21.35
Drum + Dry Shale after Cycle 1, gm	1774.11
Dry Shale after Cycle 1	513.13
Slake Durability Index (First cycle)	**99.3%**
Water Temerature Before Cycle 2, *C	21.2
Water Temerature After Cycle 2, *C	21.5
Average Temp during Cycle 2, *C	21.35
Drum + Dry Shale after Cycle 2, gm	1771.28
Dry Shale after Cycle 2	510.3
Slake Durability Index (Second cycle)	**98.8%**

Type II—Retained specimen consist of large and small fragments.

Initial Photograph



Final Photograph



Input Validation: tmp Reviewed By: SVG Date Tested: 6/28/2016

Slake Durability of Shales and Similar Weak Rocks - ASTM D4644

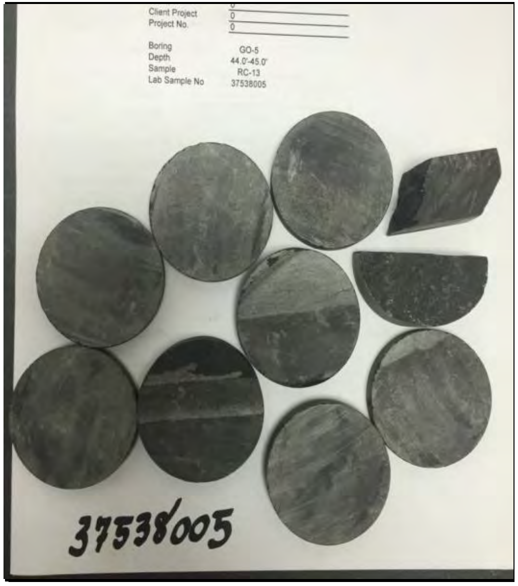
Client	Professional Service Industries, Inc. (PSI)	Boring	GO-5
Client Project	Proposed Eastern Panhandle Expansion I-68 Crossing	Depth	44.0'-45.0'
Project No.	37538	Sample	RC-13
		Lab Sample No.	37538005
	6493 Eastern Panhandle Expansion, I-68 HDD Crossing		
Visual Description:	Gray Shale		

Initial Water Content

Drum ID	C
Drum + Wet Shale, gm	1755.36
Drum + Dry Shale, gm	1753.65
Drum Wt., gm	1257.69
Water Content, %	0.34%
Initial Dry Shale Weight, gm	495.96
Water Temerature Before Cycle 1, *C	21.4
Water Temerature After Cycle 1, *C	21.6
Average Temp during Cycle 1, *C	21.5
Drum + Dry Shale after Cycle 1, gm	1752.4
Dry Shale after Cycle 1	494.71
Slake Durability Index (First cycle)	**99.7%**
Water Temerature Before Cycle 2, *C	21.6
Water Temerature After Cycle 2, *C	22
Average Temp during Cycle 2, *C	21.8
Drum + Dry Shale after Cycle 2, gm	1751.12
Dry Shale after Cycle 2	493.43
Slake Durability Index (Second cycle)	**99.5%**

Type I—Retained specimen remain virtually unchanged

Initial Photograph



Final Photograph

Input Validation: tmp　　Reviewed By: SVG　　Date Tested: 6/28/2016

Slake Durability of Shales and Similar Weak Rocks - ASTM D4644

Client	Professional Service Industries, Inc. (PSI)	Boring	GO-5
Client Project	Proposed Eastern Panhandle Expansion I-68 Crossing	Depth	64.0'-65.0'
Project No.	37538	Sample	RC-17
		Lab Sample No.	37538006

6493 Eastern Panhandle Expansion, I-68 HDD Crossing

Visual Description: Gray Shale

Initial Water Content

Drum ID	D
Drum + Wet Shale, gm	1759.33
Drum + Dry Shale, gm	1757.69
Drum Wt., gm	1260.82
Water Content, %	0.33%
Initial Dry Shale Weight, gm	496.87
Water Temerature Before Cycle 1, *C	19.7
Water Temerature After Cycle 1, *C	20
Average Temp during Cycle 1, *C	19.85
Drum + Dry Shale after Cycle 1, gm	1755.39
Dry Shale after Cycle 1	494.57
Slake Durability Index (First cycle)	**99.5%**
Water Temerature Before Cycle 2, *C	21
Water Temerature After Cycle 2, *C	21.6
Average Temp during Cycle 2, *C	21.3
Drum + Dry Shale after Cycle 2, gm	1752.69
Dry Shale after Cycle 2	491.87
Slake Durability Index (Second cycle)	**99.0%**

Type II—Retained specimen consist of large and small fragments.

Initial Photograph

Final Photograph

Input Validation: tmp   Reviewed By: SVG   Date Tested: 6/30/2016

Soil Resistivity Test Results

EnSite USA Project Number 6493
Eastern Panhandle Expansion, I-68 Crossing,
PSI Project Number 0512719-1
July 12, 2016

Client Professional Service Industries, Inc. (PSI)
Client Project 6493 - Proposed Eastern Panhandle Expansion I-68 Crossing
Project No. 37538

						pH AASHTO T289			Soil Resistivity AASHTO T-288		
Lab Sample ID	**Boring**	**Depth**	**Sample**	**Sample Received**	**Matrix**	**Result**	Date Tested	Tested By	**Result, Ohm-cm**	Date Tested	Tested By
37538001	GO-4	1.0'-5.0'	SS: 1,2	6/27/2016	Soil	5.8	6/29/2016	TX	8950	6/29/2016	TX

Input Validation: tmp Reviewed By: SVG

Important Information About Your Geotechnical Report

Important Information about Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

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

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

Read the Full Report

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

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

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

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

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

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

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

Subsurface Conditions Can Change

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

Most Geotechnical Findings Are Professional Opinions

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

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual

subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

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

Do Not Redraw the Engineer's Logs

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

Give Contractors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

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

Geoenvironmental Concerns Are Not Covered

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

Obtain Professional Assistance To Deal with Mold

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

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

Membership in ASFE/THE BEST PEOPLE ON EARTH exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

ASFE
THE BEST PEOPLE ON EARTH

8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@asfe.org www.asfe.org

Information To Build On
Engineering • Consulting • Testing

July 22, 2016
Revised September 9, 2016

EnSite USA, Inc.
109 Fieldview Drive
Versailles, KY, 40383
Attn: Grace Northcutt, P.E.

Re: Report for Geotechnical Subsurface Exploration & Engineering Services
6493 – Eastern Panhandle Expansion
Potomac River Crossing, Preliminary Investigation
Washington County, Maryland and Morgan County, West Virginia.
PSI Project Number 0512713-1

Dear Ms. Northcutt:

Thank you for choosing Professional Service Industries, Inc. (PSI) as your consultant for the referenced project. Authorization to perform services was provided through PSI Proposal No. 0512-182348 dated June 13, 2016. The proposal was executed by Ms. Northcutt, P.E. representing EnSite USA, Inc.

This letter report presents the results of borings performed by PSI at two locations along the proposed HDD alignment at Potomac River Crossing. Approximate boring locations are presented in the Appendix Figures: 1A Site Vicinity Map and 1B: Boring Location Plans.

Scope of Services

PSI's services consisted of field exploration, laboratory testing, and preparation of a geotechnical engineering report for the proposed HDD location. Field work included drilling 2 test borings (Borings GO-1 and GO-2R), utilizing hollow-stem auger (HSA) drilling, wash rotary drilling, and rock coring in conformance with ASTM standards.

Laboratory testing determined unit weight, moisture content, Atterberg limits, grain size distribution tests, pH and resistivity testing, unconfined compressive strength and slake durability testing. All tests were performed per ASTM standards.

Summary of Field Exploration and Laboratory Testing

The borings were completed with a track-mounted drill rig with HSA in conformance with ASTM standards. Standard Penetration Testing (SPT) and split-spoon sampling of overburden soils was performed at 2.5 foot intervals for the first 10 feet and at 5-foot intervals thereafter to the termination depths to evaluate the strength and relative consistency of the soils encountered. Below auger refusal depth, rock coring was performed using NQ coring equipment. All recovered soil and rock samples were visually classified by a PSI geotechnical engineer and a graphical log developed for each boring. Boring depths and depths at which auger refusal were encountered are summarized in Table 1 below.

Table 1 – Summary of Boring Depths

Boring	Approximate Termination Depth (feet)	Ground Surface Elevation (feet, NAVD)	Approximate Depth/Elevation of Top of Weathered Rock	Approximate Depth/Elevation of Auger Refusal
GO-1	305	624	23 feet, EL ±601MSL	24 feet, EL ±600MSL
GO-2R	154	411	20 feet, EL ±391MSL	24 feet, EL ±387MSL

The boring logs included in the Appendix approximate depths and visual descriptions of overburden soil, underlying rock materials encountered, soil SPT test results, rock core recovery and quality designation (RQD) values, and measurements of groundwater depth where encountered. The total length of recovered rock core, divided by the length of the run, is referred to as rock core recovery and is expressed as a percentage. The Rock Quality Designation (RQD) is a measure of the rock mass quality and is defined as the total length of sound, intact rock core pieces 4 inches or more in length divided by the length of the rock core run, also expressed as a percentage. The rock core recovery and RQD values are indicated on the Boring Logs included with this report.

Geotechnical Investigation Results

A brief summary of subsurface stratigraphy as encountered at the various borings are presented below. All soil is classified per the Unified Soil Classification System (ASTM D-2487):

Surficial Materials: Approximately 3 inches of surficial topsoil were encountered at the ground surface of Borings GO-1 and GO-2R.

Alluvium with thickness from 2 feet to 16 feet consisting of soft to very stiff lean clay (CL) and fat clay (CH).

Residuum: Residual soil classified as clayey silt (ML) and lean clay (CL) were encountered to depths ranging from approximately 2 to 17 feet below existing surface grades at both borings. The residual soil was between 3 to 21 feet thick in the boring locations. SPT N-values in this layer ranged from approximately 6 to 30 blows per foot (BPF).

Weathered Rock: Typically consisting of weathered shale, weathered rock was encountered at both boring locations. The weathered rock samples consisted of soft shale with limestone floaters. SPT N-values were typically in excess of 50 BPF. Auger refusal was encountered within the weathered rock at depths ranging from approximately 23 to 24 feet below existing grades.

Bedrock: Bedrock materials encountered below the auger refusal depths consisted primarily of cyclic sequences of Limestone, and shale, with occasional layers of sandy shale, cherty limestone and sandstone. Voids were encountered in Boring GO-2R. Core recoveries ranged from 13 to 100 percent. RQD values ranged from 0 to 100 percent.

The above subsurface descriptions are of a generalized nature provided to highlight the major strata encountered. The boring logs included in the Appendix should be reviewed for specific information as to individual boring locations. The stratification lines shown on the boring logs represent the conditions only at the actual boring locations. The stratification lines represent the approximate boundaries between subsurface materials and the actual transition may be gradual.

Table 2 – Overburden Soil Classification Test Results

Boring	Sample No.	Sample Depth (feet)	USCS Classification (1)	Moisture Content (%)	Atterberg Limits			Grain-Size Distribution		
					Liquid Limit	Plastic Limit	Plasticity Index	Gravel (%)	Sand (%)	Fines (%)
GO-1	S-1	0.0 – 1.5		14						
GO-1	S-2	2.5 – 4.0	CH	21	55	27	28	0.0	2.6	97.4
GO-1	ST-1	2.0 – 4.0	CL	17	41	24	17			
GO-1	S-3	5.0 – 6.5		23						
GO-1	S-4	8.5 – 10.0	ML	25	39	27	12	0.0	0.2	99.8
GO-1	ST-2	11.0 – 11.5	ML	26	45	29	16	0.0	0.4	99.6
GO-1	S-5	13.5 – 15.0		21						
GO-1	ST-3	18.3 – 18.8	CL	21	44	26	18	0.0	10.0	90.0
GO-1	S-6	18.5 – 20.0		25						
GO-2R	S-1	0.0 – 1.5		21						
GO-2R	ST-1	1.5 – 2.0	CL	19	38	22	16	0.0	24.9	75.1
GO-2R	S-2	2.5 – 4.0	CH	30	65	25	38			
GO-2R	ST-2	2.0 – 4.0	CH	24	55	22	33			
GO-2R	S-3	5.0 – 6.5	CH	27	59	23	36	0.0	10.7	89.3
GO-2R	ST-3	8.5 – 9.0	CL	20	33	18	15	0.0	9.2	90.8
GO-2R	S-4	8.5 – 10.0		18						
GO-2R	S-5	13.5 – 15.0	CL	17				0.0	40.2	55.8
GO-2R	S-6	18.5 – 20.0		14						

(1) For USCS Soil Classification definitions, refer to the General Notes in Attachment

(2) ST – Shelby Tube soil sample

(3) S – Split spoon soil sample

Table 3 – Rock Recovery and RQD Field Coring Test Results

Boring	Depth (feet)	Elevation (feet)	Run Length (feet)	Recovery (%)	RQD (%)	Hardness (Moh,s)
GO-1	24 - 29	600 - 595	5	100	89	3
GO-1	29- 34	595 – 590	5	100	90	3
GO-1	34 - 39	590 – 585	5	100	100	3
GO-1	39 - 44	585 – 580	5	100	78	3
GO-1	44 - 49	580 – 575	5	100	88	3

Table 3 – Rock Recovery and RQD Field Coring Test Results Continued

Boring	Depth (feet)	Elevation (feet)	Run Length (feet)	Recovery (%)	RQD (%)	Hardness (Moh,s)
GO-1	49 - 54	575 – 570	5	100	90	3
GO-1	54 - 59	570 – 565	5	87	45	3
GO-1	59 - 64	565 – 560	5	100	83	3 - 4
GO-1	64 - 69	560 – 555	5	100	95	3 - 4
GO-1	69 - 74	555 – 550	5	100	92	3 - 4
GO-1	74 – 79	550 – 545	5	100	97	3 - 4
GO-1	79 – 84	545 – 540	5	100	94	3 - 4
GO-1	84 – 89	540 – 535	5	100	95	3 - 4
GO-1	89 – 94	535 – 530	5	100	95	3 - 4
GO-1	94 – 99	530 – 525	5	100	88	3 - 4
GO-1	99 – 104	525 – 520	5	100	67	3 - 4
GO-1	104 – 109	520 – 515	5	100	50	3 - 4
GO-1	109 – 114	515 – 510	5	100	47	3 - 4
GO-1	114 – 119	510 – 505	5	100	64	3 - 4
GO-1	119 – 124	505 – 500	5	100	53	3 - 4
GO-1	124 – 129	500 – 495	5	100	59	3 - 4
GO-1	129 – 134	495 – 490	5	100	100	3 - 4
GO-1	134 – 139	490 – 485	5	100	96	3 - 4
GO-1	139 – 144	485 – 480	5	100	93	3 - 4
GO-1	144 – 149	480 – 475	5	97	95	3 - 4
GO-1	149 – 154	475 – 470	5	100	98	3 - 4
GO-1	154 – 159	470 – 465	5	100	91	3 - 4
GO-1	159 – 164	465 – 460	5	98	96	3 - 4
GO-1	164 – 169	460 – 455	5	100	100	3 - 4
GO-1	169 – 174	455 – 450	5	100	73	3 - 4
GO-1	174 – 179	450 – 445	5	90	75	3 - 4
GO-1	179 – 184	445 – 440	5	100	75	3 - 4
GO-1	184 – 189	440 – 435	5	100	83	3 - 4

Table 3 – Rock Recovery and RQD Field Coring Test Results Continued

Boring	**Depth (feet)**	**Elevation (feet)**	**Run Length (feet)**	**Recovery (%)**	**RQD (%)**	**Hardness (Moh,s)**
GO-1	189 – 194	435 – 430	5	100	55	3 - 4
GO-1	194 – 199	430 – 425	5	100	63	3 - 4
GO-1	199 – 204	425 – 420	5	100	88	3 - 4
GO-1	204 - 209	420 – 415	5	100	88	3 - 4
GO-1	209 - 214	415 – 410	5	100	93	3 - 4
GO-1	214 - 219	410 – 405	5	100	88	3 - 4
GO-1	219 - 224	405 – 400	5	100	93	3 - 4
GO-1	224 - 229	400 – 395	5	100	51	3 - 4
GO-1	229 – 234	395 – 390	5	93	70	3 - 4
GO-1	234 – 239	390 – 385	5	100	65	3 - 4
GO-1	239 – 244	385 – 380	5	100	75	3 - 4
GO-1	244 – 249	380 – 375	5	97	68	3 - 4
GO-1	249 – 254	375 – 370	5	95	85	3 - 4
GO-1	254 – 259	370 – 365	5	95	83	3 - 4
GO-1	259 – 264	365 – 360	5	100	46	3 - 4
GO-1	264 – 269	360 – 355	5	43	0	0
GO-1	269 – 274	355 – 350	5	13	0	0
GO-1	274 – 279	350 – 345	5	87	13	3
GO-1	279 – 284	345 – 340	5	87	30	3
GO-1	284 – 289	340 – 335	5	82	25	7 - 8
GO-1	289 – 294	335 – 330	5	100	58	7 - 8
GO-1	294 – 299	330 – 325	5	100	62	7 - 8
GO-1	299 – 304	325 - 320	5	100	78	7 - 8
GO-1	304 – 305	320 – 319	1	100	100	7 - 8
GO-2R	24 – 29	387 – 382	5	90	35	3 - 4
GO-2R	29 – 34	382 – 377	5	100	40	3 - 4
GO-2R	34 – 39	377 – 372	5	100	95	3

Table 3 – Rock Recovery and RQD Field Coring Test Results Continued

Boring	**Depth (feet)**	**Elevation (feet)**	**Run Length (feet)**	**Recovery (%)**	**RQD (%)**	**Hardness (Moh,s)**
GO-2R	39 – 44	372 – 367	5	100	52	3
GO-2R	44 – 49	367 – 362	5	100	82	3
GO-2R	49 – 54	362 – 357	5	88	70	3
GO-2R	54 – 59	357 – 352	5	95	93	3
GO-2R	59 – 64	352 – 347	5	100	78	3
GO-2R	64 – 69	347 – 342	5	95	52	3
GO-2R	69 – 74	342 – 337	5	100	42	3
GO-2R	74 – 79	337 – 332	5	100	98	3
GO-2R	79 – 84	332 – 327	5	100	72	3
GO-2R	84 – 89	327 – 322	5	100	85	3
GO-2R	89 – 94	322 – 317	5	98	83	2.5 - 3
GO-2R	94 – 99	317 – 312	5	95	70	2.5 - 3
GO-2R	99 – 104	312 – 307	5	100	66	2.5 - 3
GO-2R	104 – 109	307 – 302	5	85	62	2.5 - 3
GO-2R	109 – 114	302 – 297	5	82	18	2.5 - 3
GO-2R	114 – 119	297 – 292	5	100	73	2.5 - 3
GO-2R	119 – 124	292 – 287	5	93	85	2.5 - 3
GO-2R	124 – 129	287 – 282	5	100	95	2.5 - 3
GO-2R	129 – 134	282 – 277	5	55	23	2.5 - 3
GO-2R	134 – 139	277 – 272	5	87	58	2.5 - 3
GO-2R	139 – 144	272 – 267	5	100	60	2.5 - 3
GO-2R	144 – 149	267 – 262	5	87	17	4 - 5
GO-2R	149 – 154	262 – 257	5	100	75	4 - 5

Table 4 – Rock Unconfined Compressive Strength Test Results

Boring	Approximate Sample Depth (feet)	Rock Classification	Unit Weight (pcf)	Unconfined Compressive Strength	
				(psi)	(tsf)
GO-1	43.0 - 43.5	Shale	162.33	1310	94.3
GO-1	64.0 - 64.5	Shale	163.90	510	36.7
GO-1	98.0 - 98.5	Shale	160.29	1950	140.4
GO-1	149.5 - 150.0	Shale	165.00	2720	195.8
GO-1	195.0 - 195.5	Shale	158.80	1560	112.3
GO-1	233.0 - 233.5	Sandstone	130.92	970	69.8
GO-1	287.5 - 288.0	Limestone-Cherty	*	11570	833.1
GO-1	290.0 - 290.5	Shale	155.46	5680	408.9
GO-2R	27.0 - 27.5	Shale	167.27	6870	494.6
GO-2R	50.0 - 50.5	Shale	156.14	4210	303.1
GO-2R	84.0 - 84.5	Shale	172.74	3870	278.6
GO-2R	85.0 - 85.5	Shale	172.51	4340	312.5
GO-2R	116.0 - 116.5	Shale	169.63	3630	261.4
GO-2R	131.0 - 131.5	Shale	160.46	4400	316.8
* Test could not be performed					

Table 5 – Soil Unconfined Compressive Strength Test Results

Boring	Approximate Sample Depth (feet)	Water Content (%)	Dry Unit Weight (pcf)	Soil Classification	Unconfined Compressive Strength	
					(psf)	(tsf)
GO-1	4.0 – 6.0	16.7	116.8	ML	5530	2.76
GO-1	11.0 – 11.5	26.4	94.5	ML	2780	1.39
GO-1	18.3 – 18.8	21.2	97.2	CL	2300	1.15
GO-2R	1.5 – 2.0	13.8	100.3	CL	1440	0.72
GO-2R	4.0 – 6.0	24.4	101.5	CH	4464	2.23
GO-2R	8.5 - 9.0	20.2	104.0	CL	2180	1.09

The **durability** of the shale is a measurement of its deterioration over time interaction with the water weathering properties. The durability of the shale was determined on selected samples of shales per Slake Durability of Shales and Similar Weak Rocks, ASTM D-4644 Standard.

Table 6 – Slake Durability Test Results

Boring	Approximate Sample Depth (feet)	Rock Classification	Slake Durability Index First Cycle (%)	Slake Durability Index Second Cycle (%)
GO-1	51.0 – 52.0	Shale	99.5	99.3
GO-1	64.5 – 65.5	Shale	99.5	99.2
GO-1	95.0 – 96.0	Shale	94.8	90.0
GO-1	129.0 – 130.0	Shale	99.2	98.7
GO-1	203.0 – 204.0	Shale	66.1	42.9
GO-1	255.0 – 256.0	Shale	60.5	29.4
GO-2R	43.0 – 44.0	Shale	99.9	99.8
GO-2R	58.0 – 59.0	Shale	99.4	99.1
GO-2R	70.5 – 71.5	Shale	99.9	99.9
GO-2R	100.0 – 101.0	Shale	99.6	99.3
GO-2R	122.0 – 123.0	Shale	99.6	99.4
GO-2R	151.5 – 152.5	Shale	99.8	99.6

Two (2) representative soil samples were selected by PSI for soil resistivity testing. Table 7 below presents a summary of the test results. A detailed report is included in the Appendix.

Table 7 – Soil Resistivity Test Results

Location	GO-1	GO-2R
Depth (Foot)	4.0' to 6.0'	4.0' to 6.0'
pH - AASHTO T289	4.6	7.1
Soil Resistivity – AASHTO T-288	4200 Ohm-cm	1500 Ohm-cm

Should there be any questions, please do not hesitate to contact our office at (703) 698-9300. PSI would be pleased to continue providing geotechnical services throughout the implementation of the project, and we look forward to working with you on this and future projects.

Respectfully submitted,
PROFESSIONAL SERVICE INDUSTRIES, INC.

Lubomir D. Peytchev

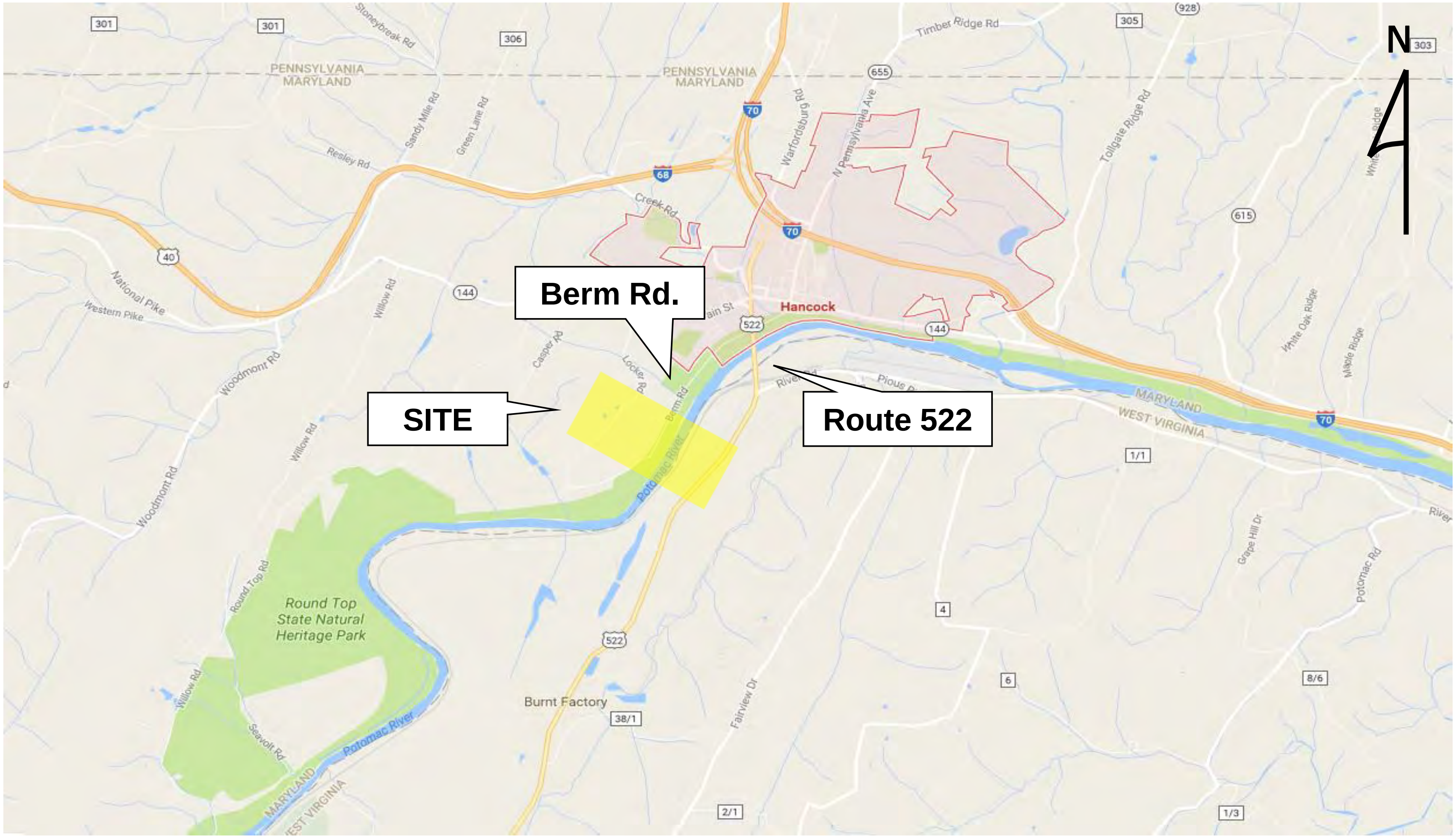
Lubomir D. Peytchev, P.E.
Senior Geotechnical Engineer

Naseer Nayeem, P.E.
Vice President/Principal Consultant

Appendix:
- Figure 1A: Site Vicinity Map and Figure 1B: Boring Location Plan
- Boring Logs and General Notes
- Cross Section Showing the General Stratigraphy
- Laboratory Test Results
- Slake Durability Test Results
- Soil Resistivity Test Results

Figure 1A Site Vicinity Map and Figure 1B – Boring Location Plan

Map Source: Google



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REVISIONS

Site Vicinity Map (Figure 1A)
6493- Eastern Panhandle Expansion Potomac River Crossing

Washington County, MD and Morgan County, WV		JULY 21, 2016
G.C.	Not Drawn To Scale	0512-713-1

LEGEND:

● B-1 - PROPOSED BORING

(10 FT) - BORING DEPTH

NOTES:
1. ALL BORINGS WILL BE ADVANCED WITH HOLLOW-STEM AUGERS.
2. SPT SAMPLING WILL BE PERFORMED IN ALL BORINGS.
3. BORING DEPTHS ARE AS SHOWN
4. BORING SPOILS WILL USED TO BACKFILL THE BORE HOLES.

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BORING LOCATION PLAN (FIGURE 1B)
6493- EASTERN PANHANDLE EXPANSION POTOMAC RIVER CROSSING

WASHINGTON COUNTY, MD AND MORGAN COUNTY, WV
JULY 21, 2016

G.C.
N.T.S.
0512-713-1

Boring Logs and General Notes

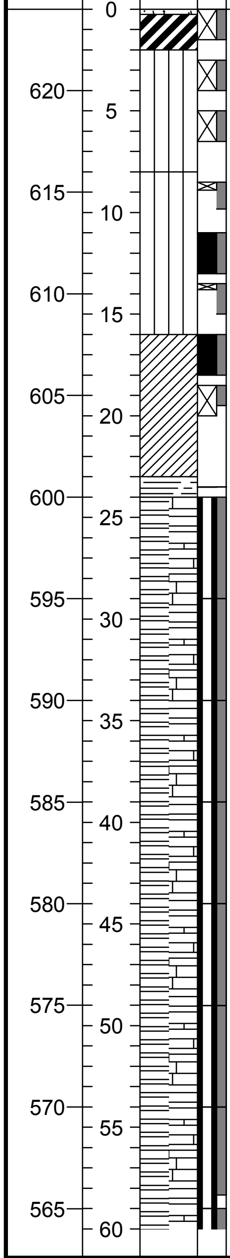
BORING GO-1

DATE STARTED: 6/29/16
DATE COMPLETED: 7/7/16
COMPLETION DEPTH 305.0 ft
BENCHMARK: N/A
ELEVATION: 624 ft
LATITUDE: 39.6803639°
LONGITUDE: 78.1952222°
STATION: N/A **OFFSET:** N/A
REMARKS:

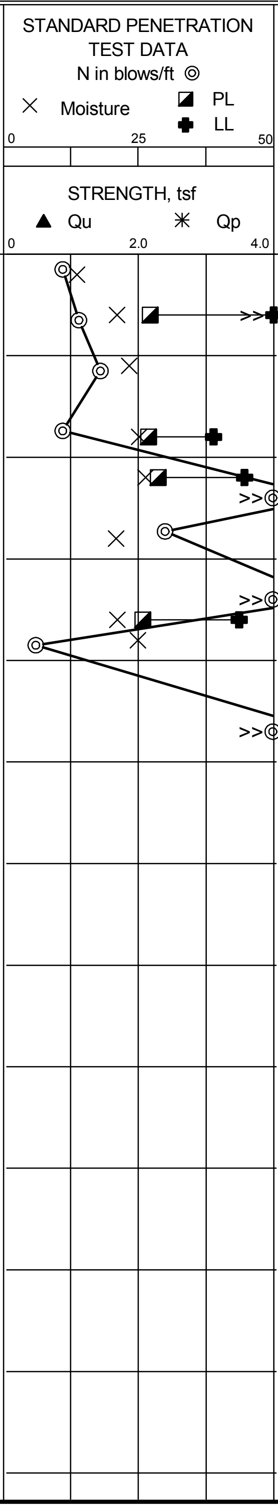
DRILL COMPANY: Connelly Drilling, Inc.
DRILLER: Tom Chew **LOGGED BY:** J. Thonnfend
DRILL RIG: CME 55 LC
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: 2-in SS1.874-in Core Standard
HAMMER TYPE: Automatic
EFFICIENCY N/A
REVIEWED BY: Lubomir Peytchev

Water		
▽ While Drilling	Dry	feet
▼ Upon Completion	Dry	feet
▽ Delay	N/A	feet

BORING LOCATION:



Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS) RQD & Recovery % (NX)	Moisture, %	Additional Remarks
1	18	Approximately 3 inches of Topsoil	Top Soil	1-6-5 N=11	14	
		Soft to stiff, moist, light brown, **FAT CLAY (USCS CH)** some gravel, roots. **Alluvium**	CH			
2	18	Stiff to very stiff, moist, brown, red, trace gray, **SILT (USCS ML)** some gravel. **Residuum**	ML	2-6-8 N=14	21	LL = 55 PL = 27 Fines=97.4%
3	18			6-8-10 N=18	23	
4	16	Stiff to very stiff, moist, dark brown, brown, **SILT (USCS ML)** some gravel, trace shale fragments **Residuum**	ML	2-5-6 N=11	25	LL = 39 PL = 27 Fines=99.8%
	24				26	LL = 45 PL = 29 Fines=99.6%
5	18			7-14-16 N=30	21	
	24	Medium stiff to stiff, very moist, dark brown, olive brown, black **lean CLAY (USCS CL)** some gravel, shale fragments, wood chips. **Residuum**	CL		21	LL = 44 PL = 26 Fines=90.0%
6	12			1-4-2 N=6	25	
7	0	Weathered Rock, gray, dark gray, soft SHALE and hard LIMESTONE	WEATHERED SHALE	50/0"		
8	60	Interbedded, slightly weathered, medium bedded to thin bedded, gray, dark gray , fine grained to medium grained, very soft to soft SHALE and hard LIMESTONE, dip of 25 to 45 degrees, (RQD from 45% to 100 %), Devonian [Marcellus Formation and Needmore Shale]	Shale and Limestone	RQD=89 Rec=100%		
9	60			RQD=90 Rec=100%		
10	60			RQD=100 Rec=100%		
11	60			RQD=78 Rec=100%		
12	60			RQD=88 Rec=100%		
13	60			RQD=90 Rec=100%		
14	52			RQD=45 Rec=87%		



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The stratification lines represent approximate boundaries. The transition may be gradual.

DATE STARTED: 6/29/16
DATE COMPLETED: 7/7/16
COMPLETION DEPTH 305.0 ft
BENCHMARK: N/A
ELEVATION: 624 ft
LATITUDE: 39.6803639°
LONGITUDE: 78.1952222°
STATION: N/A **OFFSET:** N/A
REMARKS:

DRILL COMPANY: Connelly Drilling, Inc.
DRILLER: Tom Chew **LOGGED BY:** J. Thonnfend
DRILL RIG: CME 55 LC
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: 2-in SS1.874-in Core Standard
HAMMER TYPE: Automatic
EFFICIENCY N/A
REVIEWED BY: Lubomir Peytchev

BORING GO-1

Water		
▽	While Drilling	Dry feet
▼	Upon Completion	Dry feet
⊻	Delay	N/A feet

BORING LOCATION:

Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS) RQD & Recovery % (NX)	Moisture, %	STANDARD PENETRATION TEST DATA N in blows/ft ◎ × Moisture ◪ PL ✚ LL 0 25 50 STRENGTH, tsf ▲ Qu ✳ Qp 0 2.0 4.0	Additional Remarks
	60					Interbedded, slightly weathered, medium bedded to thin bedded, gray, dark gray , fine grained to medium grained, very soft to soft SHALE and hard LIMESTONE, dip of 25 to 45 degrees, (RQD from 45% to 100 %), Devonian [Marcellus Formation and Needmore Shale]					
				15	60			RQD=83 Rec=100%			
560	65										
				16	60			RQD=95 Rec=100%			
555	70						Shale and Limestone				
				17	60			RQD=92 Rec=100%			
550	75										
				18	60			RQD=97 Rec=100%			
545	80					Slightly weathered, medium bedded to thin bedded, black, dark gray, trace white , fine grained to medium grained, soft SHALE, trace coal seams, dip of 20 to 60 degrees, (RQD from 47% to 95 %), Devonian [Marcellus Formation and Needmore Shale]					
				19	60			RQD=94 Rec=100%			
540	85										
				20	60			RQD=95 Rec=100%			
535	90										
				21	60			RQD=95 Rec=100%			
530	95										
				22	60			RQD=88 Rec=100%			
525	100						Shale				
				23	60			RQD=67 Rec=100%			
520	105										
				24	60			RQD=50 Rec=100%			
515	110										
				25	60			RQD=47 Rec=100%			
510	115										
				26	60			RQD=64 Rec=100%			
505	120										

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The stratification lines represent approximate boundaries. The transition may be gradual.

DATE STARTED: 6/29/16
DATE COMPLETED: 7/7/16
COMPLETION DEPTH 305.0 ft
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BORING GO-1

Water		
▽ While Drilling	Dry	feet
▼ Upon Completion	Dry	feet
▽ Delay	N/A	feet

BORING LOCATION:

Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS) RQD & Recovery % (NX)	Moisture, %	STANDARD PENETRATION TEST DATA N in blows/ft ◎ × Moisture ◪ PL ✚ LL 0 25 50 STRENGTH, tsf ▲ Qu ✳ Qp 0 2.0 4.0	Additional Remarks
500	120			27	60	Slightly weathered, medium bedded to thin bedded, black, dark gray, trace white , fine grained to medium grained, soft SHALE, trace coal seams, dip of 20 to 60 degrees, (RQD from 47% to 95 %), Devonian [Marcellus Formation and Needmore Shale]	Shale	RQD=53 Rec=100%			
495	125			28	60			RQD=59 Rec=100%			
490	130			29	60	Interbedded, slightly weathered, medium bedded to thin bedded, gray, dark gray , fine grained to medium grained, soft SHALE and hard LIMESTONE, dip of 25 to 50 degrees, (RQD from 55% to 100 %), Devonian [Marcellus Formation and Needmore Shale]	Shale and Limestone	RQD=100 Rec=100%			
485	135			30	60			RQD=96 Rec=100%			
480	140			31	60			RQD=93 Rec=100%			
475	145			32	58			RQD=95 Rec=97%			
470	150			33	60			RQD=98 Rec=100%			
465	155			34	60			RQD=91 Rec=100%			
460	160			35	59			RQD=96 Rec=98%			
455	165			36	60			RQD=100 Rec=100%			
450	170			37	60			RQD=73 Rec=100%			
445	175			38	54			RQD=75 Rec=90%			
	180										

Continued Next Page

PROJECT NO.: 0512713-1
PROJECT: 6493-Eastern Panhandle Expansion
LOCATION: Potomac River Crossing
Washington County
Hancock, MD

The stratification lines represent approximate boundaries. The transition may be gradual.

DATE STARTED: 6/29/16
DATE COMPLETED: 7/7/16
COMPLETION DEPTH 305.0 ft
BENCHMARK: N/A
ELEVATION: 624 ft
LATITUDE: 39.6803639°
LONGITUDE: 78.1952222°
STATION: N/A **OFFSET:** N/A
REMARKS:

DRILL COMPANY: Connelly Drilling, Inc.
DRILLER: Tom Chew **LOGGED BY:** J. Thonnfend
DRILL RIG: CME 55 LC
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: 2-in SS1.874-in Core Standard
HAMMER TYPE: Automatic
EFFICIENCY N/A
REVIEWED BY: Lubomir Peytchev

BORING GO-1

Water		
▽ While Drilling	Dry	feet
▼ Upon Completion	Dry	feet
▽ Delay	N/A	feet

BORING LOCATION:

Elevation (feet)	Depth, (feet)	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS) RQD & Recovery % (NX)	Moisture, %	STANDARD PENETRATION TEST DATA N in blows/ft ◎ × Moisture ◪ PL ✚ LL 0 25 50 / STRENGTH, tsf ▲ Qu ✳ Qp 0 2.0 4.0	Additional Remarks
440	180	39	60	Interbedded, slightly weathered, medium bedded to thin bedded, gray, dark gray , fine grained to medium grained, soft SHALE and hard LIMESTONE, dip of 25 to 50 degrees, (RQD from 55% to 100 %), Devonian [Marcellus Formation and Needmore Shale]	Shale and Limestone	RQD=75 Rec=100%			
435	185	40	60			RQD=83 Rec=100%			
430	190	41	60			RQD=55 Rec=100%			
425	195	42	60			RQD=63 Rec=100%			
420	200	43	60	Interbedded, slightly weathered, medium bedded to thin bedded, red, fine grained, soft SHALE and hard LIMESTONE, dip of 25 to 45 degrees, (RQD from 88% to 93 %), Devonian [Marcellus Formation and Needmore Shale]	Shale and Limestone	RQD=88 Rec=100%			
415	205	44	60			RQD=88 Rec=100%			
410	210	45	60			RQD=93 Rec=100%			
405	215	46	60			RQD=88 Rec=100%			
400	220	47	60			RQD=93 Rec=100%			
395	225	48	56	Interbedded, slightly weathered, medium bedded to thin bedded, white, gray, trace red, yellow, fine grained to medium grained, soft SANDSTONE and soft SHALE, dip of 10 to 25 degrees, (RQD from 46% to 75 %), Devonian [Marcellus Formation and Needmore Shale]	SANDSTONE	RQD=51 Rec=93%			
390	230	49	60			RQD=70 Rec=100%			
385	235	50	60			RQD=65 Rec=100%			
	240								

Continued Next Page

PROJECT NO.: 0512713-1
PROJECT: 6493-Eastern Panhandle Expansion
LOCATION: Potomac River Crossing
Washington County
Hancock, MD

The stratification lines represent approximate boundaries. The transition may be gradual.

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DATE COMPLETED: 7/7/16
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LONGITUDE: 78.1952222°
STATION: N/A **OFFSET:** N/A
REMARKS:

DRILL COMPANY: Connelly Drilling, Inc.
DRILLER: Tom Chew **LOGGED BY:** J. Thonnfend
DRILL RIG: CME 55 LC
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: 2-in SS1.874-in Core Standard
HAMMER TYPE: Automatic
EFFICIENCY N/A
REVIEWED BY: Lubomir Peytchev

BORING GO-1

Water		
▽	While Drilling	Dry feet
▼	Upon Completion	Dry feet
▼	Delay	N/A feet

BORING LOCATION:

Elevation (feet)	Depth, (feet)	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS) RQD & Recovery % (NX)	Moisture, %	STANDARD PENETRATION TEST DATA N in blows/ft ◎ × Moisture ◪ PL ✚ LL 0 25 50 STRENGTH, tsf ▲ Qu ✳ Qp 0 2.0 4.0	Additional Remarks
380	240	51	60	Interbedded, slightly weathered, medium bedded to thin bedded, white, gray, trace red, yellow, fine grained to medium grained, soft SANDSTONE and soft SHALE, dip of 10 to 25 degrees, (RQD from 46% to 75 %), Devonian [Marcellus Formation and Needmore Shale]	SANDSTONE	RQD=75 Rec=100%			
375	245	52	58			RQD=68 Rec=97%			
370	250	53	57			RQD=85 Rec=95%			
365	255	54	57			RQD=83 Rec=95%			
360	260	55	60			RQD=46 Rec=100%			
355	265	56	26	Loose wet brown poorly-graded **SAND (USCS SP)** trace limestone floaters, Devonian [Marcellus Formation and Needmore Shale]	SP	RQD=0 Rec=43%			
350	270	57	8			RQD=0 Rec=13%			
345	275	58	52	Interbedded, slightly weathered, medium bedded to thin bedded, white, gray, fine grained to medium grained, medium hard SANDSTONE and sand seams, dip of 10 to 35 degrees, (RQD from 13% to 30 %), Devonian [Marcellus Formation and Needmore Shale]	SANDSTONE	RQD=13 Rec=87%			
340	280	59	52			RQD=30 Rec=87%			
335	285	60	49			RQD=25 Rec=82%			
330	290	61	60	Interbedded, slightly weathered, medium bedded to thin bedded, white, gray, fine grained, medium hard LIMESTONE - CHERTY and sand seams, dip of 45 degrees, (RQD from 58% to 100 %), Devonian [Marcellus Formation and Needmore Shale]	LIMESTONE - CHERTY	RQD=58 Rec=100%			
325	295	62	60			RQD=62 Rec=100%			
	300								

Continued Next Page

Professional Service Industries, Inc.
2930 Eskridge Rd
Fairfax, VA 22031
Telephone: (703) 698-9300

PROJECT NO.: 0512713-1
PROJECT: 6493-Eastern Panhandle Expansion
LOCATION: Potomac River Crossing
Washington County
Hancock, MD

The stratification lines represent approximate boundaries. The transition may be gradual.

DATE STARTED: 6/29/16
DATE COMPLETED: 7/7/16
COMPLETION DEPTH 305.0 ft
BENCHMARK: N/A
ELEVATION: 624 ft
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STATION: N/A **OFFSET:** N/A
REMARKS:

DRILL COMPANY: Connelly Drilling, Inc.
DRILLER: Tom Chew **LOGGED BY:** J. Thonnfend
DRILL RIG: CME 55 LC
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: 2-in SS1.874-in Core Standard
HAMMER TYPE: Automatic
EFFICIENCY N/A
REVIEWED BY: Lubomir Peytchev

BORING GO-1

Water			
	▽	While Drilling	Dry feet
	▼	Upon Completion	Dry feet
	▽	Delay	N/A feet

BORING LOCATION:

Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS) RQD & Recovery % (NX)	Moisture, %	STANDARD PENETRATION TEST DATA N in blows/ft ◎ × Moisture ◪ PL ✚ LL 0 25 50 STRENGTH, tsf ▲ Qu ✳ Qp 0 2.0 4.0	Additional Remarks
	300										
				63	60		LIMESTONE - CHERTY	RQD=78 Rec=100%			
320	305			64	12			RQD=100 Rec=100%			
						Bottom of test boring at 305 feet					

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PROJECT NO.: 0512713-1
PROJECT: 6493-Eastern Panhandle Expansion
LOCATION: Potomac River Crossing
Washington County
Hancock, MD

The stratification lines represent approximate boundaries. The transition may be gradual.

DATE STARTED: 7/7/16
DATE COMPLETED: 7/11/16
COMPLETION DEPTH 154.0 ft
BENCHMARK: N/A
ELEVATION: 411 ft
LATITUDE: 39.683732°
LONGITUDE: 78.199167°
STATION: N/A **OFFSET:** N/A
REMARKS:

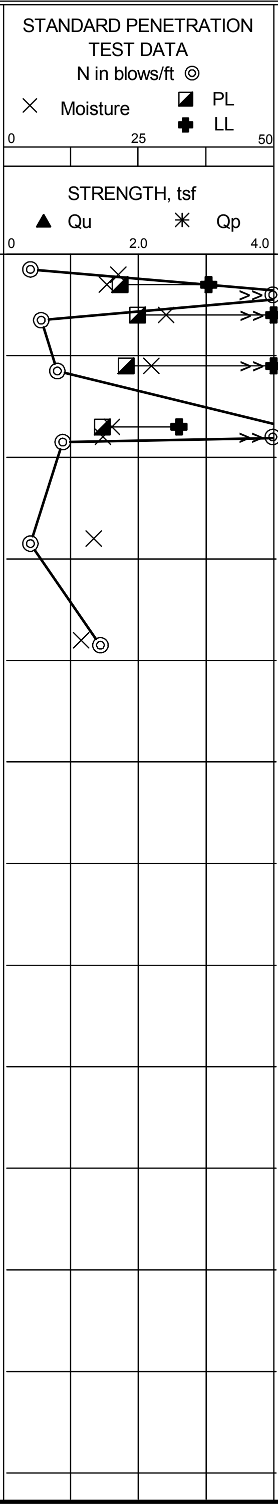
DRILL COMPANY: Connelly Drilling, Inc.
DRILLER: Tom Chew **LOGGED BY:** J. Thonnfend
DRILL RIG: CME 55 LC
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: 2-in SS1.874-in Core Standard
HAMMER TYPE: Automatic
EFFICIENCY N/A
REVIEWED BY: Lubomir Peytchev

BORING GO-2R

Water		
▽	While Drilling	Dry feet
▼	Upon Completion	Dry feet
▼	Delay	N/A feet

BORING LOCATION:

Elevation (feet)	Depth, (feet)	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS) RQD & Recovery % (NX)	Moisture, %	Additional Remarks
410	0	1	16 24	Approximately 3 inches of Topsoil	Top Soil	2-2-3 N=5	21 19	LL = 38 PL = 22 Fines=75.1%
				Medium stiff, moist, brown, dark brown, **lean CLAY (USCS CL)** some gravel, roots. **Alluvium**	CL			
		2	18			3-3-4 N=7	30	LL = 63 PL = 25
405	5	3	18	Medium stiff to stiff, moist, mottled brown **FAT CLAY (USCS CH)** some sand. **Alluvium**	CH	3-4-6 N=10	27	LL = 59 PL = 23 Fines=89.3%
400	10	4	24 18	Stiff, moist, mottled brown, **lean CLAY (USCS CL)** trace sand. **Alluvium**	CL	2-3-8 N=11	20 18	LL = 33 PL = 18 Fines=90.8%
395	15	5	14	Medium stiff, moist, mottled brown **sandy lean CLAY (USCS CL)** trace gravel. **Alluvium**	CL	2-2-3 N=5	17	Fines=55.8%
390	20	6	12	Stiff, moist, mottled brown, **lean CLAY (USCS CL)** trace shale fragments, sand, gravel. **Residuum**	CL	18-11-7 N=18	14	
				Weathered Rock, gray, dark gray, soft SHALE and hard LIMESTONE, Silurian [Wills Creek Shale and Bloomsburg Formation]	WEATHERED SHALE			
385	25	7	2	Interbedded, slightly weathered, medium bedded to thin bedded, gray, light gray, trace brown, fine grained to medium grained, medium hard SHALE and hard LIMESTONE, dip from 30 to 40 degrees, (RQD from 35 to 95 %) Silurian [Wills Creek Shale and Bloomsburg Formation]	Shale and Limestone	4-48-50/2"		
		8	54			RQD=35 Rec=90%		
380	30	9	60			RQD=40 Rec=100%		
375	35	10	60			RQD=95 Rec=100%		
370	40	11	60			RQD=52 Rec=100%		
365	45	12	60			RQD=82 Rec=100%		
360	50	13	53		Shale and Limestone	RQD=70 Rec=88%		
355	55	14	57			RQD=93 Rec=95%		
	60							



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PROJECT NO.: 0512713-1
PROJECT: 6493-Eastern Panhandle Expansion
LOCATION: Potomac River Crossing
Washington County
Hancock, MD

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DRILL RIG: CME 55 LC
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: 2-in SS1.874-in Core Standard
HAMMER TYPE: Automatic
EFFICIENCY N/A
REVIEWED BY: Lubomir Peytchev

BORING GO-2R

Water		
∇ While Drilling	Dry	feet
▼ Upon Completion	Dry	feet
▼ Delay	N/A	feet

BORING LOCATION:

Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS) RQD & Recovery % (NX)	Moisture, %	STANDARD PENETRATION TEST DATA N in blows/ft ◎ × Moisture ◪ PL ✚ LL 0 25 50 STRENGTH, tsf ▲ Qu ✳ Qp 0 2.0 4.0	Additional Remarks
350	60			15	60	Interbedded, slightly weathered, medium bedded to thin bedded, brown and gray, fine grained to medium grained, medium hard SHALE and hard LIMESTONE, dip from 30 to 40 degrees, (RQD from 42 to 93 %) Silurian [Wills Creek Shale and Bloomsburg Formation]	Shale and Limestone	RQD=78 Rec=100%			
345	65			16	57			RQD=52 Rec=95%			
340	70			17	60	Void from 51.3' to 51.4' Void from 53.1' to 53.2'		RQD=42 Rec=100%			
335	75			18	60	Void from 53.4' to 54' Interbedded, slightly weathered, medium bedded to thin bedded, gray, trace brown, fine grained to medium grained, medium hard SHALE and hard LIMESTONE, dip from 30 to 45 degrees, (RQD from 66 to 98 %) Silurian [Wills Creek Shale and Bloomsburg Formation]	Shale and Limestone	RQD=98 Rec=100%			
330	80			19	60			RQD=72 Rec=100%			
325	85			20	60			RQD=85 Rec=100%			
320	90			21	59			RQD=83 Rec=98%			
315	95			22	57			RQD=70 Rec=95%			
310	100			23	60			RQD=66 Rec=100%			
305	105			24	51	Interbedded, slightly weathered, medium bedded to thin bedded, gray, trace yellow and brown, fine grained to medium grained, medium hard SHALE and hard LIMESTONE, dip from 30 to 45 degrees, (RQD from 17 to 95 %) Silurian [Wills Creek Shale and Bloomsburg Formation]	Shale and Limestone	RQD=62 Rec=85%			
300	110			25	49	Sand seam from 131.6' to 134'		RQD=18 Rec=82%			
295	115			26	60	Sand seam from 136' to 136.7' Sand seam from 138.5' to 139'		RQD=73 Rec=100%			
	120										

Continued Next Page

PROJECT NO.: 0512713-1
PROJECT: 6493-Eastern Panhandle Expansion
LOCATION: Potomac River Crossing
Washington County
Hancock, MD

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DATE STARTED: 7/7/16
DATE COMPLETED: 7/11/16
COMPLETION DEPTH 154.0 ft
BENCHMARK: N/A
ELEVATION: 411 ft
LATITUDE: 39.683732°
LONGITUDE: 78.199167°
STATION: N/A **OFFSET:** N/A
REMARKS:

DRILL COMPANY: Connelly Drilling, Inc.
DRILLER: Tom Chew **LOGGED BY:** J. Thonnfend
DRILL RIG: CME 55 LC
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: 2-in SS1.874-in Core Standard
HAMMER TYPE: Automatic
EFFICIENCY N/A
REVIEWED BY: Lubomir Peytchev

BORING GO-2R

Water		
▽	While Drilling	Dry feet
▼	Upon Completion	Dry feet
▽	Delay	N/A feet

BORING LOCATION:

Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS) RQD & Recovery % (NX)	Moisture, %	STANDARD PENETRATION TEST DATA N in blows/ft ◎ × Moisture ◪ PL ✚ LL 0 25 50 STRENGTH, tsf ▲ Qu ✳ Qp 0 2.0 4.0	Additional Remarks
290	120			27	56	Interbedded, slightly weathered, medium bedded to thin bedded, gray, trace yellow and brown, fine grained to medium grained, medium hard SHALE and hard LIMESTONE, dip from 30 to 45 degrees, (RQD from 17 to 95 %) Silurian [Wills Creek Shale and Bloomsburg Formation]	Shale and Limestone	RQD=85 Rec=93%			
285	125			28	60			RQD=95 Rec=100%			
280	130			29	33	Sand seam from 131.6' to 134' Sand seam from 136' to 136.7'		RQD=23 Rec=55%			
275	135			30	52	Sand seam from 138.5' to 139'		RQD=58 Rec=87%			
270	140			31	60			RQD=60 Rec=100%			
265	145			32	52			RQD=17 Rec=87%			
260	150			33	60			RQD=75 Rec=100%			
						Bottom of test boring at 154 feet.					

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GENERAL NOTES

SAMPLE IDENTIFICATION

The Unified Soil Classification System (USCS), AASHTO 1988 and ASTM designations D2487 and D-2488 are used to identify the encountered materials unless otherwise noted. Coarse-grained soils are defined as having more than 50% of their dry weight retained on a #200 sieve (0.075mm); they are described as: boulders, cobbles, gravel or sand. Fine-grained soils have less than 50% of their dry weight retained on a #200 sieve; they are defined as silts or clay depending on their Atterberg Limit attributes. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size.

DRILLING AND SAMPLING SYMBOLS

SFA: Solid Flight Auger - typically 4" diameter flights, except where noted.
HSA: Hollow Stem Auger - typically 3¼" or 4¼ I.D. openings, except where noted.
M.R.: Mud Rotary - Uses a rotary head with Bentonite or Polymer Slurry
R.C.: Diamond Bit Core Sampler
H.A.: Hand Auger
P.A.: Power Auger - Handheld motorized auger

☒ SS: Split-Spoon - 1 3/8" I.D., 2" O.D., except where noted.
■ ST: Shelby Tube - 3" O.D., except where noted.
▯ RC: Rock Core
⬇ TC: Texas Cone
✋ BS: Bulk Sample
▦ PM: Pressuremeter
CPT-U: Cone Penetrometer Testing with Pore-Pressure Readings

SOIL PROPERTY SYMBOLS

N: Standard "N" penetration: Blows per foot of a 140 pound hammer falling 30 inches on a 2-inch O.D. Split-Spoon.
N_{60}: A "N" penetration value corrected to an equivalent 60% hammer energy transfer efficiency (ETR)
Q_u: Unconfined compressive strength, TSF
Q_p: Pocket penetrometer value, unconfined compressive strength, TSF
w%: Moisture/water content, %
LL: Liquid Limit, %
PL: Plastic Limit, %
PI: Plasticity Index = (LL-PL),%
DD: Dry unit weight, pcf
▼, ▽, ▼: Apparent groundwater level at time noted

RELATIVE DENSITY OF COARSE-GRAINED SOILS

Relative Density	N - Blows/foot
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	50 - 80
Extremely Dense	80+

ANGULARITY OF COARSE-GRAINED PARTICLES

Description	Criteria
Angular:	Particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular:	Particles are similar to angular description, but have rounded edges
Subrounded:	Particles have nearly plane sides, but have well-rounded corners and edges
Rounded:	Particles have smoothly curved sides and no edges

GRAIN-SIZE TERMINOLOGY

Component	Size Range
Boulders:	Over 300 mm (>12 in.)
Cobbles:	75 mm to 300 mm (3 in. to 12 in.)
Coarse-Grained Gravel:	19 mm to 75 mm (¾ in. to 3 in.)
Fine-Grained Gravel:	4.75 mm to 19 mm (No.4 to ¾ in.)
Coarse-Grained Sand:	2 mm to 4.75 mm (No.10 to No.4)
Medium-Grained Sand:	0.42 mm to 2 mm (No.40 to No.10)
Fine-Grained Sand:	0.075 mm to 0.42 mm (No. 200 to No.40)
Silt:	0.005 mm to 0.075 mm
Clay:	<0.005 mm

PARTICLE SHAPE

Description	Criteria
Flat:	Particles with width/thickness ratio > 3
Elongated:	Particles with length/width ratio > 3
Flat & Elongated:	Particles meet criteria for both flat and elongated

RELATIVE PROPORTIONS OF FINES

Descriptive Term	% Dry Weight
Trace:	< 5%
With:	5% to 12%
Modifier:	>12%

GENERAL NOTES

(Continued)

CONSISTENCY OF FINE-GRAINED SOILS

Q_U - TSF	N - Blows/foot	Consistency
0 - 0.25	0 - 2	Very Soft
0.25 - 0.50	2 - 4	Soft
0.50 - 1.00	4 - 8	Firm (Medium Stiff)
1.00 - 2.00	8 - 15	Stiff
2.00 - 4.00	15 - 30	Very Stiff
4.00 - 8.00	30 - 50	Hard
8.00+	50+	Very Hard

MOISTURE CONDITION DESCRIPTION

Description	Criteria
Dry:	Absence of moisture, dusty, dry to the touch
Moist:	Damp but no visible water
Wet:	Visible free water, usually soil is below water table

RELATIVE PROPORTIONS OF SAND AND GRAVEL

Descriptive Term	% Dry Weight
Trace:	< 15%
With:	15% to 30%
Modifier:	>30%

STRUCTURE DESCRIPTION

Description	Criteria
Stratified:	Alternating layers of varying material or color with layers at least ¼-inch (6 mm) thick
Laminated:	Alternating layers of varying material or color with layers less than ¼-inch (6 mm) thick
Fissured:	Breaks along definite planes of fracture with little resistance to fracturing
Slickensided:	Fracture planes appear polished or glossy, sometimes striated

Description	Criteria
Blocky:	Cohesive soil that can be broken down into small angular lumps which resist further breakdown
Lensed:	Inclusion of small pockets of different soils
Layer:	Inclusion greater than 3 inches thick (75 mm)
Seam:	Inclusion 1/8-inch to 3 inches (3 to 75 mm) thick extending through the sample
Parting:	Inclusion less than 1/8-inch (3 mm) thick

SCALE OF RELATIVE ROCK HARDNESS

Q_U - TSF	Consistency
2.5 - 10	Extremely Soft
10 - 50	Very Soft
50 - 250	Soft
250 - 525	Medium Hard
525 - 1,050	Moderately Hard
1,050 - 2,600	Hard
>2,600	Very Hard

ROCK BEDDING THICKNESSES

Description	Criteria
Very Thick Bedded	Greater than 3-foot (>1.0 m)
Thick Bedded	1-foot to 3-foot (0.3 m to 1.0 m)
Medium Bedded	4-inch to 1-foot (0.1 m to 0.3 m)
Thin Bedded	1¼-inch to 4-inch (30 mm to 100 mm)
Very Thin Bedded	½-inch to 1¼-inch (10 mm to 30 mm)
Thickly Laminated	1/8-inch to ½-inch (3 mm to 10 mm)
Thinly Laminated	1/8-inch or less "paper thin" (<3 mm)

ROCK VOIDS

Voids	Void Diameter
Pit	<6 mm (<0.25 in)
Vug	6 mm to 50 mm (0.25 in to 2 in)
Cavity	50 mm to 600 mm (2 in to 24 in)
Cave	>600 mm (>24 in)

GRAIN-SIZED TERMINOLOGY

(Typically Sedimentary Rock)

Component	Size Range
Very Coarse Grained	>4.76 mm
Coarse Grained	2.0 mm - 4.76 mm
Medium Grained	0.42 mm - 2.0 mm
Fine Grained	0.075 mm - 0.42 mm
Very Fine Grained	<0.075 mm

ROCK QUALITY DESCRIPTION

Rock Mass Description	RQD Value
Excellent	90 -100
Good	75 - 90
Fair	50 - 75
Poor	25 -50
Very Poor	Less than 25

DEGREE OF WEATHERING

Slightly Weathered:	Rock generally fresh, joints stained and discoloration extends into rock up to 25 mm (1 in), open joints may contain clay, core rings under hammer impact.
Weathered:	Rock mass is decomposed 50% or less, significant portions of the rock show discoloration and weathering effects, cores cannot be broken by hand or scraped by knife.
Highly Weathered:	Rock mass is more than 50% decomposed, complete discoloration of rock fabric, core may be extremely broken and gives clunk sound when struck by hammer, may be shaved with a knife.

SOIL CLASSIFICATION CHART

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	CLEAN GRAVELS	●	GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		(LITTLE OR NO FINES)	○	GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES	○	GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)	▨	GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	SAND AND SANDY SOILS	CLEAN SANDS	⁘	SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		(LITTLE OR NO FINES)	░	SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
	MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	SANDS WITH FINES	░	SM	SILTY SANDS, SAND - SILT MIXTURES
		(APPRECIABLE AMOUNT OF FINES)	▨	SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS	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, SILTY CLAYS, LEAN CLAYS
			☰	OL	ORGANIC SILTS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50	▥	MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
			▧	CH	INORGANIC CLAYS OF HIGH PLASTICITY
			≋	OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			⁂	PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

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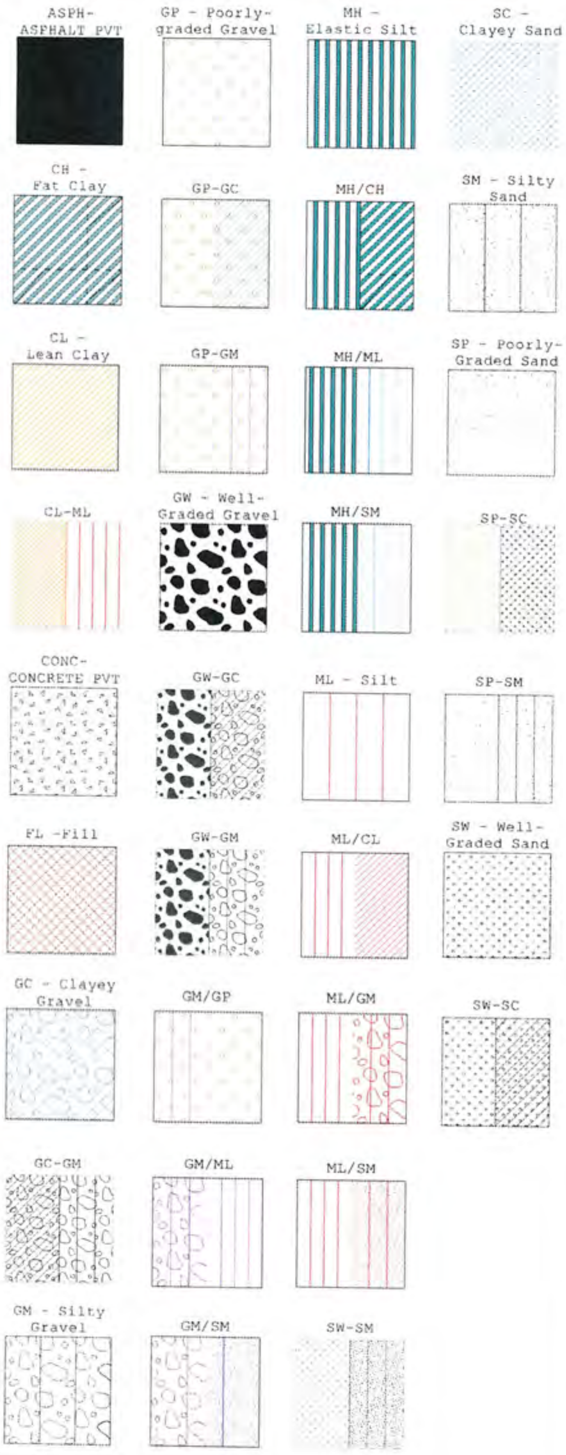
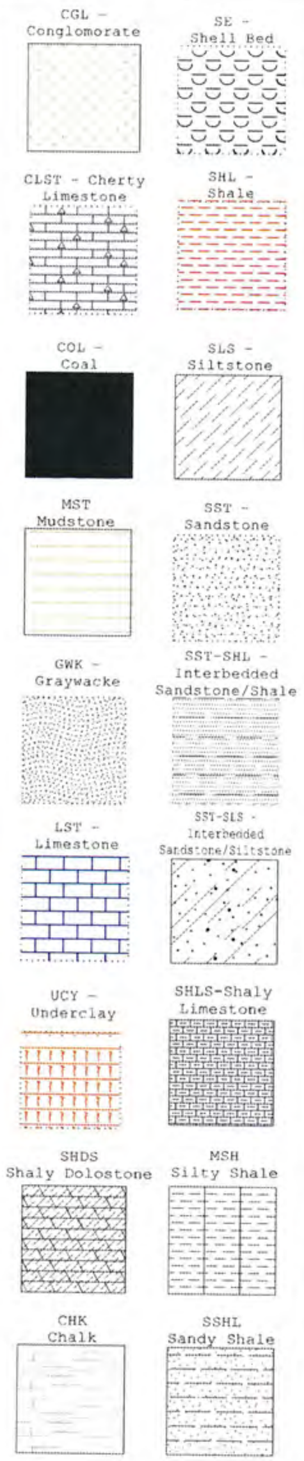
MATERIAL AND SAMPLE SYMBOLS LIST

Pavement/Soils

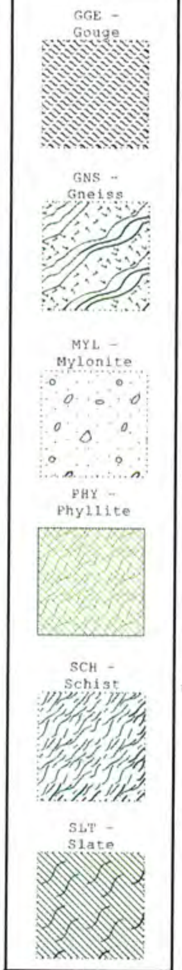


Sedimentary Rocks



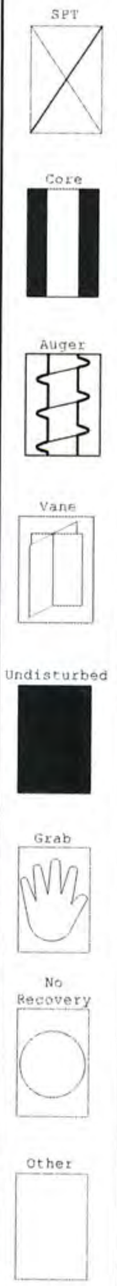
Igneous Rocks

Metamorphic Rocks



Misc.

Sampling

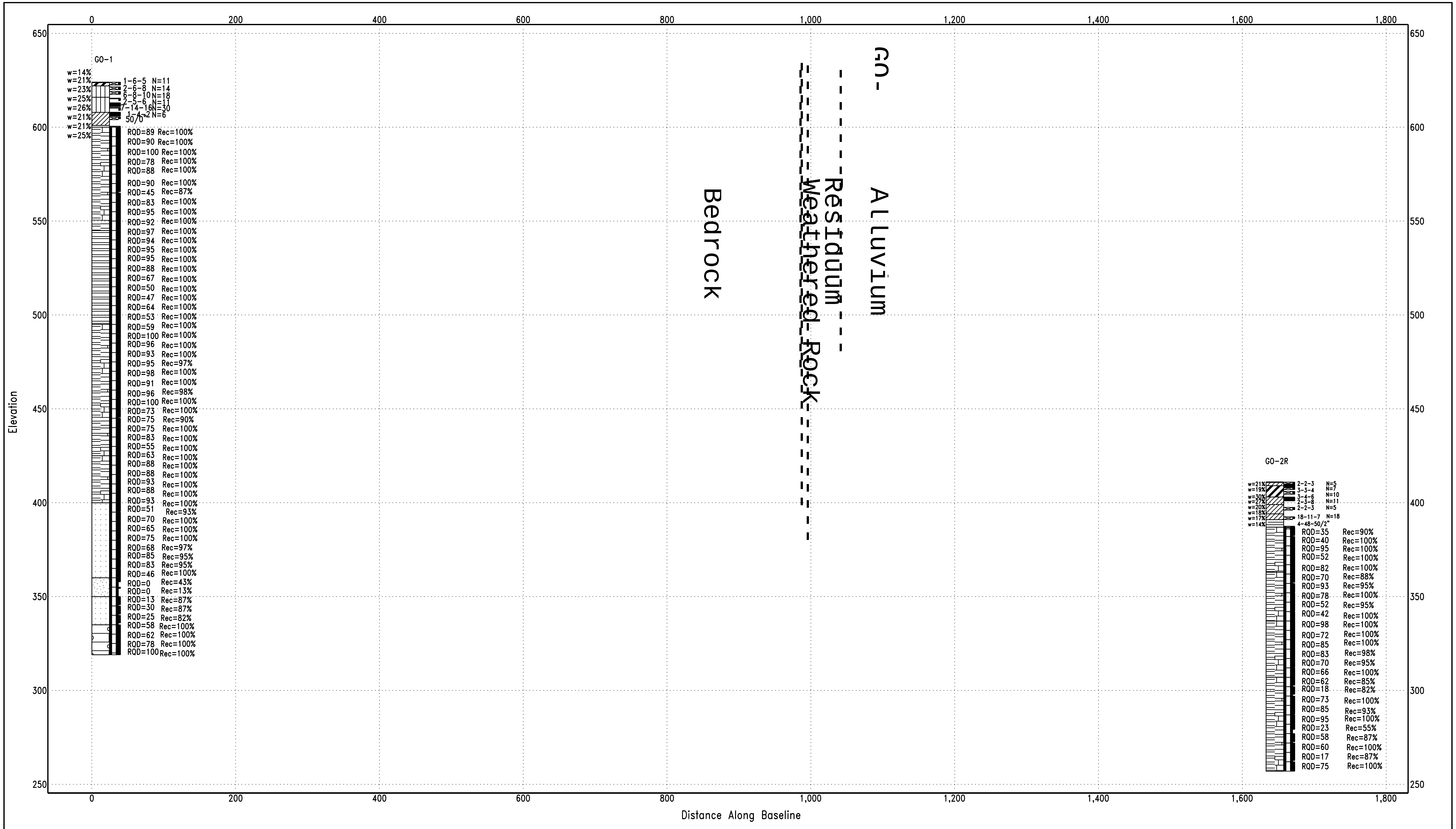


MATERIAL AND SAMPLE SYMBOLS LIST

Pavement/Soils	Sedimentary Rocks	Igneous Rocks	Metamorphic Rocks	Sampling
▨ TOPS-TOPSOIL	▨ BLD-Boulder Bed	▨ CHT Charnocktite	▨ MSLS Metasiltstone	
▨ SC/CH	▨ DLS Dolostone		▨ MSST Metasandstone	
▨ CH/CL	▨ LST-DLS-Interbedded Limestone/Dolostone		▨ QZT - Quartzite	
▨ CH/MH			▨ SPS Soapstone	
▨ CH/SC			▨ MBST Metabasalt	
▨ CL/ML			▨ MBL Marble	
▨ CL/SC				
▨ CL/CH				
▨ GP/GW				
▨ CRA Crushed Aggregate				
▨ GW/GP				
▨ ML/MH				
▨ GC/SC				
▨ OH/OL				
▨ GP/SP				
▨ OL/OH				
▨ PT Peat				
▨ OH Organic				
▨ SC/CL				
▨ OL Organic				
▨ SC/GC				
▨ SC-SM				
▨ SP/SW				
▨ SM/GM				
▨ SM/MH				
▨ SM/ML				
▨ SM/SC				
▨ SP/GP				
▨ SW/SP				

Cross Section Showing the General Stratigraphy

EnSite USA Project Number 6493
Eastern Panhandle Expansion, Potomac River
Crossing PSI Project Number 0512713-1
Revised September 9, 2016



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Profile

Eastern Panhandle Expansion
PSI Project Number: 0512713-1

Washington County
Hancock
MD

Bed

A

R

Laboratory Test Results

Laboratory Summary Sheet

Sheet 1 of 1

Borehole	Approx. Depth	Liquid Limit	Plastic Limit	Plasticity Index	Qu (tsf)	%<#200 Sieve	Est. Specific Gravity	Water Content (%)	Dry Density (pcf)	Satur-ation (%)	Void Ratio
GO-1	1							14			
GO-1	3	55	27	28		97.4%		21			
GO-1	5.5							23			
GO-1	9	39	27	12		99.8%		25			
GO-1	11	45	29	16		99.6%		26			
GO-1	14							21			
GO-1	18	44	26	18		90.0%		21			
GO-1	19							25			
GO-2R	1							21			
GO-2R	1.5	38	22	16		75.1%		19			
GO-2R	3	63	25	38				30			
GO-2R	5.5	59	23	36		89.3%		27			
GO-2R	8.5	33	18	15		90.8%		20			
GO-2R	9							18			
GO-2R	14					55.8%		17			
GO-2R	19							14			

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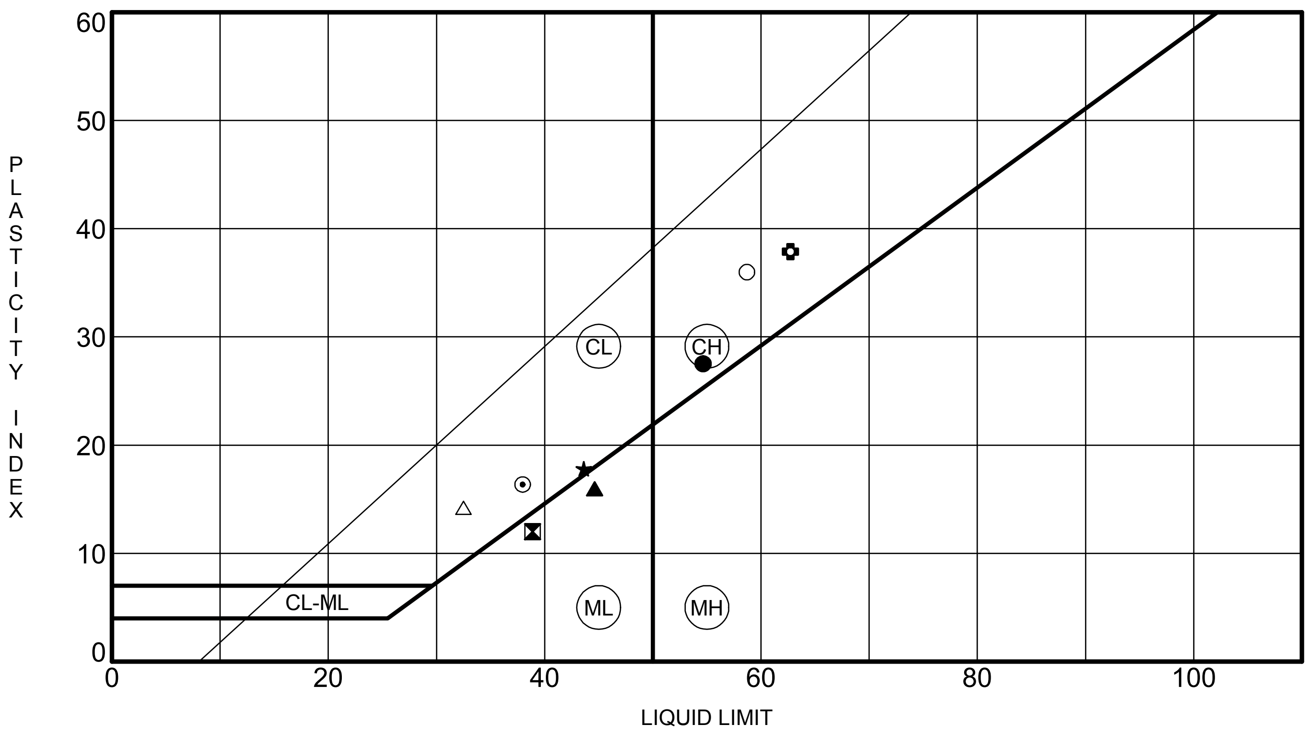
Summary of Laboratory Results

PSI Job No.: 0512713-1
Project: 6493-Eastern Panhandle Expansion
Location: Potomac River Crossing
Washington County
Hancock, MD



	Boring	Depth (ft)	LL	PL	PI	Fines	Classification (*Visual)
●	**GO-1**	**3.0**	**55**	**27**	**28**	**97.4**	
⧗	**GO-1**	**9.0**	**39**	**27**	**12**	**99.8**	
▲	**GO-1**	**11.0**	**45**	**29**	**16**	**99.6**	
★	**GO-1**	**18.0**	**44**	**26**	**18**	**90.0**	
⊙	**GO-2R**	**1.5**	**38**	**22**	**16**	**75.1**	
✿	**GO-2R**	**3.0**	**63**	**25**	**38**		
○	**GO-2R**	**5.5**	**59**	**23**	**36**	**89.3**	
△	**GO-2R**	**8.5**	**33**	**18**	**15**	**90.8**	

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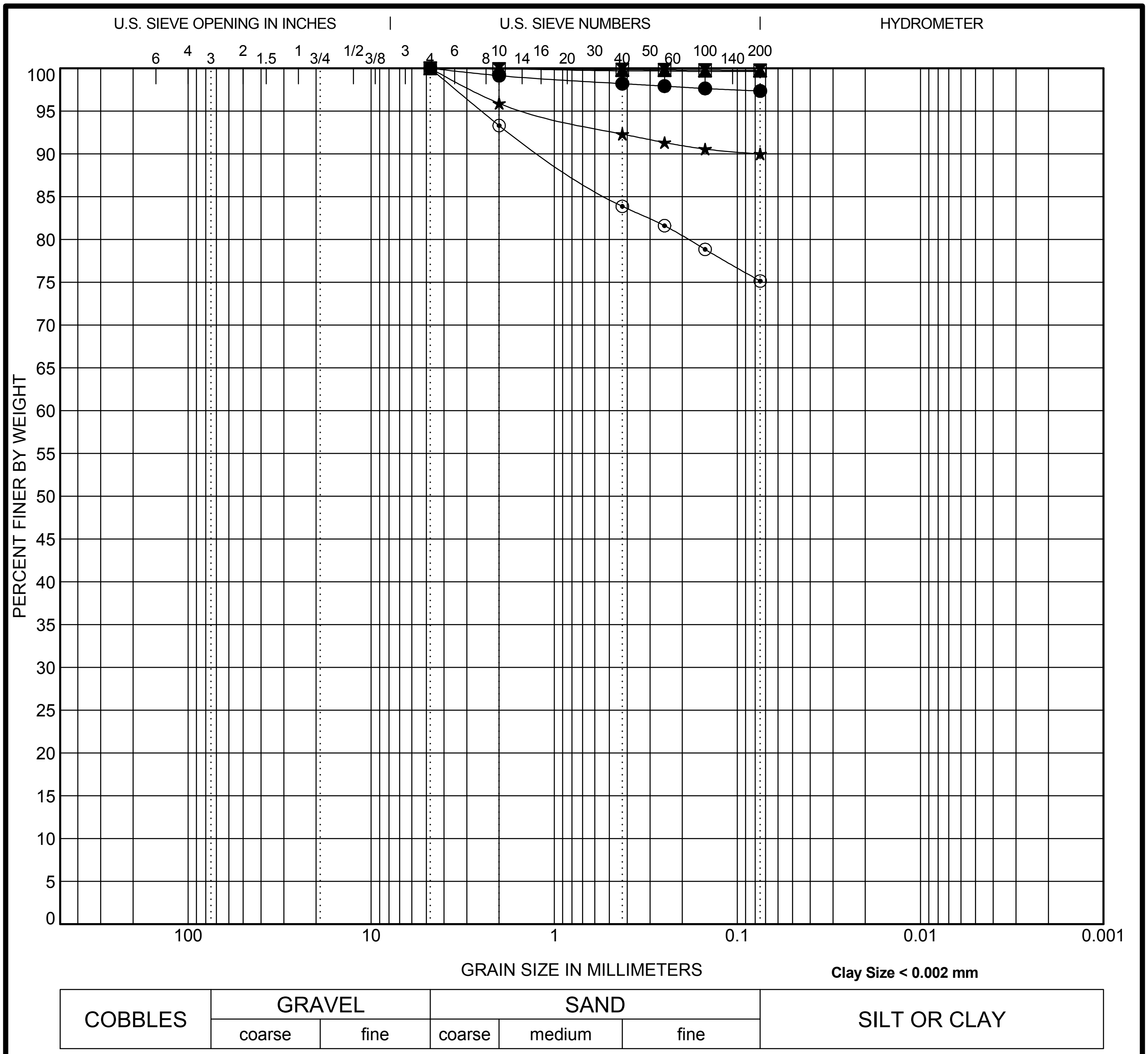
ATTERBERG LIMIT RESULTS

PSI Job No.: 0512713-1
Project: 6493-Eastern Panhandle Expansion
Location: Potomac River Crossing
Washington County



	Specimen Identification		Classification	LL	PL	PI	Cc	Cu
●	**GO-1**	**3.0**		**55**	**27**	**28**		
⧗	**GO-1**	**9.0**		**39**	**27**	**12**		
▲	**GO-1**	**11.0**		**45**	**29**	**16**		
★	**GO-1**	**18.0**		**44**	**26**	**18**		
⊙	**GO-2R**	**1.5**		**38**	**22**	**16**		

	Specimen Identification		D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**GO-1**	**3.0**	**4.75**				**0.0**	**2.6**	**97.4**	
⧗	**GO-1**	**9.0**	**4.75**				**0.0**	**0.2**	**99.8**	
▲	**GO-1**	**11.0**	**2**				**0.0**	**0.4**	**99.6**	
★	**GO-1**	**18.0**	**4.75**				**0.0**	**10.0**	**90.0**	
⊙	**GO-2R**	**1.5**	**4.75**				**0.0**	**24.9**	**75.1**	

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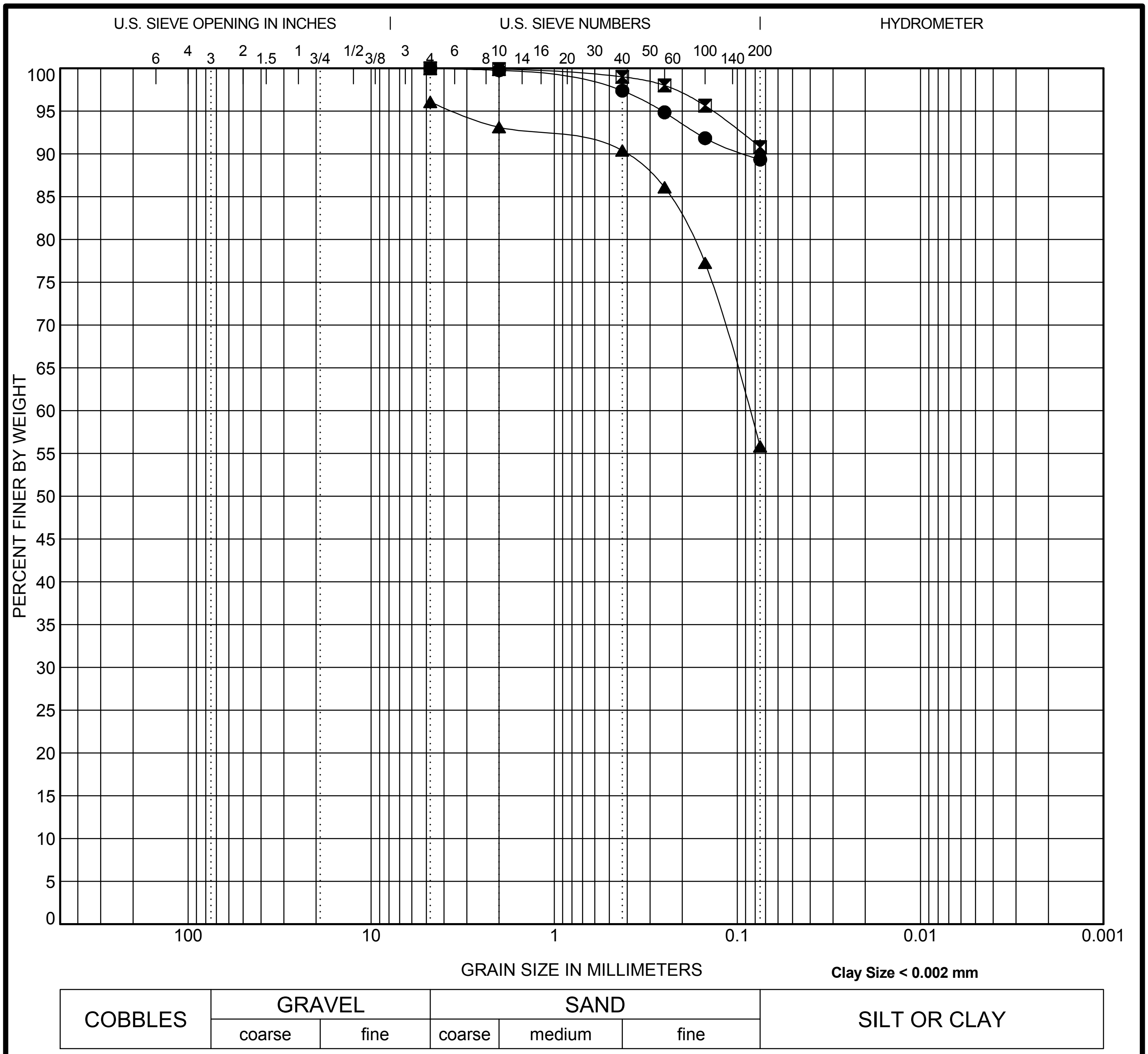
GRAIN SIZE DISTRIBUTION

Project: 6493-Eastern Panhandle Expansion
PSI Job No.: 0512713-1
Location: Potomac River Crossing
Washington County



Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	**GO-2R**	**5.5**		**59**	**23**	**36**		
☒	**GO-2R**	**8.5**		**33**	**18**	**15**		
▲	**GO-2R**	**14.0**						

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**GO-2R**	**5.5**	**4.75**				**0.0**	**10.7**	**89.3**	
☒	**GO-2R**	**8.5**	**4.75**				**0.0**	**9.2**	**90.8**	
▲	**GO-2R**	**14.0**	**4.75**	**0.086**			**0.0**	**40.2**	**55.8**	

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GRAIN SIZE DISTRIBUTION

Project:	6493-Eastern Panhandle Expansion
PSI Job No.:	0512713-1
Location:	Potomac River Crossing Washington County

UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOIL - ASTM D 2166

Sheet 3 of 3

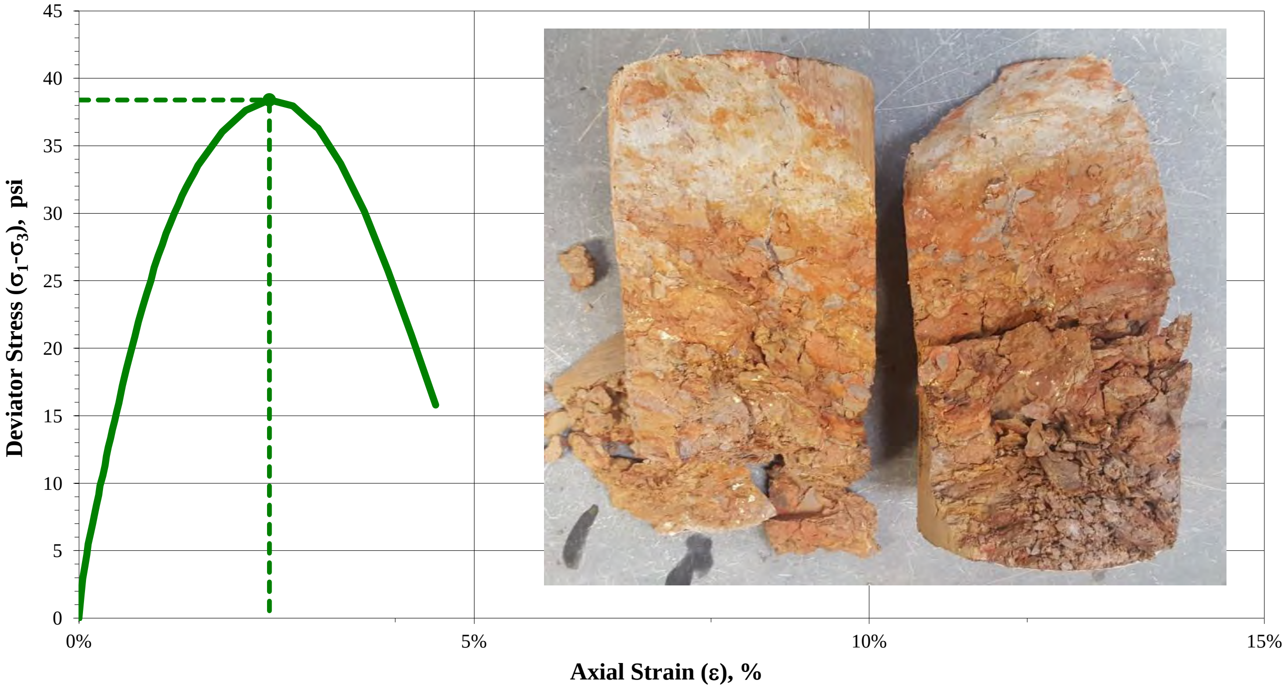
Client	Professional Service Ind	Boring	GO-1
Client No.	Proposed Eastern Panhar	Depth	4.0'-6.0'
Project No.	37622	Sample	ST-1
		Lab ID No.	37622001

Confining Stress (σ_3), psi	**0.0**
UC Strength (q_u), psi	**38.4**
Shear Strength (S_u), psi	**19.2**
Strain at Failure (ε_f), %	**2.4%**

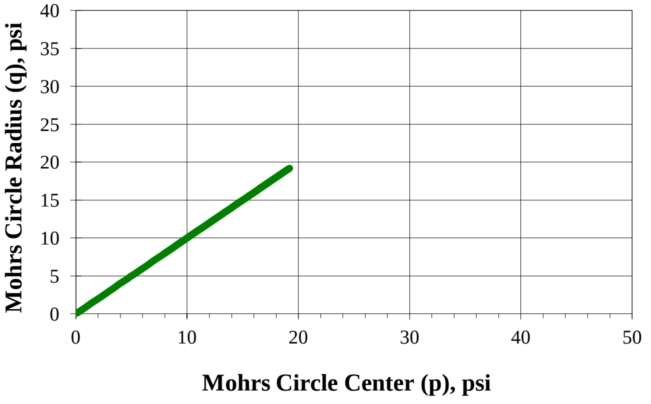
Visual Description: Brown Clay with Shale

Sample Condition Undisturbed

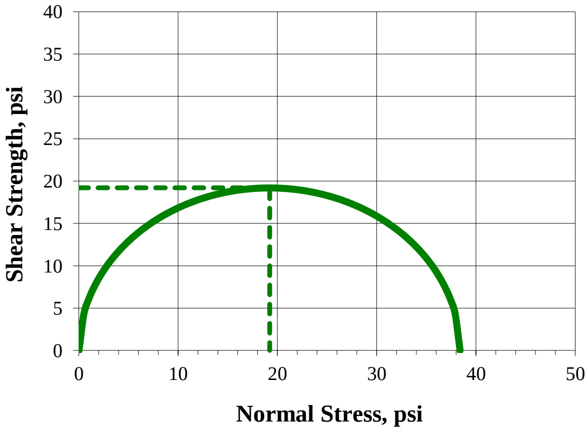
STRESS VS. STRAIN



STRESS PATHS



MOHRS CIRCLE



UNCONFINED COMPRESSIVE STRENGTH OF COHESIVE SOIL - ASTM D 2166

Sheet 3 of 3

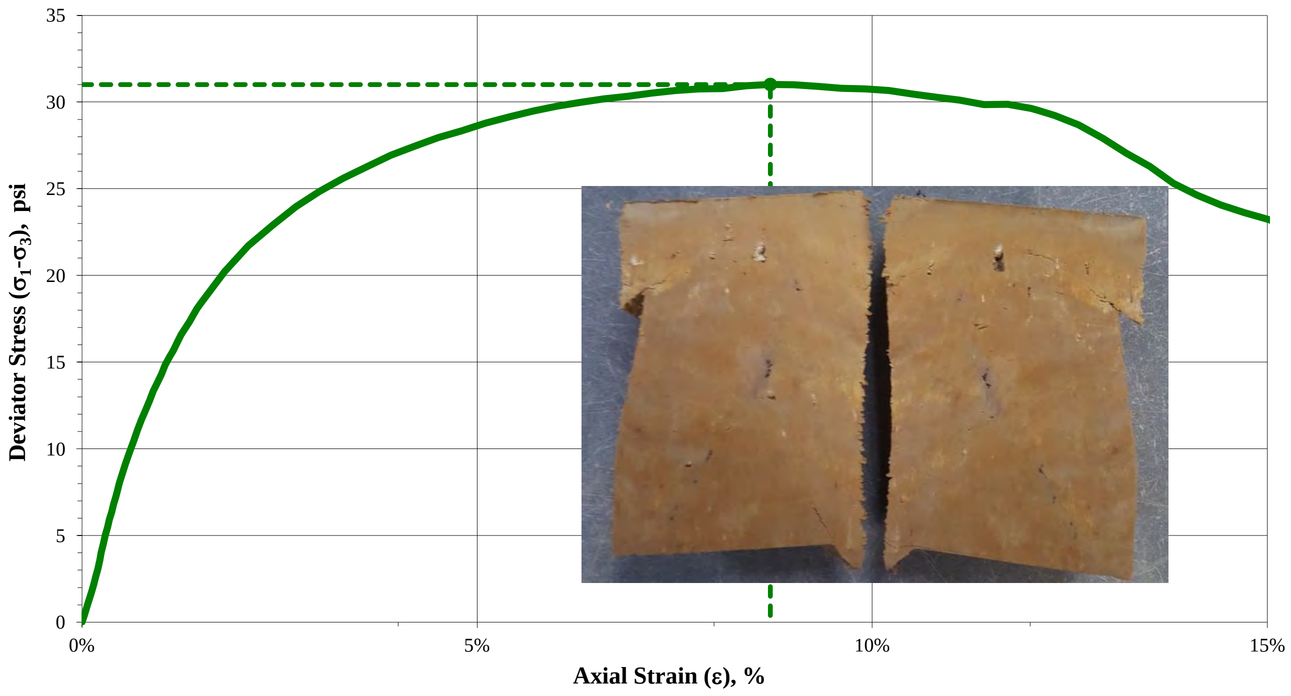
Client	Professional Service Indu	Boring	GO-2R
Client No.	Proposed Eastern Panhar	Depth	4.0'-6.0'
Project No.	37622	Sample	ST-2
		Lab ID No.	37622002

Confining Stress (σ_3), psi	**0.0**
UC Strength (q_u), psi	**31.0**
Shear Strength (S_u), psi	**15.5**
Strain at Failure (ε_f), %	**8.7%**

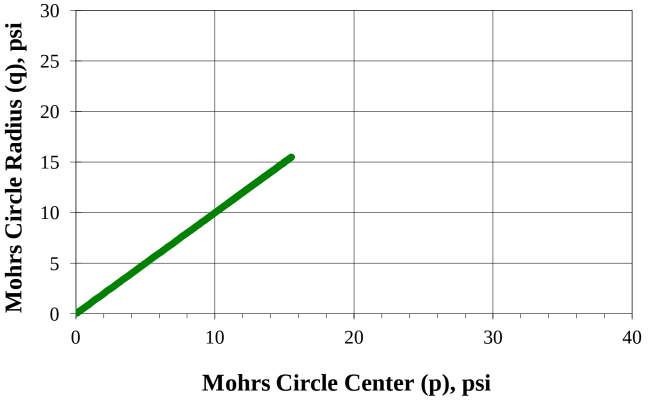
Visual Description: Brown Clay

Sample Condition Undisturbed

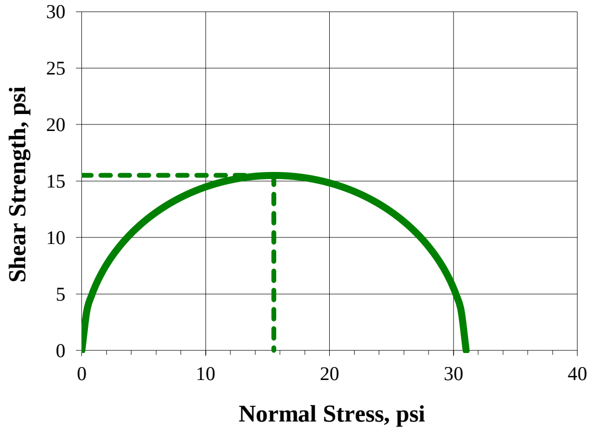
STRESS VS. STRAIN



STRESS PATHS



MOHRS CIRCLE



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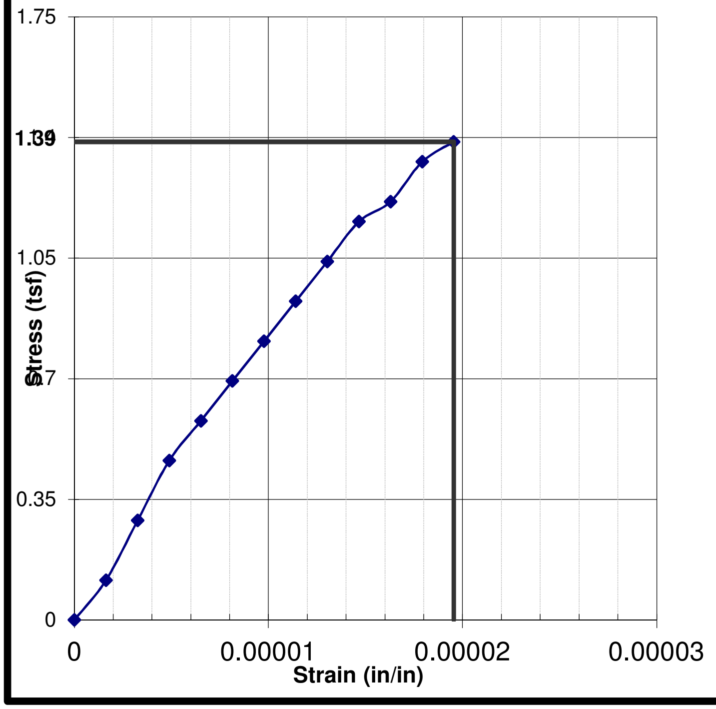
Unconfined Compression Test

Date:	7/18/2016	Project:	EnsiteUSA-CPG Pipeline, Washington County, MD
Tested by:	Redha K Hasan	Project No:	0512713-1
Client:	CPG	Location:	Hancock, MD

Average Initial Height (in):	6.14	Boring :	GO1
Average Initial Diameter (in):	2.82	Sample Number:	ST-2
Water Content %:	26.4	Sample Depth:	11.0'-11.5'
Wet Density (pcf):	119.4		
Dry Density (pcf):	94.5	Soil Description:	ML
LL - PL = PI:	45 - 26 = 16		

Stress Vs. Strain



Failure Picture

Unconfined Compression Strength q_u (tsf):	**1.39**
Height to Diameter Ratio:	**2.2**
Percent Strain at Failure :	**0.00%**
Average Rate of Strain to Failure (% Strain/min):	**0.00%**

This test was performed according to ASTM D2166 .
Unconfined Compressive Strength of Cohesive Soil.

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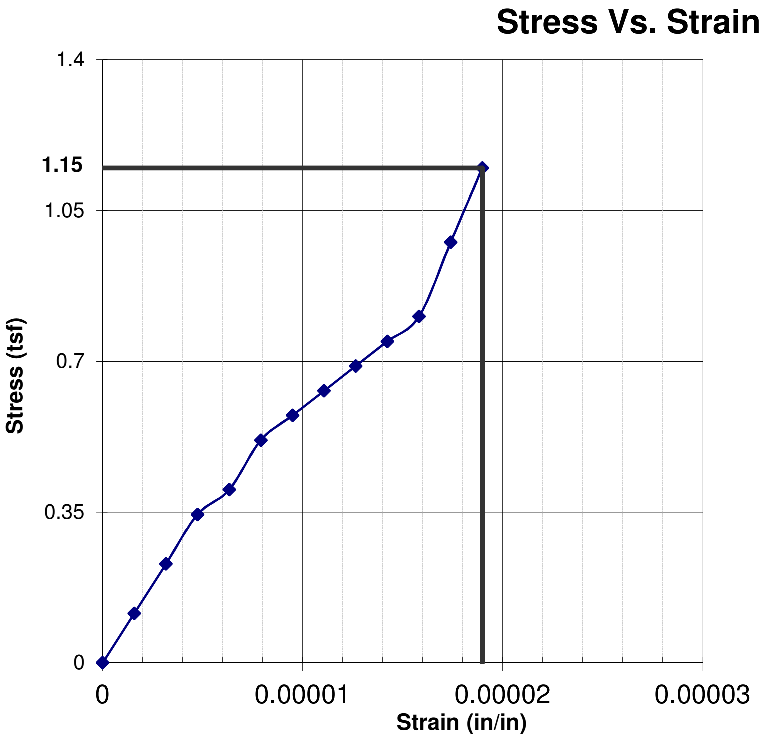
Unconfined Compression Test

Date:	7/18/2016	Project:	EnsiteUSA-CPG Pipeline, Washington County, MD
Tested by:	Redha K Hasan	Project No:	0512713-1
Client:		Location:	Hancock. MD

Average Initial Height (in):	6.32	Boring :	GO1
Average Initial Diameter (in):	2.83	Sample Number:	ST3
Water Content %:	21.2	Sample Depth:	18.3'-18.8'
Wet Density (pcf):	117.7		
Dry Density (pcf):	97.2	Soil Description:	CL
LL - PL = PI:	44 - 26 = 18		



Failure Picture

Unconfined Compression Strength q_u (tsf):	**1.15**
Height to Diameter Ratio:	**2.2**
Percent Strain at Failure :	**0.00%**
Average Rate of Strain to Failure (% Strain/min):	**0.00%**

This test was performed according to ASTM D2166 .
Unconfined Compressive Strength of Cohesive Soil.

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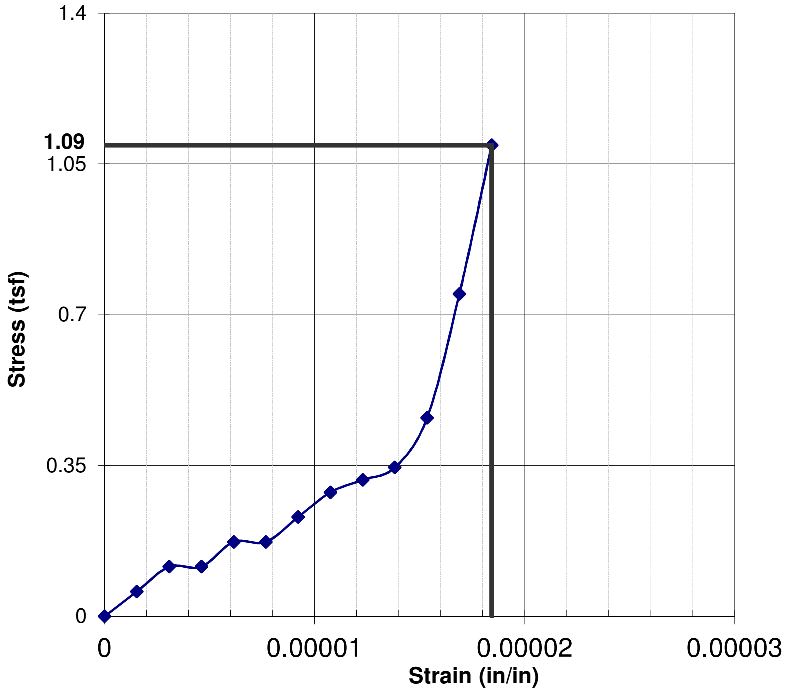
Unconfined Compression Test

Date:	7/18/2016	Project:	EnsiteUSA-CPG Pipeline, Washington County, MD
Tested by:	Redha K Hasan	Project No:	0512713-1
Client:	CPG	Location:	Hancock, MD

Average Initial Height (in):	6.51	Boring :	GO2R
Average Initial Diameter (in):	2.82	Sample Number:	ST-3
Water Content %:	0.0	Sample Depth:	8.5'-9.0'
Wet Density (pcf):	125.0		
Dry Density (pcf):	104.0	Soil Description:	CL
LL - PL = PI:	33 - 18 = 15		

Stress Vs. Strain



Failure Picture

Unconfined Compression Strength q_u (tsf):	**1.09**
Height to Diameter Ratio:	**2.3**
Percent Strain at Failure :	**0.00%**
Average Rate of Strain to Failure (% Strain/min):	**0.00%**

This test was performed according to ASTM D2166 .
Unconfined Compressive Strength of Cohesive Soil.

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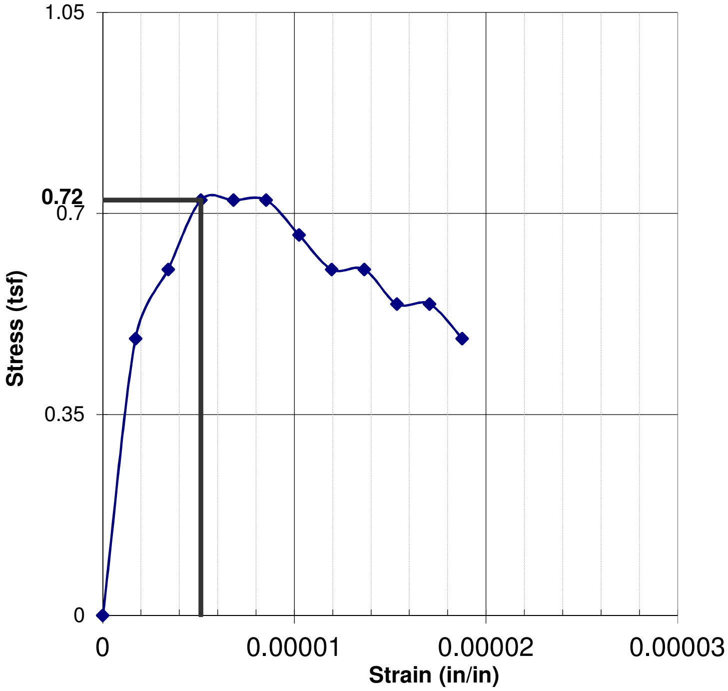
Unconfined Compression Test

Date:	7/18/2016	Project:	EnsiteUSA-CPG Pipeline, Washington County, MD
Tested by:	Redha K Hasan	Project No:	0512713-1
Client:	CPG	Location:	Hancock. MD

Average Initial Height (in):	5.86	Boring :	GO2 R
Average Initial Diameter (in):	2.76	Sample Number:	ST-1
Water Content %:	13.8	Sample Depth:	1.5'-2.0'
Wet Density (pcf):	114.1		
Dry Density (pcf):	100.3	Soil Description:	CL
LL - PL = PI:	38 - 22 = 16		

Stress Vs. Strain



Failure Picture

Unconfined Compression Strength q_u (tsf):	**0.72**
Height to Diameter Ratio:	**2.1**
Percent Strain at Failure :	**0.00%**
Average Rate of Strain to Failure (% Strain/min):	**0.00%**

This test was performed according to ASTM D2166 .
Unconfined Compressive Strength of Cohesive Soil.

Slake Durability Test Results

EnSite USA Project Number 6493
Eastern Panhandle Expansion, Potomac River Crossing
PSI Project Number 0512713-1
Revised September 9, 2016

Slake Durability of Shales and Similar Weak Rocks - ASTM D4644

Client	Professional Service Industries, Inc. (PSI)	Boring	GO-1
Client Project	Proposed Eastern Panhandle Expansion	Depth	51.0'-52.0'
Project No.	37622	Sample	RC-13
		Lab Sample No.	37622003

Visual Description: Gray Limestone

Initial Water Content

Drum ID	B
Drum + Wet Shale, gm	1723.59
Drum + Dry Shale, gm	1714.96
Drum Wt., gm	1244.24
Water Content, %	1.83%
Initial Dry Shale Weight, gm	470.72
Water Temerature Before Cycle 1, *C	19.5
Water Temerature After Cycle 1, *C	19.6
Average Temp during Cycle 1, *C	19.55
Drum + Dry Shale after Cycle 1, gm	1712.79
Dry Shale after Cycle 1	468.55
Slake Durability Index (First cycle)	**99.5%**
Water Temerature Before Cycle 2, *C	20.1
Water Temerature After Cycle 2, *C	21.1
Average Temp during Cycle 2, *C	20.6
Drum + Dry Shale after Cycle 2, gm	1711.58
Dry Shale after Cycle 2	467.34
Slake Durability Index (Second cycle)	**99.3%**

Type II—Retained specimen consist of large and small fragments.

Initial Photograph

Final Photograph

Input Validation: tmp Reviewed By: SVG Date Tested: 7/14/2016

Slake Durability of Shales and Similar Weak Rocks - ASTM D4644

Client	Professional Service Industries, Inc. (PSI)	Boring	GO-1
Client Project	Proposed Eastern Panhandle Expansion	Depth	64.5'-65.5'
Project No.	37622	Sample	RC-16
		Lab Sample No.	37622004

Visual Description: Gray Limestone

Initial Water Content

Drum ID	A
Drum + Wet Shale, gm	1711.15
Drum + Dry Shale, gm	1703.49
Drum Wt., gm	1244.85
Water Content, %	1.67%
Initial Dry Shale Weight, gm	458.64
Water Temerature Before Cycle 1, *C	19.9
Water Temerature After Cycle 1, *C	20.1
Average Temp during Cycle 1, *C	20
Drum + Dry Shale after Cycle 1, gm	1700.98
Dry Shale after Cycle 1	456.13
Slake Durability Index (First cycle)	**99.5%**
Water Temerature Before Cycle 2, *C	21.2
Water Temerature After Cycle 2, *C	21.6
Average Temp during Cycle 2, *C	21.4
Drum + Dry Shale after Cycle 2, gm	1699.6
Dry Shale after Cycle 2	454.75
Slake Durability Index (Second cycle)	**99.2%**

Type II—Retained specimen consist of large and small fragments.

Initial Photograph

Final Photograph

Input Validation: tmp Reviewed By: SVG Date Tested: 7/14/2016

Slake Durability of Shales and Similar Weak Rocks - ASTM D4644

Client	Professional Service Industries, Inc. (PSI)	Boring	GO-1
Client Project	Proposed Eastern Panhandle Expansion	Depth	95.0'-96.0'
Project No.	37622	Sample	RC-22
		Lab Sample No.	37622005

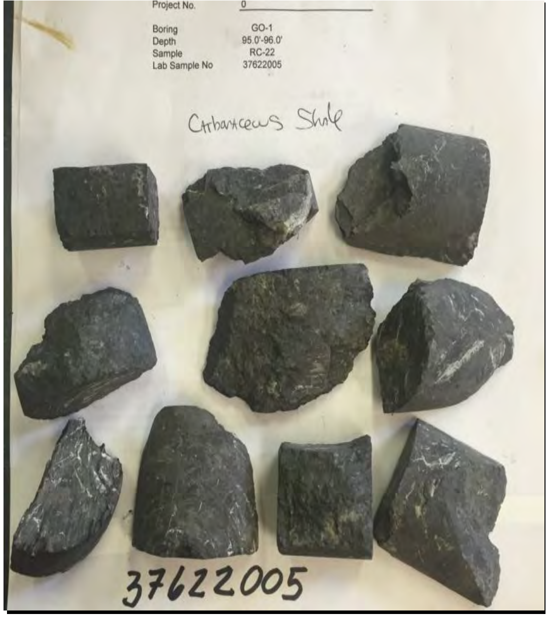
Visual Description: Gray Carbonaceous Shale

Initial Water Content

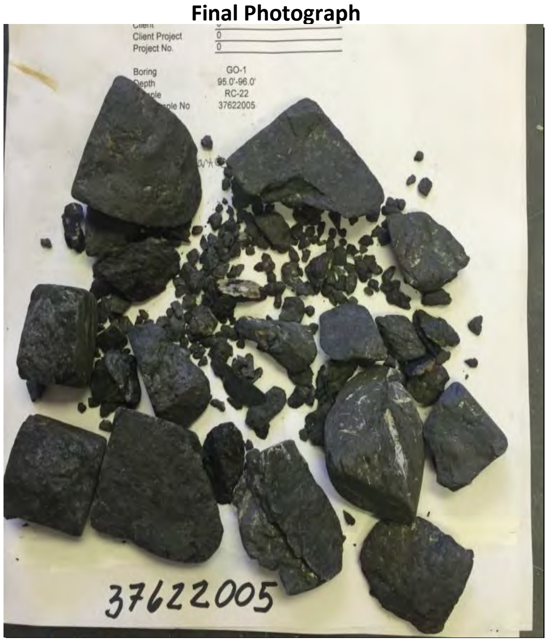
Drum ID	B
Drum + Wet Shale, gm	1760.42
Drum + Dry Shale, gm	1756.59
Drum Wt., gm	1244.15
Water Content, %	0.75%
Initial Dry Shale Weight, gm	512.44
Water Temerature Before Cycle 1, *C	21.1
Water Temerature After Cycle 1, *C	21.5
Average Temp during Cycle 1, *C	21.3
Drum + Dry Shale after Cycle 1, gm	1729.77
Dry Shale after Cycle 1	485.62
Slake Durability Index (First cycle)	**94.8%**
Water Temerature Before Cycle 2, *C	22.1
Water Temerature After Cycle 2, *C	22.6
Average Temp during Cycle 2, *C	22.35
Drum + Dry Shale after Cycle 2, gm	1705.12
Dry Shale after Cycle 2	460.97
Slake Durability Index (Second cycle)	**90.0%**

Type II—Retained specimen consist of large and small fragments.

Initial Photograph



Final Photograph



Input Validation: tmp Reviewed By: SVG Date Tested: 7/19/2016

Slake Durability of Shales and Similar Weak Rocks - ASTM D4644

Client	Professional Service Industries, Inc. (PSI)	Boring	GO-1
Client Project	Proposed Eastern Panhandle Expansion	Depth	129.0'-130.0'
Project No.	37622	Sample	RC-29
		Lab Sample No.	37622006

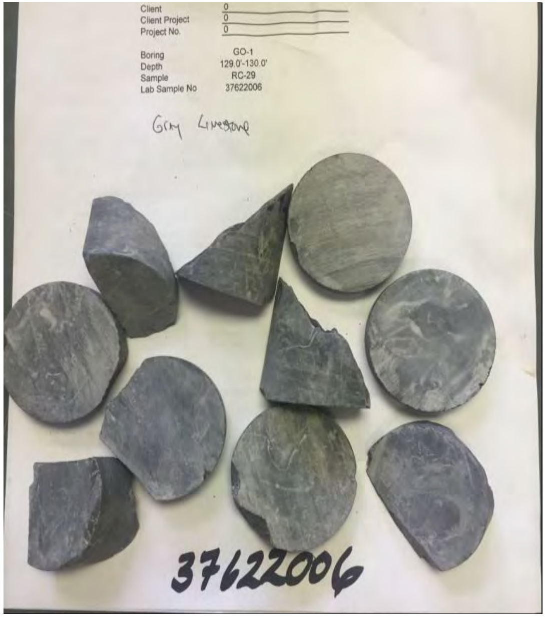
Visual Description: Gray Limestone

Initial Water Content

Drum ID	A
Drum + Wet Shale, gm	1760.93
Drum + Dry Shale, gm	1754.3
Drum Wt., gm	1244.79
Water Content, %	1.30%
Initial Dry Shale Weight, gm	509.51
Water Temerature Before Cycle 1, *C	21.8
Water Temerature After Cycle 1, *C	22
Average Temp during Cycle 1, *C	21.9
Drum + Dry Shale after Cycle 1, gm	1749.97
Dry Shale after Cycle 1	505.18
Slake Durability Index (First cycle)	**99.2%**
Water Temerature Before Cycle 2, *C	21.2
Water Temerature After Cycle 2, *C	21.3
Average Temp during Cycle 2, *C	21.25
Drum + Dry Shale after Cycle 2, gm	1747.9
Dry Shale after Cycle 2	503.11
Slake Durability Index (Second cycle)	**98.7%**

Type II—Retained specimen consist of large and small fragments.

Initial Photograph



Final Photograph

Input Validation: tmp   Reviewed By: SVG   Date Tested: 7/18/2016

Slake Durability of Shales and Similar Weak Rocks - ASTM D4644

Client	Professional Service Industries, Inc. (PSI)	Boring	GO-1
Client Project	Proposed Eastern Panhandle Expansion	Depth	203.0'-204.0'
Project No.	37622	Sample	RC-43
		Lab Sample No.	37622007

Visual Description: Brown Shale *(Sample Did Not Meet Mass Requirements)*

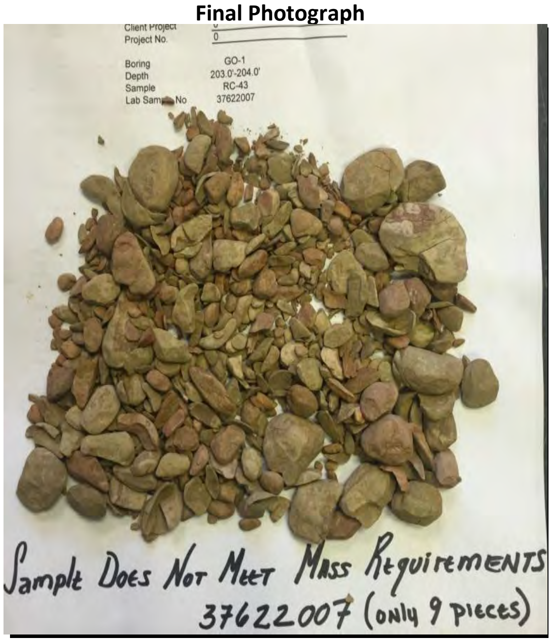
Initial Water Content

Drum ID	A
Drum + Wet Shale, gm	1685.07
Drum + Dry Shale, gm	1678.41
Drum Wt., gm	1244.75
Water Content, %	1.54%
Initial Dry Shale Weight, gm	433.66
Water Temerature Before Cycle 1, *C	20.9
Water Temerature After Cycle 1, *C	21.4
Average Temp during Cycle 1, *C	21.15
Drum + Dry Shale after Cycle 1, gm	1531.42
Dry Shale after Cycle 1	286.67
Slake Durability Index (First cycle)	**66.1%**
Water Temerature Before Cycle 2, *C	21.9
Water Temerature After Cycle 2, *C	22.3
Average Temp during Cycle 2, *C	22.1
Drum + Dry Shale after Cycle 2, gm	1430.81
Dry Shale after Cycle 2	186.06
Slake Durability Index (Second cycle)	**42.9%**

Type III—Retained specimen is exclusively small fragments.

Initial Photograph

Final Photograph



Input Validation: tmp Reviewed By: SVG Date Tested: 7/19/2016

Slake Durability of Shales and Similar Weak Rocks - ASTM D4644

Client	Professional Service Industries, Inc. (PSI)	Boring	GO-1
Client Project	Proposed Eastern Panhandle Expansion	Depth	255.0'-256.0'
Project No.	37622	Sample	RC-54
		Lab Sample No.	37622008

Visual Description: Brown Sandstone

Initial Water Content

Drum ID	C
Drum + Wet Shale, gm	1745.13
Drum + Dry Shale, gm	1732.73
Drum Wt., gm	1257.63
Water Content, %	2.61%
Initial Dry Shale Weight, gm	475.1
Water Temerature Before Cycle 1, *C	19.7
Water Temerature After Cycle 1, *C	20
Average Temp during Cycle 1, *C	19.85
Drum + Dry Shale after Cycle 1, gm	1545.18
Dry Shale after Cycle 1	287.55
Slake Durability Index (First cycle)	**60.5%**
Water Temerature Before Cycle 2, *C	21.1
Water Temerature After Cycle 2, *C	21.4
Average Temp during Cycle 2, *C	21.25
Drum + Dry Shale after Cycle 2, gm	1397.53
Dry Shale after Cycle 2	139.9
Slake Durability Index (Second cycle)	**29.4%**

Type III—Retained specimen is exclusively small fragments.

Initial Photograph

Final Photograph

Input Validation: tmp Reviewed By: SVG Date Tested: 7/14/2016

Slake Durability of Shales and Similar Weak Rocks - ASTM D4644

Client	Professional Service Industries, Inc. (PSI)	Boring	GO-2R
Client Project	Proposed Eastern Panhandle Expansion	Depth	43.0'-44.0'
Project No.	37622	Sample	RC-11
		Lab Sample No.	37622009

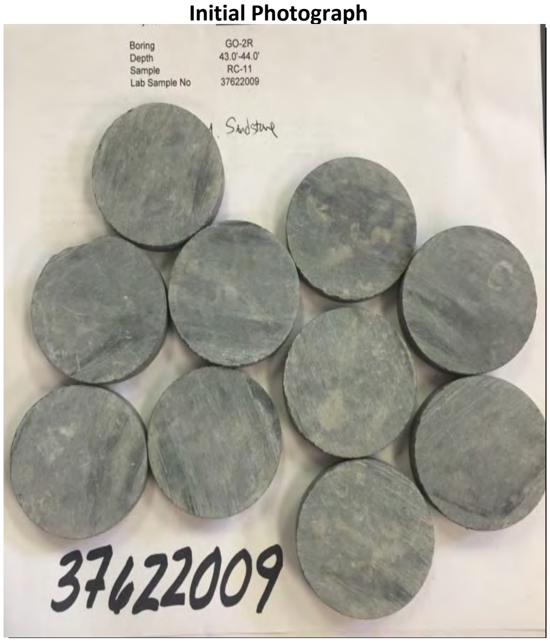
Visual Description: Gray Sandstone

Initial Water Content

Drum ID	D
Drum + Wet Shale, gm	1782.91
Drum + Dry Shale, gm	1780.48
Drum Wt., gm	1260.77
Water Content, %	0.47%
Initial Dry Shale Weight, gm	519.71
Water Temerature Before Cycle 1, *C	20.4
Water Temerature After Cycle 1, *C	21.2
Average Temp during Cycle 1, *C	20.8
Drum + Dry Shale after Cycle 1, gm	1780.17
Dry Shale after Cycle 1	519.4
Slake Durability Index (First cycle)	**99.9%**
Water Temerature Before Cycle 2, *C	21.4
Water Temerature After Cycle 2, *C	22.2
Average Temp during Cycle 2, *C	21.8
Drum + Dry Shale after Cycle 2, gm	1779.33
Dry Shale after Cycle 2	518.56
Slake Durability Index (Second cycle)	**99.8%**

Type I—Retained specimen remain virtually unchanged

Initial Photograph



Final Photograph

Input Validation: tmp Reviewed By: SVG Date Tested: 7/19/2016

Slake Durability of Shales and Similar Weak Rocks - ASTM D4644

Client	Professional Service Industries, Inc. (PSI)	Boring	GO-2R
Client Project	Proposed Eastern Panhandle Expansion	Depth	58.0'-59.0'
Project No.	37622	Sample	RC-14
		Lab Sample No.	37622010

Visual Description: Gray Shale

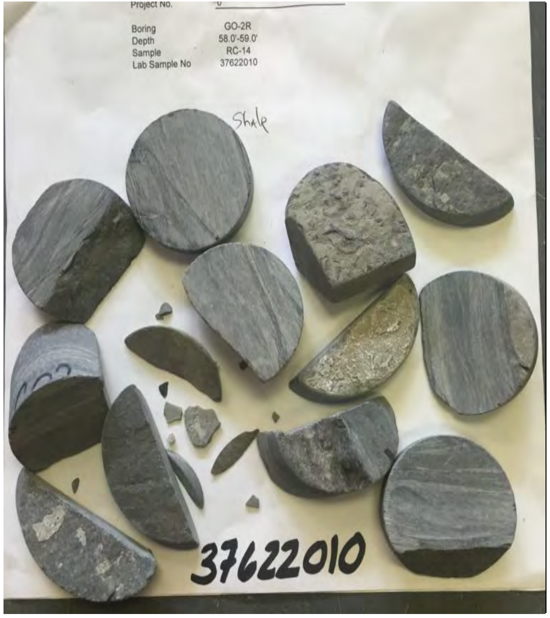
Initial Water Content

Drum ID	C
Drum + Wet Shale, gm	1769.7
Drum + Dry Shale, gm	1767.32
Drum Wt., gm	1257.94
Water Content, %	0.47%
Initial Dry Shale Weight, gm	509.38
Water Temerature Before Cycle 1, *C	21.7
Water Temerature After Cycle 1, *C	21.9
Average Temp during Cycle 1, *C	21.8
Drum + Dry Shale after Cycle 1, gm	1764.24
Dry Shale after Cycle 1	506.3
Slake Durability Index (First cycle)	**99.4%**
Water Temerature Before Cycle 2, *C	20.7
Water Temerature After Cycle 2, *C	21.2
Average Temp during Cycle 2, *C	20.95
Drum + Dry Shale after Cycle 2, gm	1762.76
Dry Shale after Cycle 2	504.82
Slake Durability Index (Second cycle)	**99.1%**

Type I—Retained specimen remain virtually unchanged

Initial Photograph

Final Photograph



Input Validation: tmp Reviewed By: SVG Date Tested: 7/18/2016

Slake Durability of Shales and Similar Weak Rocks - ASTM D4644

Client	Professional Service Industries, Inc. (PSI)	Boring	GO-2R
Client Project	Proposed Eastern Panhandle Expansion	Depth	70.5'-71.5'
Project No.	37622	Sample	RC-11
		Lab Sample No.	37622011

Visual Description: Brown Limestone

Initial Water Content

Drum ID	D
Drum + Wet Shale, gm	1736.6
Drum + Dry Shale, gm	1728.84
Drum Wt., gm	1260.89
Water Content, %	1.7%
Initial Dry Shale Weight, gm	467.95
Water Temerature Before Cycle 1, *C	19.7
Water Temerature After Cycle 1, *C	19.9
Average Temp during Cycle 1, *C	19.8
Drum + Dry Shale after Cycle 1, gm	1728.33
Dry Shale after Cycle 1	467.44
Slake Durability Index (First cycle)	**99.9%**
Water Temerature Before Cycle 2, *C	20.4
Water Temerature After Cycle 2, *C	21.2
Average Temp during Cycle 2, *C	20.8
Drum + Dry Shale after Cycle 2, gm	1728.34
Dry Shale after Cycle 2	467.45
Slake Durability Index (Second cycle)	**99.9%**

Type I—Retained specimen remain virtually unchanged

Initial Photograph

Final Photograph

Input Validation: tmp   Reviewed By: SVG   Date Tested: 7/14/2016

Slake Durability of Shales and Similar Weak Rocks - ASTM D4644

Client	Professional Service Industries, Inc. (PSI)	Boring	GO-2R
Client Project	Proposed Eastern Panhandle Expansion	Depth	100.0'-101.0'
Project No.	37622	Sample	RC-14
		Lab Sample No.	37622012

Visual Description: Gray Interbedded Shale and Siltstone

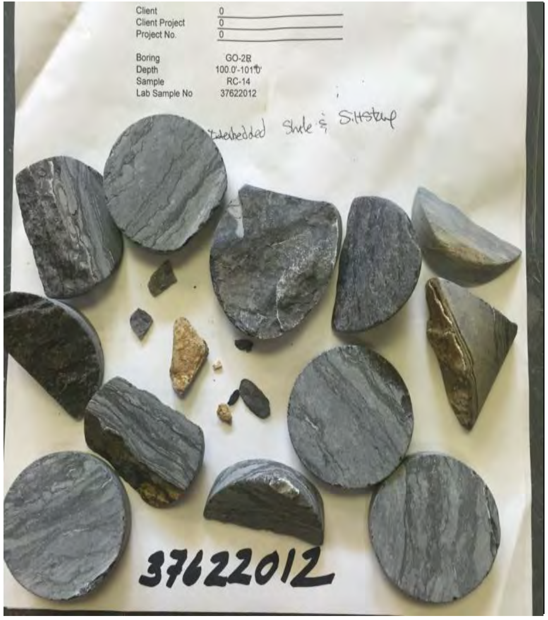
Initial Water Content

Drum ID	D
Drum + Wet Shale, gm	1794.64
Drum + Dry Shale, gm	1793.39
Drum Wt., gm	1260.89
Water Content, %	0.23%
Initial Dry Shale Weight, gm	532.5
Water Temerature Before Cycle 1, *C	21.4
Water Temerature After Cycle 1, *C	21.8
Average Temp during Cycle 1, *C	21.6
Drum + Dry Shale after Cycle 1, gm	1791.2
Dry Shale after Cycle 1	530.31
Slake Durability Index (First cycle)	**99.6%**
Water Temerature Before Cycle 2, *C	20.5
Water Temerature After Cycle 2, *C	20.9
Average Temp during Cycle 2, *C	20.7
Drum + Dry Shale after Cycle 2, gm	1789.45
Dry Shale after Cycle 2	528.56
Slake Durability Index (Second cycle)	**99.3%**

Type I—Retained specimen remain virtually unchanged

Initial Photograph

Final Photograph



Input Validation: tmp Reviewed By: SVG Date Tested: 7/18/2016

Slake Durability of Shales and Similar Weak Rocks - ASTM D4644

Client	Professional Service Industries, Inc. (PSI)	Boring	GO-2R
Client Project	Proposed Eastern Panhandle Expansion	Depth	122.0'-123.0'
Project No.	37622	Sample	RC-9
		Lab Sample No.	37622013

Visual Description: Gray Shale

Initial Water Content

Drum ID	C
Drum + Wet Shale, gm	1787.6
Drum + Dry Shale, gm	1783.77
Drum Wt., gm	1257.52
Water Content, %	0.73%
Initial Dry Shale Weight, gm	526.25
Water Temerature Before Cycle 1, *C	20.3
Water Temerature After Cycle 1, *C	20.9
Average Temp during Cycle 1, *C	20.6
Drum + Dry Shale after Cycle 1, gm	1781.8
Dry Shale after Cycle 1	524.28
Slake Durability Index (First cycle)	**99.6%**
Water Temerature Before Cycle 2, *C	21.2
Water Temerature After Cycle 2, *C	21.9
Average Temp during Cycle 2, *C	21.55
Drum + Dry Shale after Cycle 2, gm	1780.37
Dry Shale after Cycle 2	522.85
Slake Durability Index (Second cycle)	**99.4%**

Type I—Retained specimen remain virtually unchanged

Initial Photograph

Final Photograph

Input Validation: tmp Reviewed By: SVG Date Tested: 7/19/2016

Slake Durability of Shales and Similar Weak Rocks - ASTM D4644

Client	Professional Service Industries, Inc. (PSI)	Boring	GO-2R
Client Project	Proposed Eastern Panhandle Expansion	Depth	151.5'-152.5'
Project No.	37622	Sample	RC-13
		Lab Sample No.	37622014

Visual Description: Gray Interbedded Shale and Siltstone

Initial Water Content

Drum ID	B
Drum + Wet Shale, gm	1723.44
Drum + Dry Shale, gm	1713.92
Drum Wt., gm	1244.29
Water Content, %	2.03%
Initial Dry Shale Weight, gm	469.63
Water Temerature Before Cycle 1, *C	21.4
Water Temerature After Cycle 1, *C	21.7
Average Temp during Cycle 1, *C	21.55
Drum + Dry Shale after Cycle 1, gm	1712.79
Dry Shale after Cycle 1	468.5
Slake Durability Index (First cycle)	**99.8%**
Water Temerature Before Cycle 2, *C	21.3
Water Temerature After Cycle 2, *C	21.5
Average Temp during Cycle 2, *C	21.4
Drum + Dry Shale after Cycle 2, gm	1712.16
Dry Shale after Cycle 2	467.87
Slake Durability Index (Second cycle)	**99.6%**

Type I—Retained specimen remain virtually unchanged

Initial Photograph

Final Photograph

Input Validation: tmp Reviewed By: SVG Date Tested: 7/18/2016

Soil Resistivity Test Results

EnSite USA Project Number 6493
Eastern Panhandle Expansion, Potomac River Crossing
PSI Project Number 0512713-1
Revised September 9, 2016

Corrosivity Testing

Client	Professional Service Industries, Inc. (PSI)
Client Project	Proposed Eastern Panhandle Expansion
Project No.	37622

						pH AASHTO T289			Soil Resistivity AASHTO T-288		
Lab Sample ID	**Boring**	**Depth**	**Sample**	**Sample Received**	**Matrix**	**Result**	Date Tested	Tested By	**Result, Ohm-cm**	Date Tested	Tested By
37622001	GO-1	4.0'-6.0'	ST-1	7/14/2016	Soil	4.6	7/14/2016	TX	4200	7/19/2016	TX
37622002	GO-2R	4.0'-6.0'	ST-2	7/14/2016	Soil	7.1	7/14/2016	TX	1500	7/19/2016	TX

Input Validation: tmp

Reviewed By: SVG

Important Information About Your Geotechnical Report

EnSite USA Project Number 6493
Eastern Panhandle Expansion, Potomac River Crossing
PSI Project Number 0512713-1
Revised September 9, 2016

Important Information about Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

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

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

Read the Full Report

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

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

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

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

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

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

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

Subsurface Conditions Can Change

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

Most Geotechnical Findings Are Professional Opinions

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

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual

subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

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

Do Not Redraw the Engineer's Logs

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

Give Contractors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

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

Geoenvironmental Concerns Are Not Covered

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

Obtain Professional Assistance To Deal with Mold

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

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

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

ASFE
THE BEST PEOPLE ON EARTH

8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@asfe.org www.asfe.org

Information To Build On
Engineering • Consulting • Testing

February 8, 2017

EnSite USA, Inc.
109 Fieldview Drive
Versailles, KY, 40383
Attn: Jacob Shams, P.E.

Re: Addendum to Report for Geotechnical Subsurface Exploration & Engineering Services
6493 – Eastern Panhandle Expansion
Potomac River Crossing, Borings GO-3R and GO-7
Washington County, Maryland.
PSI Project Number 0512713-2

Dear Mr. Shams:

Thank you for choosing Professional Service Industries, Inc. (PSI) as your consultant for the referenced project. Authorization to perform services was provided through PSI Proposal No. 0512-187239 dated August 9, 2016. The proposal was executed by Mr. Shams, P.E. representing EnSite USA, Inc.

This addendum letter to PSI geotechnical report presents the results of borings performed by PSI at two locations along the proposed HDD alignment at Potomac River Crossing. Approximate boring locations are presented in the Appendix Figures: 1A Site Vicinity Map and 1B: Boring Location Plan.

Scope of Services

PSI's services consisted of field exploration, laboratory testing, and preparation of a geotechnical engineering report for the proposed HDD location. Field work included drilling two test borings (Borings GO-3R and GO-7), utilizing hollow-stem auger (HSA) drilling, wash rotary drilling, and rock coring in conformance with ASTM standards.

Laboratory testing determined unit weight, moisture content, Atterberg limits, grain size distribution tests, pH and resistivity testing, unconsolidated undrained compressive strength and slake durability testing. All tests were performed per ASTM standards.

Summary of Field Exploration and Laboratory Testing

The borings were completed with a track-mounted drill rig with HSA in conformance with ASTM standards. Standard Penetration Testing (SPT) and split-spoon sampling of overburden soils was performed at 2.5 foot intervals for the first 10 feet and at 5-foot intervals thereafter to the termination depths to evaluate the strength and relative consistency of the soils encountered. Below auger refusal depth, rock coring was performed using NQ coring equipment. All recovered soil and rock samples were visually classified by a PSI geotechnical engineer and a graphical log developed for each boring. Boring depth and depth at which auger refusal was encountered are summarized in Table 1 below.

Table 1 – Summary of Boring Depths

Boring	Approximate Termination Depth (feet)	Ground Surface Elevation (feet, NAVD)	Approximate Depth/Elevation of Top of Weathered Rock	Approximate Depth/Elevation of Auger Refusal
GO-3R	277	591	5.5 feet, EL ±585.5MSL	20 feet, EL ±571MSL
GO-7	134	402	6 feet, EL ±396MSL	14 feet, EL ±388MSL

The boring logs included in the Appendix approximate depths and visual descriptions of overburden soil, underlying rock materials encountered, soil SPT test results, rock core recovery and quality designation (RQD) values, and measurements of groundwater depth where encountered. The total length of recovered rock core, divided by the length of the run, is referred to as rock core recovery and is expressed as a percentage. The Rock Quality Designation (RQD) is a measure of the rock mass quality and is defined as the total length of sound, intact rock core pieces 4 inches or more in length divided by the length of the rock core run, also expressed as a percentage. The rock core recovery and RQD values are indicated on the Boring Logs included with this report.

Geotechnical Investigation Results

A brief summary of subsurface stratigraphy as encountered at the boring is presented below. All soil is classified per the Unified Soil Classification System (ASTM D-2487):

Surficial Materials: Approximately 3 inches of surficial **topsoil** were encountered at the ground surface of Boring GO-3R. Approximately 6 feet of **water** were encountered at Potomac river to the river bottom surface at Boring GO-7.

Alluvium with thickness up to 2 feet consisting of medium stiff lean Clay (CL) with gravel, roots. SPT N-values counted in this layer were 8 blows per foot of penetration (BPF).

Residuum: Residual soil classified as dense to very dense silty Gravel (GM) was encountered to depth of approximately 9 feet below existing surface grade at the Test Boring GO-3R. The residual soil was approximately 7 feet thick in the test boring location GO-3R. SPT N-values in this layer ranged from approximately 32 BPF to refusal of 50 blows per 0-inch of penetration.

Weathered Rock: Typically consisting of weathered shale, weathered rock was encountered at the test boring locations. The weathered rock samples consisted of soft shale with limestone floaters. SPT N-values in this layer ranged from approximately 50 blows per 5-inches of penetration to 50 blows per 2-inches of penetration. Auger refusal was encountered within the weathered rock at depths ranging from approximately 5.5 to 20 feet below existing grade at Boring GO-3R.

Bedrock: Bedrock materials encountered below the auger refusal depths consisted primarily of Shale and Limestone. Voids were encountered in Boring GO-7. At Boring GO-3R core recoveries ranged from 55 to 100 percent and RQD values ranged from 7 to 100 percent. At Boring GO-7 core recoveries ranged from 23 to 100 percent and RQD values ranged from 0 to 100 percent.

The above subsurface descriptions are of a generalized nature provided to highlight the major strata encountered. The boring logs included in the Appendix should be reviewed for specific information as to individual boring locations. The stratification lines shown on the boring logs represent the conditions only at the actual boring locations. The stratification lines represent the approximate boundaries between subsurface materials and the actual transition may be gradual.

Table 2 – Overburden Soil Classification Test Results

Boring	Sample No.	Sample Depth (feet)	USCS Classification [(1)]	Moisture Content (%)	Atterberg Limits			Grain-Size Distribution		
					Liquid Limit	Plastic Limit	Plasticity Index	Gravel (%)	Sand (%)	Fines (%)
GO-3R	S-1	0.0 - 1.5	CL	10						
GO-3R	ST-1	0.0 – 2.0	GM	21.4	NP	NP	NP	39.7	28.0	32.3
GO-3R	S-2	2.5 – 4.0	GM	7	26	17	9	60.2	18.5	21.3
GO-3R	S-3	5.0 – 6.5	GM	6						
GO-3R	S-4	8.5 – 10.0	GM	6						
GO-3R	S-5	13.5 – 15.0	GM	4				36.7	36.0	27.3
GO-3R	S-6	18.5 – 20.0	GM	9						

(1) For USCS Soil Classification definitions, refer to the General Notes in Attachment

(2) ST – Shelby Tube soil sample

(3) S – Split spoon soil sample

Table 3 – Rock Recovery and RQD Field Coring Test Results

Boring	Depth (feet)	Elevation (feet)	Run Length (feet)	Recovery (%)	RQD (%)	Hardness (Moh,s)
GO-3R	20 – 22	571 - 569	2	92	29	3
GO-3R	22 – 27	569 - 564	5	55	25	3
GO-3R	27 – 32	564 – 559	5	97	53	3
GO-3R	32 – 37	559 – 554	5	100	60	3
GO-3R	37 – 42	554 – 549	5	100	75	3
GO-3R	42 – 47	549 – 544	5	100	76	3
GO-3R	47 – 52	544 – 539	5	95	58	3
GO-3R	52 – 57	539 – 534	5	100	65	3
GO-3R	57 – 62	534 – 529	5	80	20	2.5 - 3
GO-3R	62 – 67	529 – 524	5	57	7	2.5 - 3
GO-3R	67 – 72	524 – 519	5	100	70	2.5 - 3
GO-3R	72 – 77	519 – 514	5	92	77	2.5 - 3
GO-3R	77 – 82	514 – 509	5	100	77	2.5 - 3

Table 3 – Rock Recovery and RQD Field Coring Test Results - Continued						
Boring	**Depth (feet)**	**Elevation (feet)**	**Run Length (feet)**	**Recovery (%)**	**RQD (%)**	**Hardness (Moh,s)**
GO-3R	82 – 87	509 - 504	5	100	73	2.5 - 3
GO-3R	87 – 92	504 - 499	5	100	83	3
GO-3R	92 – 97	499 – 494	5	100	93	3
GO-3R	97 – 102	494 – 489	5	100	100	3 - 4
GO-3R	102 – 107	489 – 484	5	100	77	3 - 4
GO-3R	107 – 112	484 – 479	5	85	40	3 - 4
GO-3R	112 – 117	479 – 474	5	100	70	3 - 4
GO-3R	117 - 122	474 – 469	5	53	37	3 - 4
GO-3R	122 – 127	469 – 464	5	72	27	3 - 4
GO-3R	127 – 132	464 – 459	5	100	75	3 and 5 - 6
GO-3R	132 – 137	459 – 454	5	60	7	3 and 5 - 6
GO-3R	137 – 142	454 – 449	5	95	67	3 and 5 - 6
GO-3R	142 – 147	449 – 444	5	100	97	3 and 5 - 6
GO-3R	147 – 152	444 – 439	5	100	95	3 and 5 - 6
GO-3R	152 – 157	439 – 434	5	100	100	3 and 5 - 6
GO-3R	157 – 162	434 – 429	5	100	97	3
GO-3R	162 – 167	429 – 424	5	95	92	3
GO-3R	167 – 172	424 – 419	5	100	75	3
GO-3R	172 -177	419 – 414	5	100	77	3
GO-3R	177 – 182	414 – 409	5	100	83	3 and 5 - 6
GO-3R	182 – 187	409 – 404	5	73	30	3 and 5 - 6
GO-3R	187 – 192	404 – 399	5	100	93	3 and 5 - 6
GO-3R	192 – 197	399 – 394	5	100	100	3 and 5 - 6
GO-3R	197 – 202	394 – 389	5	100	97	3 and 5 - 6
GO-3R	202 – 207	389 – 384	5	100	98	3 and 5 - 6
GO-3R	207 – 212	384 – 379	5	100	100	3 and 5 - 6
GO-3R	212 – 217	379 – 374	5	93	92	3 and 5 - 6
GO-3R	217 – 222	374 – 369	5	93	86	3 and 5 - 6
GO-3R	222 – 227	369 – 364	5	100	95	3 and 5 - 6

Table 3 – Rock Recovery and RQD Field Coring Test Results - Continued

Boring	Depth (feet)	Elevation (feet)	Run Length (feet)	Recovery (%)	RQD (%)	Hardness (Moh,s)
GO-3R	227 – 232	364 – 359	5	100	95	3 and 5 - 6
GO-3R	232 – 237	359 – 354	5	100	100	3 and 5 - 6
GO-3R	237 – 242	354 – 349	5	100	95	3 and 5 - 6
GO-3R	242 – 247	349 – 344	5	100	90	3 and 5 - 6
GO-3R	247 – 252	344 – 339	5	100	97	3 and 5 - 6
GO-3R	252 – 257	339 – 334	5	87	100	3 and 5 - 6
GO-3R	257 – 262	334 – 329	5	97	50	3 and 4 - 5
GO-3R	262 – 267	329 – 324	5	100	80	3 and 4 - 5
GO-3R	267 – 272	324 – 319	5	100	78	3 and 4 - 5
GO-3R	272 - 277	319 – 314	5	100	83	3 and 4 - 5
GO-7	6 – 9	396 – 393	3	50	0	7.5 - 8
GO-7	9 - 14	393 – 388	5	23	0	3 - 8
GO-7	14 – 19	388 – 383	5	96	88	2.5 - 4
GO-7	19 – 24	383 – 378	5	100	100	2.5 - 4
GO-7	24 – 29	378 – 373	5	94	86	2.5 - 4
GO-7	29 – 34	373 – 368	5	100	68	3
GO-7	34 – 39	368 – 363	5	60	0	4
GO-7	39 – 44	363 – 358	5	100	86	4
GO-7	44 – 49	358 – 353	5	96	72	3
GO-7	49 – 54	353 – 348	5	100	24	2.5 - 3
GO-7	54 – 59	348 – 343	5	84	64	2.5 - 3
GO-7	59 – 64	343 – 338	5	86	30	2.5 - 3
GO-7	64 – 69	338 – 333	5	100	70	2.5 - 3
GO-7	69 – 74	333- 328	5	72	60	2.5 - 3
GO-7	74 – 79	328 – 323	5	94	60	3 - 4
GO-7	79 – 84	323 – 318	5	100	74	3 - 4
GO-7	84 – 89	318 – 313	5	100	92	3 - 4
GO-7	89 – 94	313 – 308	5	100	82	3 - 4
GO-7	94 – 99	308 – 303	5	96	24	3 - 4

Table 3 – Rock Recovery and RQD Field Coring Test Results - Continued						
Boring	**Depth (feet)**	**Elevation (feet)**	**Run Length (feet)**	**Recovery (%)**	**RQD (%)**	**Hardness (Moh,s)**
GO-7	99 – 104	303 -298	5	96	50	3 - 4
GO-7	104 – 109	298 – 293	5	80	0	3 - 4
GO-7	109 – 114	293 – 288	5	100	34	3 - 4
GO-7	114 – 119	288 – 283	5	100	46	3
GO-7	119 – 124	283 – 278	5	100	28	3
GO-7	124 – 129	278 – 273	5	100	44	3
GO-7	129 - 134	273 - 268	5	100	72	3

Table 4 – Rock Unconfined Compressive Strength Test Results

Boring	**Approximate Sample Depth (feet)**	**Rock Classification**	**Unit Weight (pcf)**	**Unconfined Compressive Strength**	
				(psi)	**(tsf)**
GO-3R	40.5 - 41.0	**Limestone**	163.9	3528	254
GO-3R	101.0 – 101.5	**Limestone**	173.8	26,903	1937
GO-3R	168.5 – 169.0	**Shale**	175.6	1597	115
GO-3R	192.5 – 193.0	**Shale**	172.0	1569	113
GO-3R	221.0 - 221.5	**Shale**	172.7	1556	112
GO-3R	271.0 – 271.5	**Shale**	171.2	1556	112
GO-7	19.9 – 20.4	**Limestone**	158.7	4722	340
GO-7	42.1 – 42.6	**Shale**	158.3	1013	73
GO-7	90.5 – 91.0	**Shale**	144	48	3
GO-7	131.5 – 131.2	**Limestone**	168.5	7736	557

Table 5 – Soil, Unconfined Compressive Strength Test Results

Boring	**Approximate Sample Depth (feet)**	**Water Content (%)**	**Dry Unit Weight (pcf)**	**Soil Classification**	**Confining Stress**	**Shear Strength**	
					(psi)	**(psf)**	**(tsf)**
There were not available soil samples to perform Soil Unconfined Compressive Strength Tests							

The **durability** of the shale is a measurement of its deterioration over time interaction with the water weathering properties. The durability of the shale was determined on a selected sample of shales per Slake Durability of Shales and Similar Weak Rocks, ASTM D-4644 Standard.

Table 6 – Slake Durability Test Results

Boring	Approximate Sample Depth (feet)	Rock Classification	Slake Durability Index First Cycle (%)	Slake Durability Index Second Cycle (%)
GO-3R	28.4 – 29.4	Siltstone	98.3	97.3
GO-3R	141.0 – 142.0	Shale	99.3	98.5
GO-3R	177.5 – 178.5	Shale	99.5	98.9
GO-3R	228.0 – 229.0	Shale	99.7	99.5
GO-3R	273.0 – 274.0	Shale	99.7	99.5
GO-7	14.9 - 15.9	Siltstone	95.9	90.7
GO-7	54.0 - 55.0	Siltstone	99.3	99.0
GO-7	81.5 - 82.5	**Limestone**	99.0	98.4
GO-7	129.0 – 130.0	Shale	99.2	98.9

One (1) representative soil sample was selected by PSI for soil resistivity testing. Table 7 below presents a summary of the test results. A detailed report is included in the Appendix.

Table 7 – Soil Resistivity Test Results

Location	GO-3R
Depth (Foot)	0.0 – 2.0’
pH - AASHTO T289	4.8
Soil Resistivity – AASHTO T-288	28500 Ohm-cm

Should there be any questions, please do not hesitate to contact our office at (703) 698-9300. PSI would be pleased to continue providing geotechnical services throughout the implementation of the project, and we look forward to working with you on this and future projects.

Respectfully submitted,
PROFESSIONAL SERVICE INDUSTRIES, INC.

Lubomir D. Peytchev

Lubomir D. Peytchev, P.E.
Senior Geotechnical Engineer

Naseer Nayeem, P.E.
Vice President/Principal Consultant

Appendix: Figure 1A: Site Vicinity Map and Figure 1B: Boring Location Plan
Boring Logs and General Notes
Laboratory Test Results
Slake Durability Test Results
Soil Resistivity Test Results

APPENDIX A: IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL REPORT

Important Information about Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

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

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

Read the Full Report

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

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

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

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

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

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

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

Subsurface Conditions Can Change

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

Most Geotechnical Findings Are Professional Opinions

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

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual

subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

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

Do Not Redraw the Engineer's Logs

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

Give Contractors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

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

Geoenvironmental Concerns Are Not Covered

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

Obtain Professional Assistance To Deal with Mold

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

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

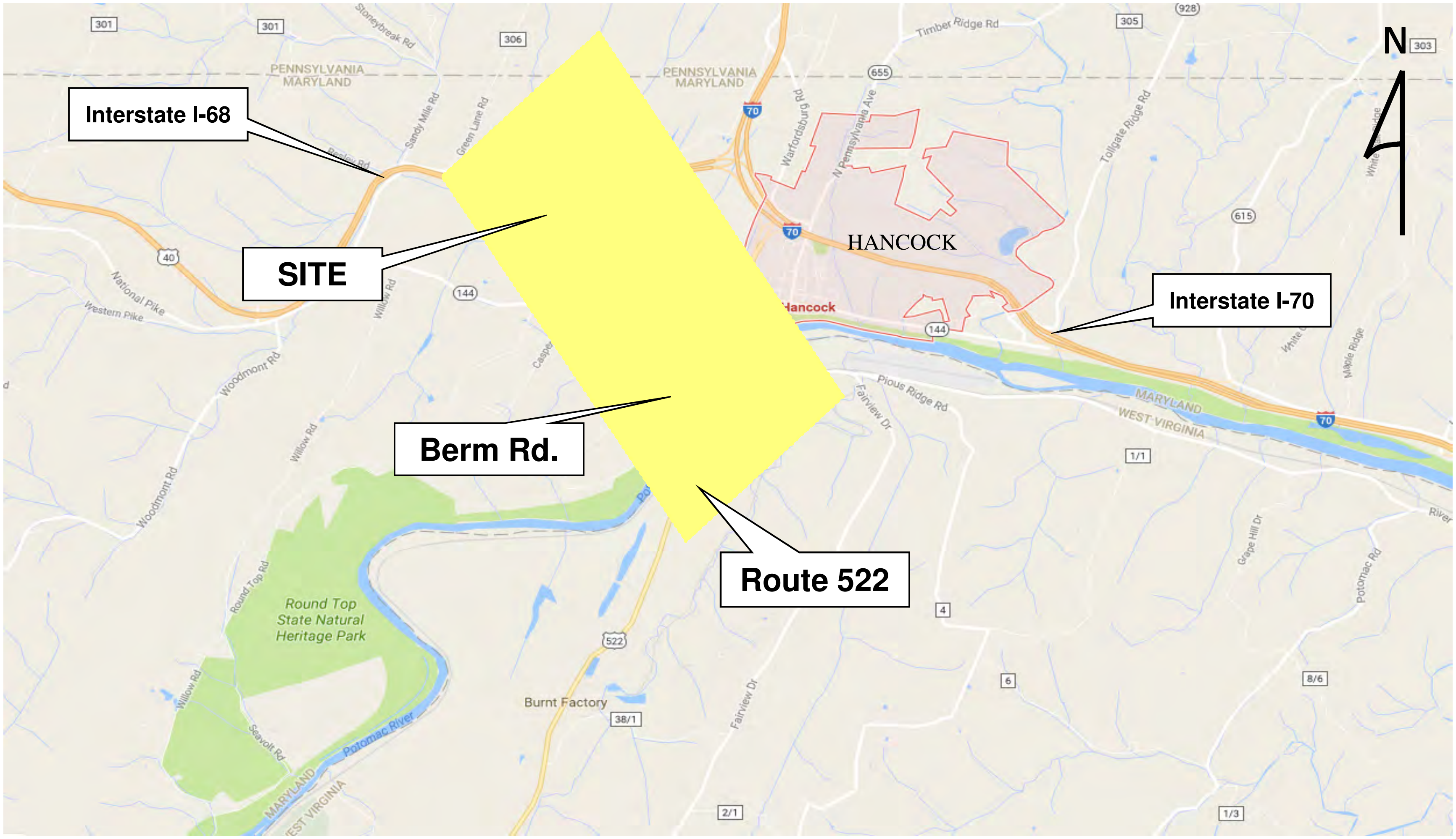
Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

ASFE
THE BEST PEOPLE ON EARTH

8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@asfe.org www.asfe.org

APPENDIX B – VICINITY MAP AND BORING LOCATION PLAN

Map Source: Google Earth

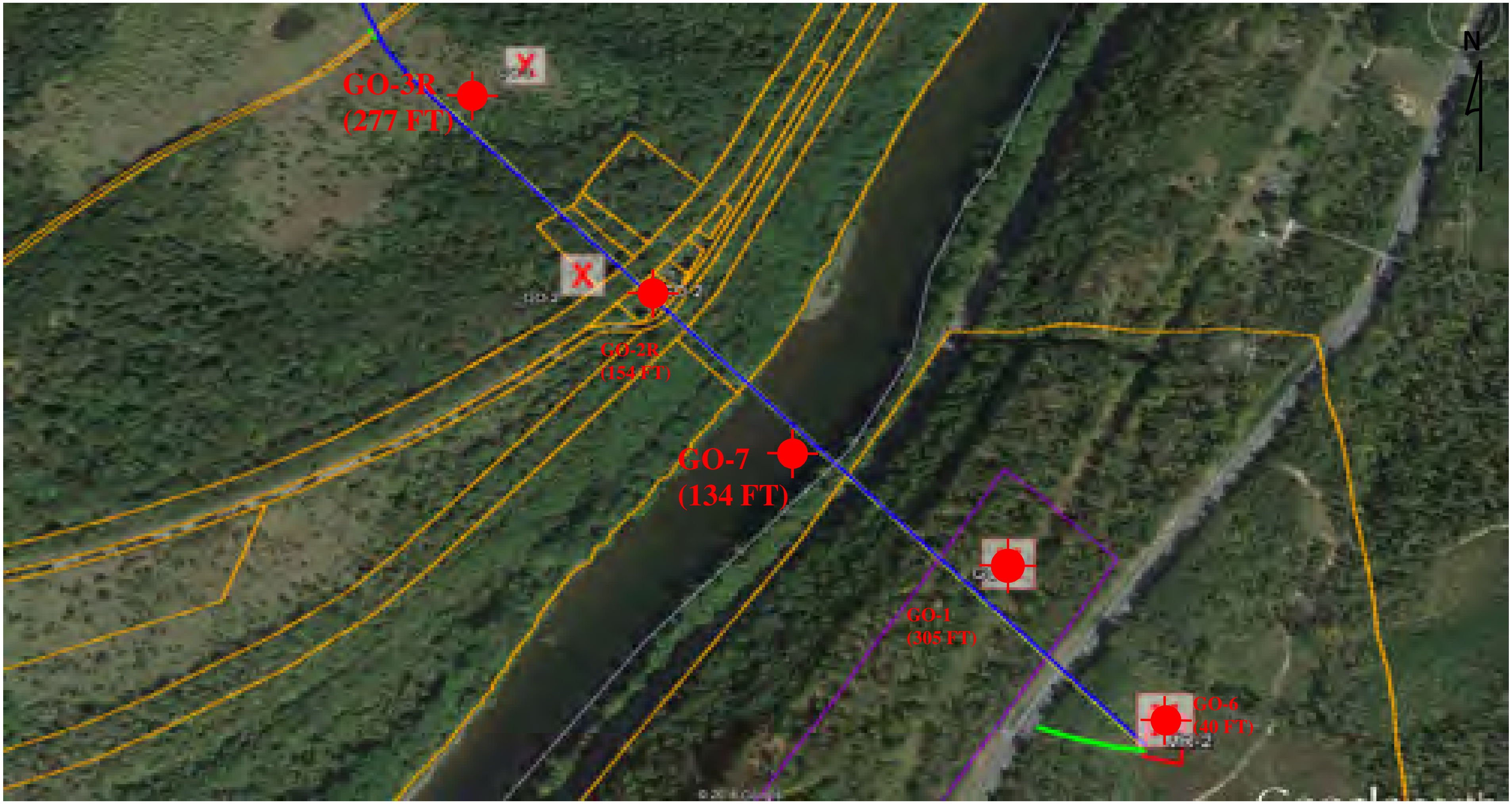


Information To Build On
Engineering • Consulting • Testing

REVISIONS

Site Vicinity Map (Figure 1A)
6493- Eastern Panhandle Expansion Potomac River Crossing

FULTON COUNTY, PA; WASHINGTON COUNTY, MD; AND MORGAN COUNTY, WV;	February 7, 2017
L.D.P. Not Drawn To Scale	0512713-2



LEGEND:

● GO-1 - PROPOSED BORING

(10 FT) - BORING DEPTH

NOTES:
1. ALL BORINGS WERE ADVANCED WITH HOLLOW-STEM AUGERS.
2. SPT SAMPLING WAS PERFORMED IN ALL BORINGS.
3. BORING DEPTHS ARE AS SHOWN
4. BORING SPOILS WERE USED TO BACKFILL THE BORE HOLES.

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REVISIONS

BORING LOCATION PLAN (FIGURE 1B)
6493- EASTERN PANHANDLE EXPANSION POTOMAC RIVER CROSSING

WASHINGTON COUNTY, MD AND MORGAN COUNTY, WV

February 7, 2017

L.D.P.

N.T.S.

0512713-2

APPENDIX C: BORING LOGS

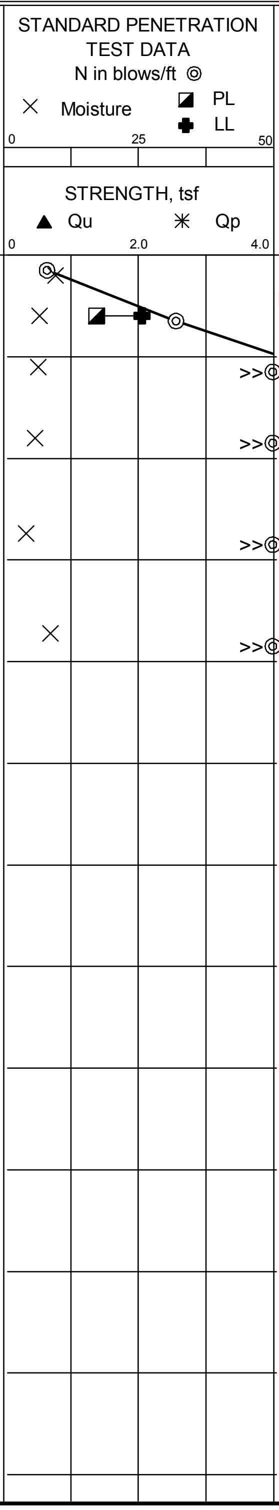
DATE STARTED: 12/15/16
DATE COMPLETED: 12/21/16
COMPLETION DEPTH 277.0 ft
BENCHMARK: N/A
ELEVATION: 591 ft
LATITUDE: 39.686351°
LONGITUDE: 78.20148°
STATION: N/A **OFFSET:** N/A
REMARKS:

DRILL COMPANY: Connelly Drilling, Inc.
DRILLER: Kevin Kersh **LOGGED BY:** Gunner Ingram
DRILL RIG: CME 550 ATV
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: 2-in SS1.874-in Core Standard
HAMMER TYPE: Automatic
EFFICIENCY N/A
REVIEWED BY: Lubomir Peytchev

BORING GO-3R

Water		
▽	While Drilling	Dry feet
▼	Upon Completion	Dry feet
▽	Delay	N/A feet

BORING LOCATION:



Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS) RQD & Recovery % (NX)	Moisture, %	Additional Remarks
590	0			1	8	Approximately 3 inches of Topsoil	Top Soil	2-3-5 N=8	10	
				2	9	Medium stiff, moist, red, **lean CLAY (USCS CL)** some gravel, roots.	CL	8-14-18 N=32	7	LL = 26 PL = 17 Fines=21.3%
585	5			3	9	Dense to very dense, moist, red **silty GRAVEL (USCS GM)** some sand. **Residuum**	GM	14-50/0"	6	
580	10			4	10	Weathered Rock, SHALE and LIMESTONE, sampled as very hard, moist, red **silty GRAVEL (USCS GM) with sand**		23-50/4"	6	
575	15			5	5		GM	50/5"	4	Fines=27.3%
	20			6	2			50/2"	9	
570				7	22	Interbedded, slightly weathered, medium bedded to thin bedded, red, fine grained to medium grained, very soft to soft SHALE and hard LIMESTONE, dip of 20 to 70 degrees, (RQD from 20 to 76 %), Silurian [Wills Creek Shale and Bloomsburg Formation]		RQD=29 Rec=92%		
565	25			8	33			RQD=25 Rec=55%		
560	30			9	58			RQD=53 Rec=97%		
555	35			10	60			RQD=60 Rec=100%		
550	40			11	60		Shale and Limestone	RQD=75 Rec=100%		
545	45			12	60			RQD=76 Rec=100%		
540	50			13	57			RQD=58 Rec=95%		
535	55			14	60			RQD=65 Rec=100%		
	60			15	48			RQD=20		

Continued Next Page

Professional Service Industries, Inc.
2930 Eskridge Rd
Fairfax, VA 22031
Telephone: (703) 698-9300

PROJECT NO.: 0512713-1
PROJECT: 6493-Eastern Panhandle Expansion
LOCATION: Potomac River Crossing
Washington County
Hancock, MD

The stratification lines represent approximate boundaries. The transition may be gradual.

DATE STARTED: 12/15/16
DATE COMPLETED: 12/21/16
COMPLETION DEPTH 277.0 ft
BENCHMARK: N/A
ELEVATION: 591 ft
LATITUDE: 39.686351°
LONGITUDE: 78.20148°
STATION: N/A **OFFSET:** N/A
REMARKS:

DRILL COMPANY: Connelly Drilling, Inc.
DRILLER: Kevin Kersh **LOGGED BY:** Gunner Ingram
DRILL RIG: CME 550 ATV
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: 2-in SS1.874-in Core Standard
HAMMER TYPE: Automatic
EFFICIENCY N/A
REVIEWED BY: Lubomir Peytchev

BORING GO-3R

Water		
▽ While Drilling	Dry	feet
▼ Upon Completion	Dry	feet
▽ Delay	N/A	feet

BORING LOCATION:

Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS) RQD & Recovery % (NX)	Moisture, %	STANDARD PENETRATION TEST DATA N in blows/ft ◎ × Moisture ◪ PL ✚ LL 0 25 50 STRENGTH, tsf ▲ Qu ✳ Qp 0 2.0 4.0	Additional Remarks
530	60							Rec=80%			
525	65			16	34	Interbedded, slightly weathered, medium bedded to thin bedded, gray, fine grained to medium grained, very soft to soft SHALE and hard LIMESTONE, dip of 40 to 50 degrees, (RQD from 7 to 100 %), Silurian [Wills Creek Shale and Bloomsburg Formation]		RQD=7 Rec=57%			
520	70			17	60			RQD=70 Rec=100%			
515	75			18	55			RQD=77 Rec=92%			
510	80			19	60			RQD=77 Rec=100%			
505	85			20	60		Shale and Limestone	RQD=73 Rec=100%			
500	90			21	60			RQD=83 Rec=100%			
495	95			22	60			RQD=93 Rec=100%			
490	100			23	60			RQD=100 Rec=100%			
485	105			24	60			RQD=77 Rec=100%			
480	110			25	51	Interbedded, slightly weathered, medium bedded to thin bedded, gray, fine grained to medium grained, very soft to soft SHALE and hard LIMESTONE, dip of 35 to 60 degrees, (RQD from 7 to 75 %), Silurian [Wills Creek Shale and Bloomsburg Formation]		RQD=40 Rec=85%			
475	115			26	60		Shale and Limestone	RQD=70 Rec=100%			
	120			27	32			RQD=37			

Continued Next Page

PROJECT NO.: 0512713-1
PROJECT: 6493-Eastern Panhandle Expansion
LOCATION: Potomac River Crossing
Washington County
Hancock, MD

The stratification lines represent approximate boundaries. The transition may be gradual.

DATE STARTED: 12/15/16
DATE COMPLETED: 12/21/16
COMPLETION DEPTH 277.0 ft
BENCHMARK: N/A
ELEVATION: 591 ft
LATITUDE: 39.686351°
LONGITUDE: 78.20148°
STATION: N/A **OFFSET:** N/A
REMARKS:

DRILL COMPANY: Connelly Drilling, Inc.
DRILLER: Kevin Kersh **LOGGED BY:** Gunner Ingram
DRILL RIG: CME 550 ATV
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: 2-in SS1.874-in Core Standard
HAMMER TYPE: Automatic
EFFICIENCY N/A
REVIEWED BY: Lubomir Peytchev

BORING GO-3R

Water			
	▽	While Drilling	Dry feet
	▼	Upon Completion	Dry feet
	▽	Delay	N/A feet

BORING LOCATION:

Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS) RQD & Recovery % (NX)	Moisture, %	STANDARD PENETRATION TEST DATA N in blows/ft ◎ × Moisture ◪ PL ✚ LL 0 25 50 STRENGTH, tsf ▲ Qu ✳ Qp 0 2.0 4.0	Additional Remarks
470	120					Interbedded, slightly weathered, medium bedded to thin bedded, gray, fine grained to medium grained, very soft to soft SHALE and hard LIMESTONE, dip of 35 to 60 degrees, (RQD from 7 to 75 %), Silurian [Wills Creek Shale and Bloomsburg Formation]	Shale and Limestone	Rec=53%			
465	125			28	43			RQD=27 Rec=72%			
460	130			29	60			RQD=75 Rec=100%			
455	135			30	36			RQD=7 Rec=60%			
450	140			31	57	Interbedded, slightly weathered, medium bedded to thin bedded, gray, fine grained to medium grained, very soft to soft SHALE and hard LIMESTONE, dip of 35 to 60 degrees, (RQD from 67 to 97 %), Silurian [Wills Creek Shale and Bloomsburg Formation]	Shale and Limestone	RQD=67 Rec=95%			
445	145			32	60			RQD=97 Rec=100%			
440	150			33	60			RQD=95 Rec=100%			
435	155			34	60	Interbedded, slightly weathered, medium bedded to thin bedded, green, red, gray, fine grained to medium grained, very soft to soft SHALE and hard LIMESTONE, dip of 35 to 45 degrees, (RQD from 97 to 100 %), Silurian [Wills Creek Shale and Bloomsburg Formation]	Shale and Limestone	RQD=100 Rec=100%			
430	160			35	60			RQD=97 Rec=100%			
425	165			36	57	Interbedded, slightly weathered, medium bedded to thin bedded, dark gray, fine grained to medium grained, very soft to soft SHALE and hard LIMESTONE, dip of 35 to 70 degrees, (RQD from 30 to 100 %), Silurian [Wills Creek Shale and Bloomsburg Formation]	Shale and Limestone	RQD=92 Rec=95%			
420	170			37	60			RQD=75 Rec=100%			
415	175			38	60			RQD=77 Rec=100%			
	180			39	60			RQD=83			

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COMPLETION DEPTH 277.0 ft
BENCHMARK: N/A
ELEVATION: 591 ft
LATITUDE: 39.686351°
LONGITUDE: 78.20148°
STATION: N/A **OFFSET:** N/A
REMARKS:

DRILL COMPANY: Connelly Drilling, Inc.
DRILLER: Kevin Kersh **LOGGED BY** Gunner Ingram
DRILL RIG: CME 550 ATV
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: 2-in SS1.874-in Core Standard
HAMMER TYPE: Automatic
EFFICIENCY N/A
REVIEWED BY: Lubomir Peytchev

BORING GO-3R

Water		
▽ While Drilling	Dry	feet
▼ Upon Completion	Dry	feet
▽ Delay	N/A	feet

BORING LOCATION:

Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS) RQD & Recovery % (NX)	Moisture, %	STANDARD PENETRATION TEST DATA N in blows/ft ◎ × Moisture ◪ PL ✚ LL 0 25 50 STRENGTH, tsf ▲ Qu ✳ Qp 0 2.0 4.0	Additional Remarks
410	180					Interbedded, slightly weathered, medium bedded to thin bedded, dark gray, fine grained to medium grained, very soft to soft SHALE and hard LIMESTONE, dip of 35 to 70 degrees, (RQD from 30 to 100 %), Silurian [Wills Creek Shale and Bloomsburg Formation]	Shale and Limestone	Rec=100%			
405	185			40	44			RQD=30 Rec=73%			
400	190			41	60			RQD=93 Rec=100%			
395	195			42	60			RQD=100 Rec=100%			
390	200			43	60			RQD=97 Rec=100%			
385	205			44	60			RQD=98 Rec=100%			
380	210			45	60			RQD=100 Rec=100%			
375	215			46	56			RQD=92 Rec=93%			
370	220			47	56	Interbedded, slightly weathered, medium bedded to thin bedded, dark gray, fine grained to medium grained, very soft to soft SHALE and hard LIMESTONE, dip of 45 to 65 degrees, (RQD from 50 to 100 %), Silurian [Wills Creek Shale and Bloomsburg Formation]	Shale and Limestone	RQD=86 Rec=93%			
365	225			48	60			RQD=95 Rec=100%			
360	230			49	60			RQD=95 Rec=100%			
355	235			50	60			RQD=100 Rec=100%			
	240			51	60			RQD=95			

Continued Next Page

Professional Service Industries, Inc.
2930 Eskridge Rd
Fairfax, VA 22031
Telephone: (703) 698-9300

PROJECT NO.: 0512713-1
PROJECT: 6493-Eastern Panhandle Expansion
LOCATION: Potomac River Crossing
Washington County
Hancock, MD

The stratification lines represent approximate boundaries. The transition may be gradual.

DATE STARTED: 12/15/16
DATE COMPLETED: 12/21/16
COMPLETION DEPTH 277.0 ft
BENCHMARK: N/A
ELEVATION: 591 ft
LATITUDE: 39.686351°
LONGITUDE: 78.20148°
STATION: N/A **OFFSET:** N/A
REMARKS:

DRILL COMPANY: Connelly Drilling, Inc.
DRILLER: Kevin Kersh **LOGGED BY** Gunner Ingram
DRILL RIG: CME 550 ATV
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: 2-in SS1.874-in Core Standard
HAMMER TYPE: Automatic
EFFICIENCY N/A
REVIEWED BY: Lubomir Peytchev

BORING GO-3R

Water		
∇ While Drilling	Dry	feet
▼ Upon Completion	Dry	feet
∇ Delay	N/A	feet

BORING LOCATION:

Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS) RQD & Recovery % (NX)	Moisture, %	STANDARD PENETRATION TEST DATA N in blows/ft ◎ × Moisture ◪ PL ✚ LL 0 25 50 STRENGTH, tsf ▲ Qu ✳ Qp 0 2.0 4.0	Additional Remarks
350	240					Interbedded, slightly weathered, medium bedded to thin bedded, dark gray, fine grained to medium grained, very soft to soft SHALE and hard LIMESTONE, dip of 45 to 65 degrees, (RQD from 50 to 100 %), Silurian [Wills Creek Shale and Bloomsburg Formation]		Rec=100%			
345	245			52	60			RQD=90 Rec=100%			
340	250			53	60			RQD=97 Rec=100%			
335	255			54	52			RQD=100 Rec=87%			
330	260			55	58		Shale and Limestone	RQD=50 Rec=97%			
325	265			56	60			RQD=80 Rec=100%			
320	270			57	60			RQD=78 Rec=100%			
315	275			58	60			RQD=83 Rec=100%			
						Bottom of test boring at 277 feet					

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PROJECT NO.: 0512713-1
PROJECT: 6493-Eastern Panhandle Expansion
LOCATION: Potomac River Crossing
Washington County
Hancock, MD

The stratification lines represent approximate boundaries. The transition may be gradual.

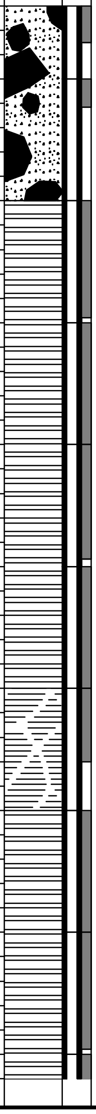
DATE STARTED: 1/17/17
DATE COMPLETED: 1/20/17
COMPLETION DEPTH 134.0 ft
BENCHMARK: N/A
ELEVATION: 402 ft
LATITUDE: 39.681887°
LONGITUDE: 78.197553°
STATION: N/A **OFFSET:** N/A
REMARKS:

DRILL COMPANY: Connelly Drilling, Inc.
DRILLER: Howie Roberts **LOGGED BY:** Rob Stickley
DRILL RIG: Diedrich D-50
DRILLING METHOD: Rock Coring
SAMPLING METHOD: 1.874-in Core Standard
HAMMER TYPE: N/A
EFFICIENCY N/A
REVIEWED BY: Lubomir Peytchev

BORING GO-7

Water		
▽	While Drilling	0 feet
▼	Upon Completion	0 feet
▽	Delay	N/A feet

BORING LOCATION:



Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	RQD & Recovery % (NX)	Moisture, %	STANDARD PENETRATION TEST DATA N in blows/ft ◎ × Moisture ◪ PL ✚ LL 0 25 50 STRENGTH, tsf ▲ Qu ✳ Qp 0 2.0 4.0	Additional Remarks
400	0 5					Potomac River, approximately 6 feet of WATER	Water				
395				1	18	Medium dense to very dense, wet, white, red, brown, gray and black **Sand, Gravel and Cobbles (Alluvium)**	Cobble Stones	RQD=0 Rec=50%			
390	10			2	14			RQD=0 Rec=23%			
385	15			3	58	Interbedded, slightly weathered, medium bedded to thin bedded, gray, dark gray, trace brown and white, fine grained to medium grained, very soft to soft SHALE and medium hard LIMESTONE, dip of 20 to 45 degrees, (RQD from 68% to 100 %), Devonian [Oriskany Sandstone and Helderberg Group]	Shale and Limestone	RQD=88 Rec=96%			
380	20			4	60			RQD=100 Rec=100%			
375	25			5	56			RQD=86 Rec=94%			
370	30			6	60			RQD=68 Rec=100%			
365	35			7	36	Interbedded, weathered, medium bedded to thin bedded, brown, gray, dark gray, trace white, fine grained to medium grained, very soft to soft SHALE and hard LIMESTONE, dip of 20 to 60 degrees, clay seams (RQD of 0 %), Devonian [Oriskany Sandstone and Helderberg Group]	Shale	RQD=0 Rec=60%			
360	40			8	60	Interbedded, slightly weathered, medium bedded to thin bedded, gray, dark gray, brown and white, fine grained to medium grained, very soft to soft SHALE and hard LIMESTONE, dip of 20 to 45 degrees, (RQD from 24% to 86 %), Devonian [Oriskany Sandstone and Helderberg Group]	Shale and Limestone	RQD=86 Rec=100%			
355	45			9	58			RQD=72 Rec=96%			
	50										

Continued Next Page

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2930 Eskridge Rd
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PROJECT NO.: 0512713-1
PROJECT: 6493-Eastern Panhandle Expansion
LOCATION: Potomac River Crossing
Washington County
Hancock, MD

DATE STARTED: 1/17/17
DATE COMPLETED: 1/20/17
COMPLETION DEPTH 134.0 ft
BENCHMARK: N/A
ELEVATION: 402 ft
LATITUDE: 39.681887°
LONGITUDE: 78.197553°
STATION: N/A OFFSET: N/A
REMARKS:

DRILL COMPANY: Connelly Drilling, Inc.
DRILLER: Howie Roberts LOGGED BY: Rob Stickley
DRILL RIG: Diedrich D-50
DRILLING METHOD: Rock Coring
SAMPLING METHOD: 1.874-in Core Standard
HAMMER TYPE: N/A
EFFICIENCY N/A
REVIEWED BY: Lubomir Peytchev

BORING GO-7

Water		
▽	While Drilling	0 feet
▼	Upon Completion	0 feet
▽	Delay	N/A feet

BORING LOCATION:

Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	RQD & Recovery % (NX)	Moisture, %	STANDARD PENETRATION TEST DATA N in blows/ft ◎ × Moisture ◪ PL ✚ LL 0 25 50 STRENGTH, tsf ▲ Qu ✳ Qp 0 2.0 4.0	Additional Remarks
350	50			10	60	Interbedded, slightly weathered, medium bedded to thin bedded, gray, dark gray, brown and white, fine grained to medium grained, very soft to soft SHALE and hard LIMESTONE, dip of 20 to 45 degrees, (RQD from 24% to 86 %), Devonian [Oriskany Sandstone and Helderberg Group]		RQD=24 Rec=100%			
345	55			11	50	Void approximately from 58.2' to 59'	Shale and Limestone	RQD=64 Rec=84%			
340	60			12	52			RQD=30 Rec=86%			
335	65			13	60	Interbedded, slightly weathered, medium bedded to thin bedded, gray, dark gray, trace brown and white, fine grained to medium grained, extremely soft to soft SHALE and hard LIMESTONE, dip of 20 to 50 degrees, (RQD from 24% to 92 %), Devonian [Oriskany Sandstone and Helderberg Group]		RQD=70 Rec=100%			
330	70			14	43	Clay seam approximately from 72.6' to 74'		RQD=60 Rec=72%			
325	75			15	56			RQD=60 Rec=94%			
320	80			16	60		Shale and Limestone	RQD=74 Rec=100%			
315	85			17	60			RQD=92 Rec=100%			
310	90			18	60			RQD=82 Rec=100%			
305	95			19	58			RQD=24 Rec=96%			
	100										

Continued Next Page

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PROJECT NO.: 0512713-1
PROJECT: 6493-Eastern Panhandle Expansion
LOCATION: Potomac River Crossing
Washington County
Hancock, MD

The stratification lines represent approximate boundaries. The transition may be gradual.

DATE STARTED: 1/17/17
DATE COMPLETED: 1/20/17
COMPLETION DEPTH 134.0 ft
BENCHMARK: N/A
ELEVATION: 402 ft
LATITUDE: 39.681887°
LONGITUDE: 78.197553°
STATION: N/A **OFFSET:** N/A
REMARKS:

DRILL COMPANY: Connelly Drilling, Inc.
DRILLER: Howie Roberts **LOGGED BY:** Rob Stickley
DRILL RIG: Diedrich D-50
DRILLING METHOD: Rock Coring
SAMPLING METHOD: 1.874-in Core Standard
HAMMER TYPE: N/A
EFFICIENCY N/A
REVIEWED BY: Lubomir Peytchev

BORING GO-7

Water		
∇	While Drilling	0 feet
▼	Upon Completion	0 feet
∇	Delay	N/A feet

BORING LOCATION:

Elevation (feet)	Depth, (feet)	Graphic Log	Sample Type	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	RQD & Recovery % (NX)	Moisture, %	STANDARD PENETRATION TEST DATA N in blows/ft ◎ × Moisture ◪ PL ✚ LL 0 25 50 STRENGTH, tsf ▲ Qu ✳ Qp 0 2.0 4.0	Additional Remarks
300	100			20	58		Shale and Limestone	RQD=50 Rec=96%			
295	105			21	48	Interbedded, weathered, medium bedded to thin bedded, brown, dark gray, gray, dark brown, trace white, fine grained to medium grained, very soft to soft SHALE and hard LIMESTONE, dip of 20 to 45 degrees, clay seams (RQD of 0 %), Devonian [Oriskany Sandstone and Helderberg Group] Clay seam approximately from 107' to 108.5'	Shale	RQD=0 Rec=80%			
290	110			22	60	Interbedded, slightly weathered, medium bedded to thin bedded, gray, dark gray, trace brown and white, fine grained to medium grained, very soft to soft SHALE and moderately hard LIMESTONE, dip of 40 to 60 degrees, (RQD from 28% to 72 %), Devonian [Oriskany Sandstone and Helderberg Group]	Shale and Limestone	RQD=34 Rec=100%			
285	115			23	60			RQD=46 Rec=100%			
280	120			24	60			RQD=28 Rec=100%			
275	125			25	60			RQD=44 Rec=100%			
270	130			26	60			RQD=72 Rec=100%			
						Bottom of test boring at 134 feet					

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GENERAL NOTES

SAMPLE IDENTIFICATION

The Unified Soil Classification System (USCS), AASHTO 1988 and ASTM designations D2487 and D-2488 are used to identify the encountered materials unless otherwise noted. Coarse-grained soils are defined as having more than 50% of their dry weight retained on a #200 sieve (0.075mm); they are described as: boulders, cobbles, gravel or sand. Fine-grained soils have less than 50% of their dry weight retained on a #200 sieve; they are defined as silts or clay depending on their Atterberg Limit attributes. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size.

DRILLING AND SAMPLING SYMBOLS

SFA: Solid Flight Auger - typically 4" diameter flights, except where noted.
HSA: Hollow Stem Auger - typically 3¼" or 4¼ I.D. openings, except where noted.
M.R.: Mud Rotary - Uses a rotary head with Bentonite or Polymer Slurry
R.C.: Diamond Bit Core Sampler
H.A.: Hand Auger
P.A.: Power Auger - Handheld motorized auger

☒ SS: Split-Spoon - 1 3/8" I.D., 2" O.D., except where noted.
▮ ST: Shelby Tube - 3" O.D., except where noted.
▯ RC: Rock Core
⬇ TC: Texas Cone
✋ BS: Bulk Sample
▦ PM: Pressuremeter
CPT-U: Cone Penetrometer Testing with Pore-Pressure Readings

SOIL PROPERTY SYMBOLS

N: Standard "N" penetration: Blows per foot of a 140 pound hammer falling 30 inches on a 2-inch O.D. Split-Spoon.
N_{60}: A "N" penetration value corrected to an equivalent 60% hammer energy transfer efficiency (ETR)
Q_u: Unconfined compressive strength, TSF
Q_p: Pocket penetrometer value, unconfined compressive strength, TSF
w%: Moisture/water content, %
LL: Liquid Limit, %
PL: Plastic Limit, %
PI: Plasticity Index = (LL-PL),%
DD: Dry unit weight, pcf
▼, ▽, ▼: Apparent groundwater level at time noted

RELATIVE DENSITY OF COARSE-GRAINED SOILS

Relative Density	N - Blows/foot
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	50 - 80
Extremely Dense	80+

ANGULARITY OF COARSE-GRAINED PARTICLES

Description	Criteria
Angular:	Particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular:	Particles are similar to angular description, but have rounded edges
Subrounded:	Particles have nearly plane sides, but have well-rounded corners and edges
Rounded:	Particles have smoothly curved sides and no edges

GRAIN-SIZE TERMINOLOGY

Component	Size Range
Boulders:	Over 300 mm (>12 in.)
Cobbles:	75 mm to 300 mm (3 in. to 12 in.)
Coarse-Grained Gravel:	19 mm to 75 mm (¾ in. to 3 in.)
Fine-Grained Gravel:	4.75 mm to 19 mm (No.4 to ¾ in.)
Coarse-Grained Sand:	2 mm to 4.75 mm (No.10 to No.4)
Medium-Grained Sand:	0.42 mm to 2 mm (No.40 to No.10)
Fine-Grained Sand:	0.075 mm to 0.42 mm (No. 200 to No.40)
Silt:	0.005 mm to 0.075 mm
Clay:	<0.005 mm

PARTICLE SHAPE

Description	Criteria
Flat:	Particles with width/thickness ratio > 3
Elongated:	Particles with length/width ratio > 3
Flat & Elongated:	Particles meet criteria for both flat and elongated

RELATIVE PROPORTIONS OF FINES

Descriptive Term	% Dry Weight
Trace:	< 5%
With:	5% to 12%
Modifier:	>12%

GENERAL NOTES

(Continued)

CONSISTENCY OF FINE-GRAINED SOILS

Q_U - TSF	N - Blows/foot	Consistency
0 - 0.25	0 - 2	Very Soft
0.25 - 0.50	2 - 4	Soft
0.50 - 1.00	4 - 8	Firm (Medium Stiff)
1.00 - 2.00	8 - 15	Stiff
2.00 - 4.00	15 - 30	Very Stiff
4.00 - 8.00	30 - 50	Hard
8.00+	50+	Very Hard

MOISTURE CONDITION DESCRIPTION

Description	Criteria
Dry:	Absence of moisture, dusty, dry to the touch
Moist:	Damp but no visible water
Wet:	Visible free water, usually soil is below water table

RELATIVE PROPORTIONS OF SAND AND GRAVEL

Descriptive Term	% Dry Weight
Trace:	< 15%
With:	15% to 30%
Modifier:	>30%

STRUCTURE DESCRIPTION

Description	Criteria
Stratified:	Alternating layers of varying material or color with layers at least ¼-inch (6 mm) thick
Laminated:	Alternating layers of varying material or color with layers less than ¼-inch (6 mm) thick
Fissured:	Breaks along definite planes of fracture with little resistance to fracturing
Slickensided:	Fracture planes appear polished or glossy, sometimes striated

Description	Criteria
Blocky:	Cohesive soil that can be broken down into small angular lumps which resist further breakdown
Lensed:	Inclusion of small pockets of different soils
Layer:	Inclusion greater than 3 inches thick (75 mm)
Seam:	Inclusion 1/8-inch to 3 inches (3 to 75 mm) thick extending through the sample
Parting:	Inclusion less than 1/8-inch (3 mm) thick

SCALE OF RELATIVE ROCK HARDNESS

Q_U - TSF	Consistency
2.5 - 10	Extremely Soft
10 - 50	Very Soft
50 - 250	Soft
250 - 525	Medium Hard
525 - 1,050	Moderately Hard
1,050 - 2,600	Hard
>2,600	Very Hard

ROCK BEDDING THICKNESSES

Description	Criteria
Very Thick Bedded	Greater than 3-foot (>1.0 m)
Thick Bedded	1-foot to 3-foot (0.3 m to 1.0 m)
Medium Bedded	4-inch to 1-foot (0.1 m to 0.3 m)
Thin Bedded	1¼-inch to 4-inch (30 mm to 100 mm)
Very Thin Bedded	½-inch to 1¼-inch (10 mm to 30 mm)
Thickly Laminated	1/8-inch to ½-inch (3 mm to 10 mm)
Thinly Laminated	1/8-inch or less "paper thin" (<3 mm)

ROCK VOIDS

Voids	Void Diameter
Pit	<6 mm (<0.25 in)
Vug	6 mm to 50 mm (0.25 in to 2 in)
Cavity	50 mm to 600 mm (2 in to 24 in)
Cave	>600 mm (>24 in)

GRAIN-SIZED TERMINOLOGY

(Typically Sedimentary Rock)

Component	Size Range
Very Coarse Grained	>4.76 mm
Coarse Grained	2.0 mm - 4.76 mm
Medium Grained	0.42 mm - 2.0 mm
Fine Grained	0.075 mm - 0.42 mm
Very Fine Grained	<0.075 mm

ROCK QUALITY DESCRIPTION

Rock Mass Description	RQD Value
Excellent	90 -100
Good	75 - 90
Fair	50 - 75
Poor	25 -50
Very Poor	Less than 25

DEGREE OF WEATHERING

Slightly Weathered:	Rock generally fresh, joints stained and discoloration extends into rock up to 25 mm (1 in), open joints may contain clay, core rings under hammer impact.
Weathered:	Rock mass is decomposed 50% or less, significant portions of the rock show discoloration and weathering effects, cores cannot be broken by hand or scraped by knife.
Highly Weathered:	Rock mass is more than 50% decomposed, complete discoloration of rock fabric, core may be extremely broken and gives clunk sound when struck by hammer, may be shaved with a knife.

SOIL CLASSIFICATION CHART

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)	●	GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
			○	GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)	○	GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
			▨	GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)	⁘	SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
			⁚	SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)	⁚	SM	SILTY SANDS, SAND - SILT MIXTURES
			▨	SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	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, SILTY CLAYS, LEAN 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 SAND OR SILTY SOILS
			▨	CH	INORGANIC CLAYS OF HIGH PLASTICITY
			≈	OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			⁂	PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

APPENDIX D: LABORATORY TESTING RESULTS

Laboratory Summary Sheet

Sheet 1 of 1

Borehole	Approx. Depth	Liquid Limit	Plastic Limit	Plasticity Index	Qu (tsf)	%<#200 Sieve	Est. Specific Gravity	Water Content (%)	Dry Density (pcf)	Satur-ation (%)	Void Ratio
GO-3R	1							10			
GO-3R	3	26	17	9		21.3%		7			
GO-3R	5.5							6			
GO-3R	9							6			
GO-3R	13.7					27.3%		4			
GO-3R	18.6							9			

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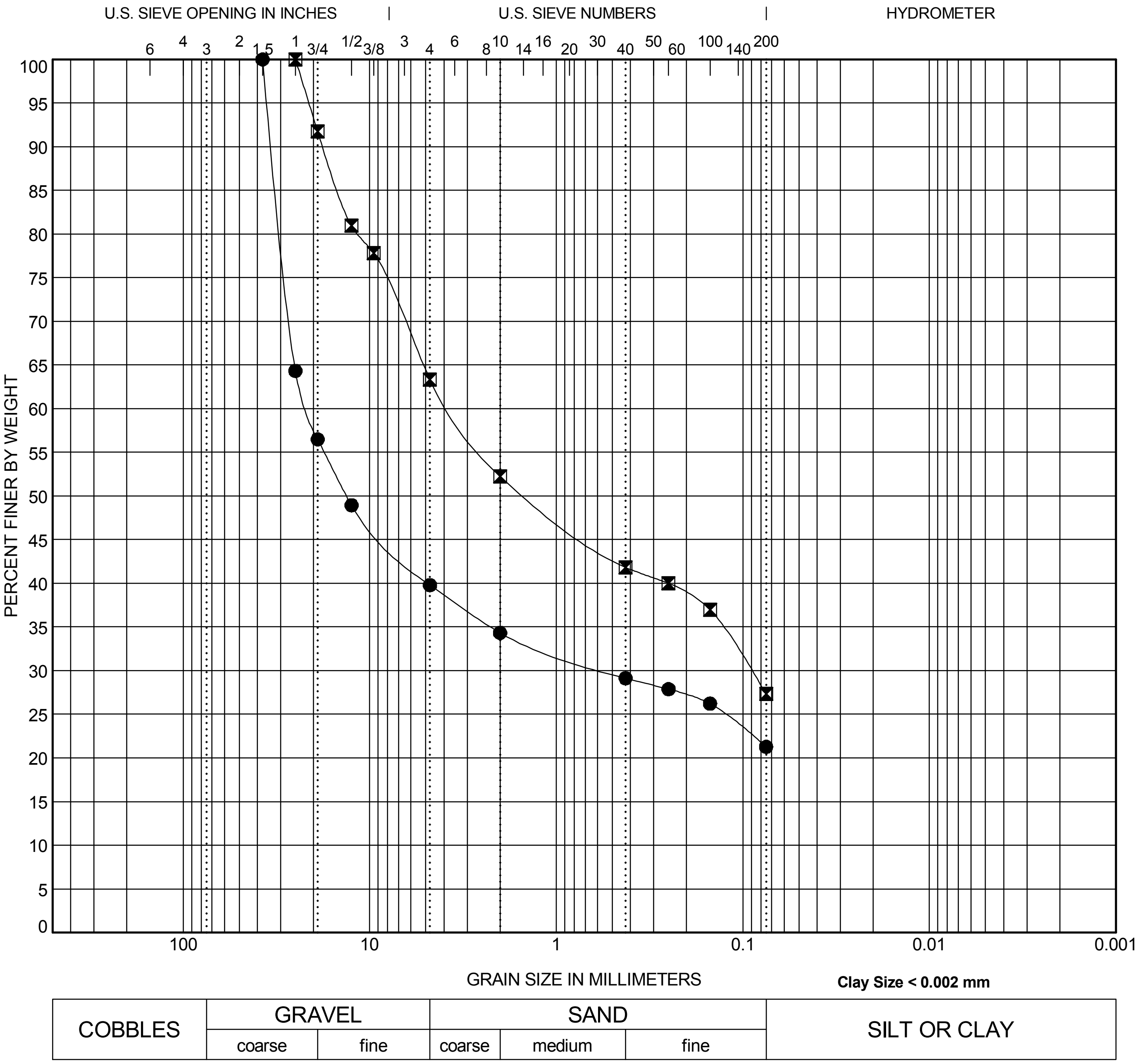
Summary of Laboratory Results

PSI Job No.: 0512713-1
Project: 6493-Eastern Panhandle Expansion
Location: Potomac River Crossing
Washington County
Hancock, MD



Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	**GO-3R**	**3.0**	Silty GRAVEL (USCS GM)with sand	**26**	**17**	**9**		
⧗	**GO-3R**	**13.7**	Silty GRAVEL (USGS GM)with sand					

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**GO-3R**	**3.0**	**37.5**	**21.485**	**0.551**		**60.2**	**18.5**	**21.3**	
⧗	**GO-3R**	**13.7**	**25**	**3.659**	**0.091**		**36.7**	**36.0**	**27.3**	

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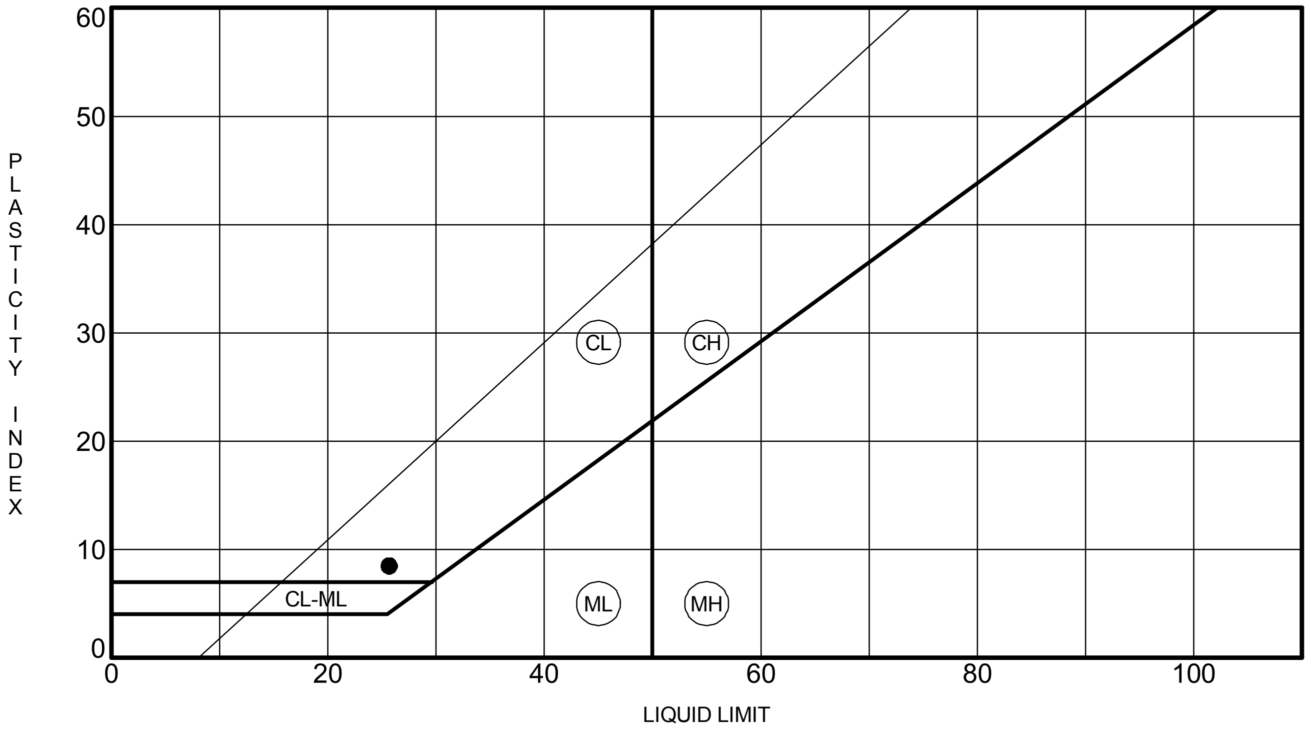
GRAIN SIZE DISTRIBUTION

Project: 6493-Eastern Panhandle Expansion
PSI Job No.: 0512713-1
Location: Potomac River Crossing
Washington County



	Boring	Depth (ft)	LL	PL	PI	Fines	Classification (*Visual)
●	**GO-3R**	**3.0**	**26**	**17**	**9**	**21.3**	Lean CLAY (USGS CL)

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ATTERBERG LIMIT RESULTS

PSI Job No.: 0512713-1
Project: 6493-Eastern Panhandle Expansion
Location: Potomac River Crossing
Washington County

PARTICLE-SIZE ANALYSIS OF SOILS - ASTM D422

Client	Professional Service Industries, Inc. (PSI)	Boring	GO-3R
Client Project	Proposed Eastern Panhandle Expansion I-68 Crossing V	Depth	0.0'-2.0'
Project No.	38212	Sample	ST-1
		Lab Sample	38212001

Sample Color: **DARK REDDISH BROWN**
USCS Group Name: **SILTY GRAVEL WITH SAND**
USCS Group Symbol: **GM** USDA: **NA** AASHTO: **A-2-4 (0)**

MECHANICAL SIEVE

Total Sample	
Total Sample Wet Wt, gm (-3")	930
Sample Split on Sieve	No. 4
Coarse Washed Dry Sample, gm	327
Wet Wt Passing Split, gm	603
Dry Wt. Passing Split, gm	497
Total Sample Dry Wt, gm	824

Split Sample - Passing No. 4	
Tare No.	2015
Tare + WS., gm	467.16
Tare + DS., gm	411.46
Tare, gm	151.26
Water Content of Split Sample	**21.4%**
Wt. of DS., gm	260.20
Wt. of +#200 Sample, gm	120.86

Sieve Size	**Nominal Opening, mm**	**Dry Wt, gm**	**Split Normalized % Retained**	**Split Normalized % Finer**	**Project Specifications**
3"	75	0	0.0%	100.0%	
2-1/2"	63	0	0.0%	100.0%	
2"	50	0	0.0%	100.0%	
1-1/2"	37.5	140.22	17.0%	83.0%	
1"	25	0	0.0%	83.0%	
3/4"	19	17.86	2.2%	80.8%	
1/2"	12.5	39.17	4.8%	76.1%	
3/8"	9.5	44.08	5.3%	70.7%	
No. 4	4.75	85.8	10.4%	60.3%	
No. 10	2	33.47	7.8%	52.5%	
No. 20	0.85	22.94	5.3%	47.2%	
No. 40	0.425	13.64	3.2%	44.1%	
No. 60	0.25	9.81	2.3%	41.8%	
No. 140	0.106	23.93	5.5%	36.2%	
No. 200	0.075	17.07	4.0%	32.3%	

USCS SOIL CLASSIFICATION

Corrected For 100% Passing a 3" Sieve

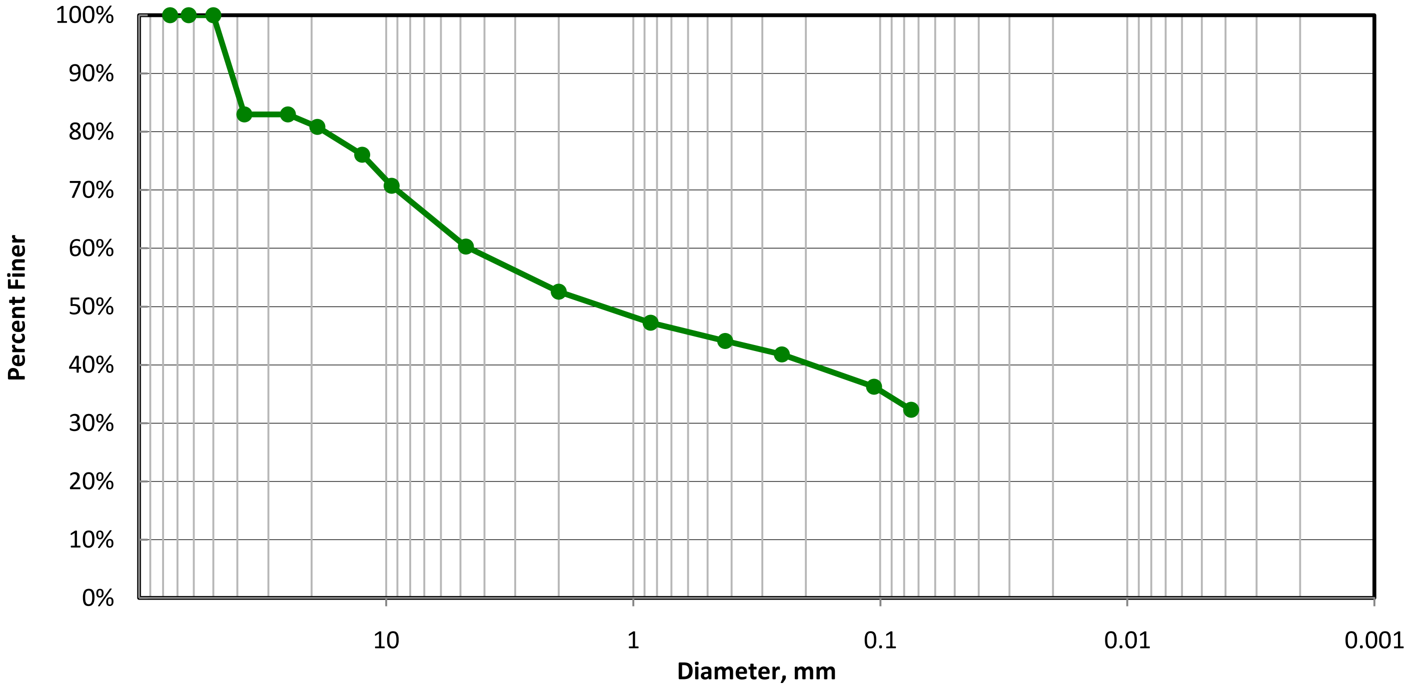
% Gravel (-3" & +#4)	**39.7**	Silt=NA Clay=NA	
Coarse=19.2; Fine=20.5		D60, mm	NA
% Sand (-#4 & +#200)	**28.0**	D30, mm	NA
Coarse=7.8; Medium=8.5; Fine=11.8		D10, mm	NA
% Fines (-#200)	**32.3**	Cc	NA
% Plus #200 (-3")	**67.7**	Cu	NA

USCS Description

SILTY GRAVEL WITH SAND

USCS Group Symbol	**Atterberg Limits Group Symbol**
GM	**NP - NON PLASTIC**

Auxiliary Information	Wt Ret, gm	% Retained	% Finer
12" Sieve - 300 mm	0	0.0	100.0
6" Sieve - 150 mm	0	0.0	100.0
3" Sieve - 75 mm	0	0.0	100.0



Input Validation MAK Reviewed By: ALO Date Tested 1/6/2017

LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS
ASTM D 4318

Client	Professional Service Industries, Inc. (PSI)	Boring	GO-3R
Client Project	Proposed Eastern Panhandle Expansion I-68 Crossing V	Depth	0.0'-2.0'
Project No.	38212	Sample	ST-1
		Lab Sample	38212001

Soil Description: DARK REDDISH BROWN NON PLASTIC MATERIAL
(-#40 Fraction)

AS-RECEIVED W.C.

Tare Number	2015
Wt. Tare & WS, gm	467.16
Wt. Tare & DS, gm	411.46
Wt. Tare, gm	151.26
Water Content, %	21.4

SAMPLE SUMMARY

C

Liquid Limit (LL), %	NA
Plastic Limit (PL), %	NA
Plasticity Index (PI)	NA
USCS Group Symbol (*-#40 Fraction*)	NP
USCS Group Name (*-#40 Fraction*)	NON PLASTIC
Sample Color:	DARK REDDISH BROWN

PLASTIC LIMIT

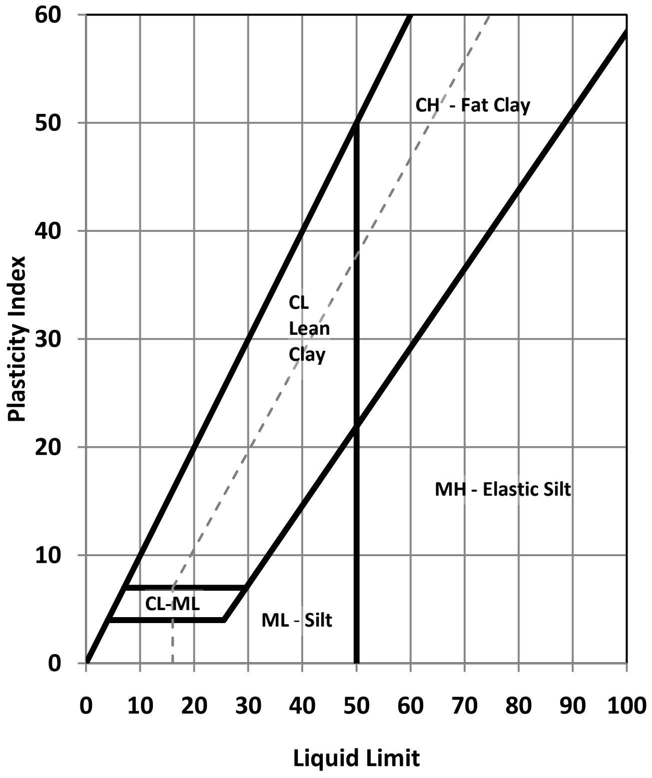
Points Run	0 Non-Plastic
Tare Number	
Wt. Tare & WS, gm	
Wt. Tare & DS, gm	
Wt. Tare, gm	
Water Content, %	

LIQUID LIMIT

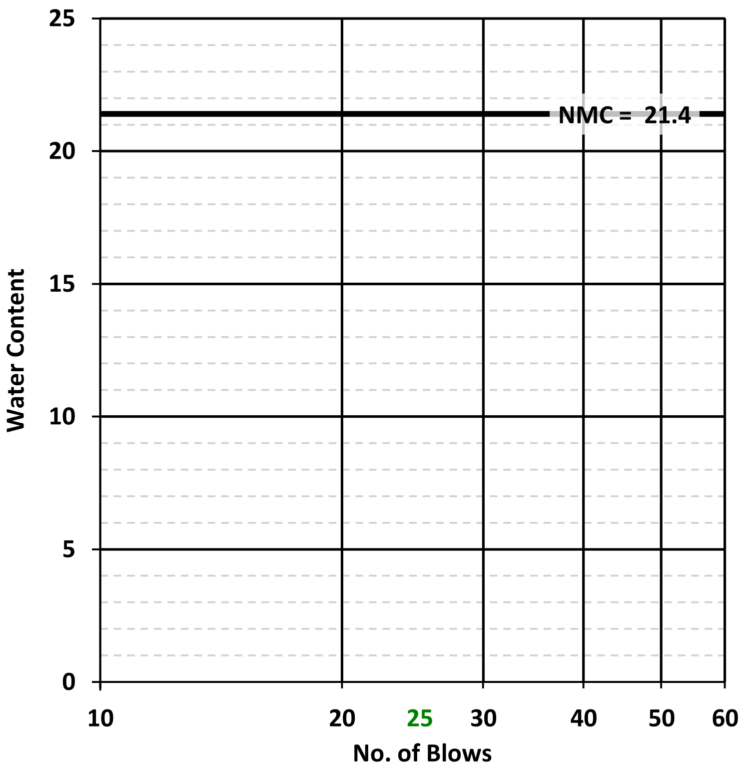
0 Non-Plastic

# of Blows

PLASTICITY CHART



FLOW CURVE



Input Validation: MAK Reviewed By: ALO Date Tested: 1/7/2017

DENSITY DETERMINATIONS

Client	Professional Service Industries, Inc. (PSI)
Project	Proposed Eastern Panhandle Expansion I-68 Crossing Wash Co MD
Project No.	38212

Boring Number	GO-3R									
Depth	0.0'-2.0'									
Sample	ST-1									
Lab Sample No.	38212001									

Water Contents

Tare Number	2015									
Wt. Tare & WS, gm	467.16									
Wt. Tare & DS, gm	411.46									
Wt. Tare, gm	151.26									
Water Content, %	**21.4%**									

Direct Measurement Method

Wt. of Wet Soil, gm	930									
Length 1, in	5.864									
Length 2, in	5.865									
Length 3, in	5.868									
Top Diameter, in	2.882									
Middle Diameter, in	2.879									
Bottom Diameter, in	2.88									
Sample Volume, cc	626.32									
Water Content ,%	21.4%									
Unit Wet Wt., gm/cc	1.48									
Unit Wet Wt., pcf	92.7									
Unit Dry Wt., pcf	**76.3**									
Unit Dry Wt., gm/cc	1.22									
Specific Gravity, Assumed	2.7									
Void Ratio,e	1.21									
Porosity, n	0.55									
Saturation, %	47.9%									

Input Validation: ALO

Reviewed By: ALO

Date: 1/9/2017

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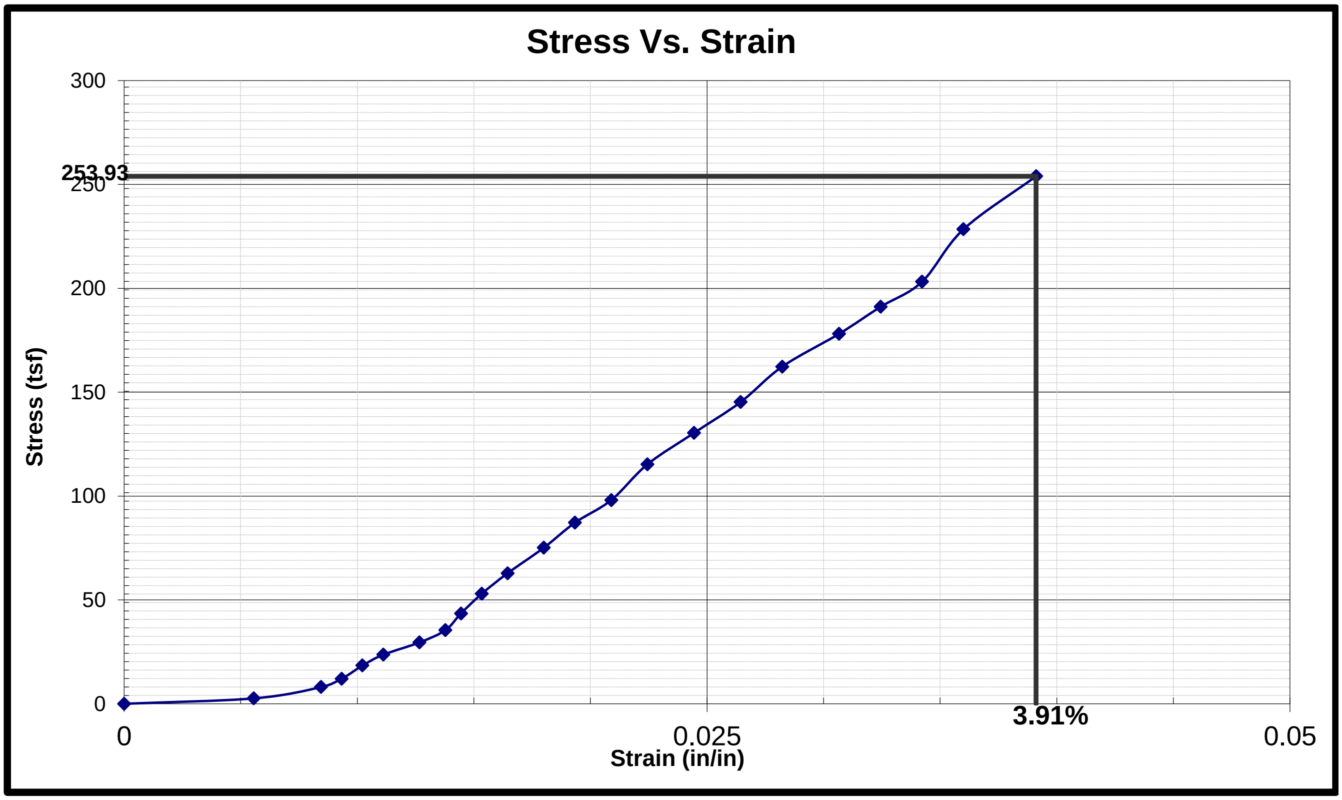
Unconfined Compression Test for Rock Cores

Date:	1/5/2017	Project:	Potomac River Crossing
Tested by:	Yamma Ershadi	Project No:	512713-2
Client:	Ensite USA	Location:	Washington County, MD

Average Initial Height (in):	4.50	Boring :	GO-3R
Average Initial Diameter (in):	1.97	Sample Number:	RC-11
Water Content %:	0.5	Sample Depth:	40.5-41.0
Wet Density (pcf):	164.7	Rock Description:	Medium hard red Limestone
Dry Density (pcf):	163.9		
LL - PL = PI:	NP - NP = NP		



Unconfined Compression Strength q_u (tsf):	**254**
Height to Diameter Ratio:	**2.3**
Percent Strain at Failure :	**3.91%**
Average Rate of Strain to Failure (% Strain/min):	**0.36%**

This test was performed according to ASTM D7012 - 14.
Compressive Strength of Intact Rock Core Specimens.

Failure Picture

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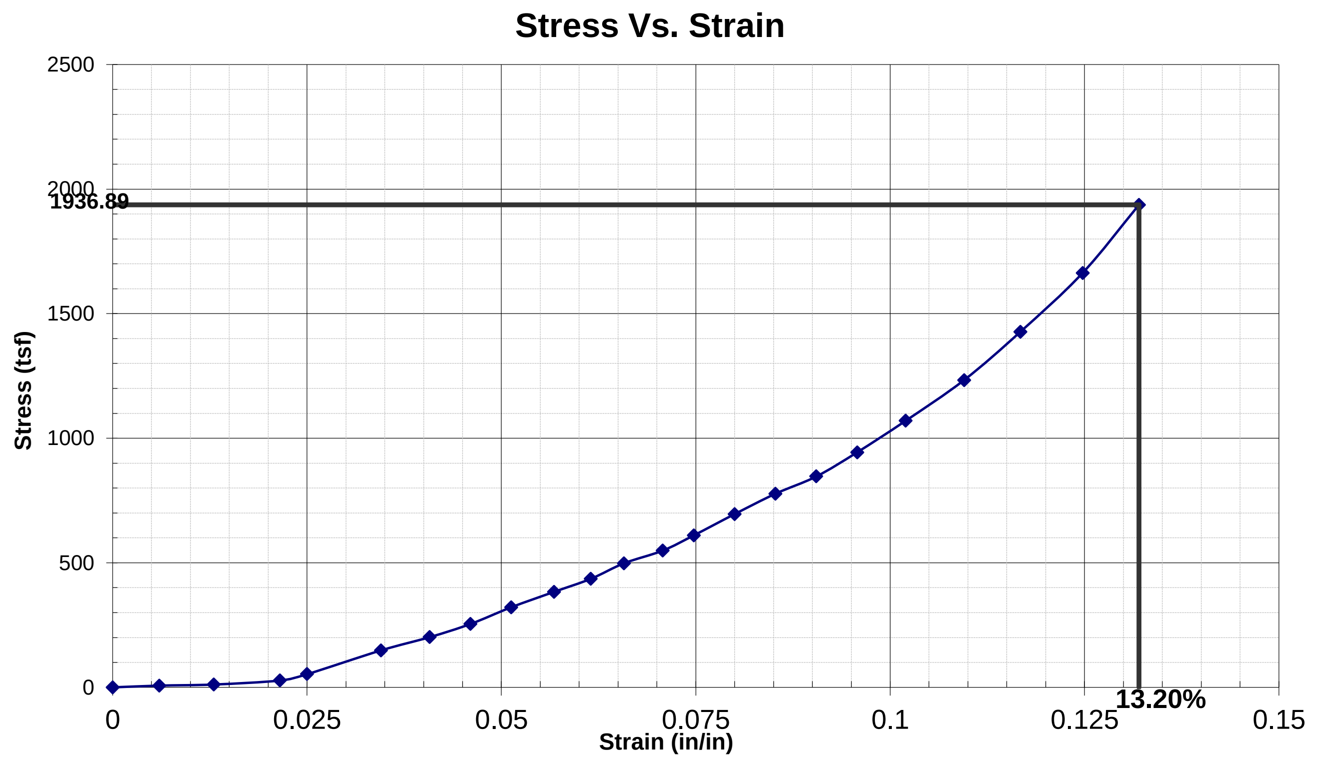
Unconfined Compression Test for Rock Cores

Date:	1/5/2017	Project:	Potomac River Crossing
Tested by:	Yamma Ershadi	Project No:	512713-2
Client:	Ensite USA	Location:	Washington County, MD

Average Initial Height (in):	4.00	Boring :	GO-3R
Average Initial Diameter (in):	1.95	Sample Number:	RC-23
Water Content %:	0.7	Sample Depth:	101.0-101.5
Wet Density (pcf):	175.0		
Dry Density (pcf):	173.8	Rock Description:	`Hard gray Limestone`
LL - PL = PI:	NP - NP = NP		



Unconfined Compression Strength q_u (tsf):	**1937**
Height to Diameter Ratio:	**2.1**
Percent Strain at Failure :	**13.20%**
Average Rate of Strain to Failure (% Strain/min):	**1.20%**

This test was performed according to ASTM D7012 - 14.
Compressive Strength of Intact Rock Core Specimens.

Failure Picture

Information To Build On
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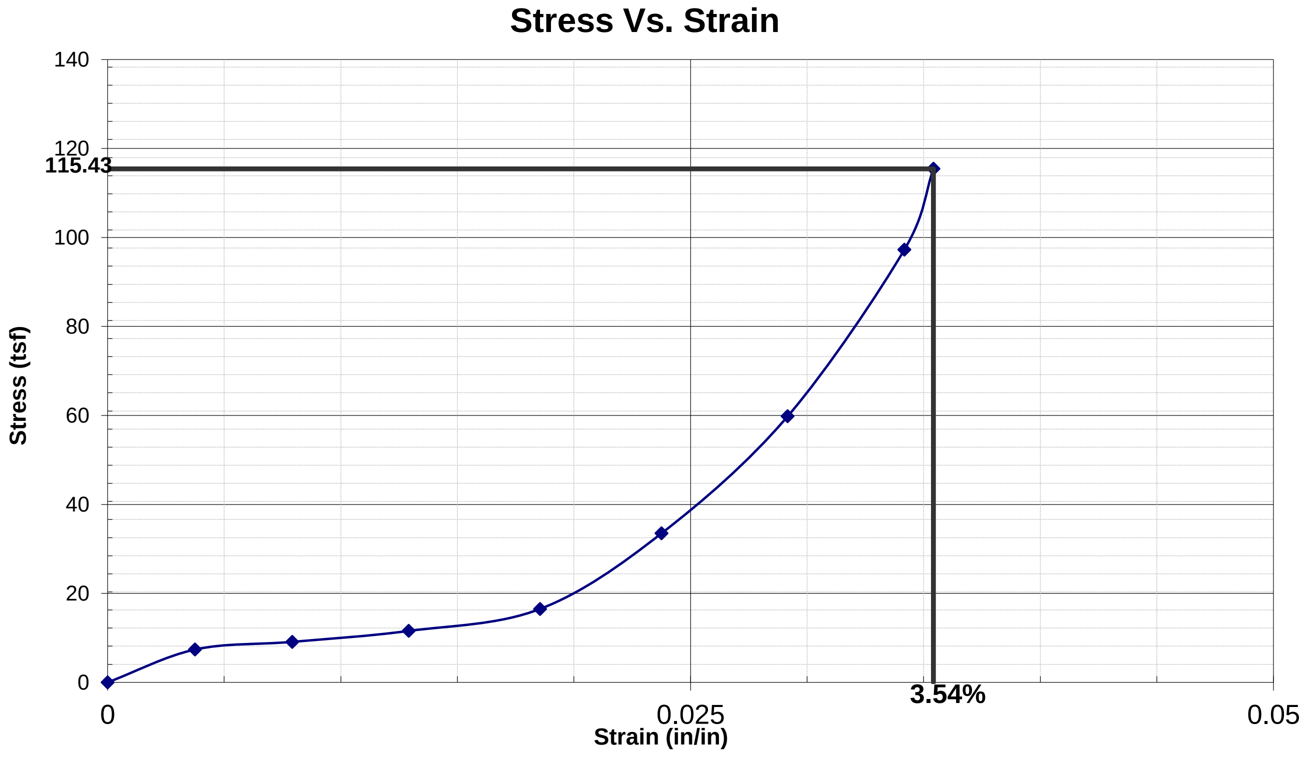
Unconfined Compression Test for Rock Cores

Date:	1/5/2017	Project:	Potomac River Crossing
Tested by:	Yamma Ershadi	Project No:	512713-2
Client:	Ensite USA	Location:	Washington County, MD

Average Initial Height (in):	4.80	Boring :	GO-3R
Average Initial Diameter (in):	1.93	Sample Number:	RC-37
Water Content %:	0.3	Sample Depth:	168.5-169
Wet Density (pcf):	176.1	Rock Description:	Soft gray Shale
Dry Density (pcf):	175.6		
LL - PL = PI:	NP - NP = NP		



Unconfined Compression Strength q_u (tsf):	**115**
Height to Diameter Ratio:	**2.5**
Percent Strain at Failure :	**3.54%**
Average Rate of Strain to Failure (% Strain/min):	**0.89%**

This test was performed according to ASTM D7012 - 14.
Compressive Strength of Intact Rock Core Specimens.

Failure Picture

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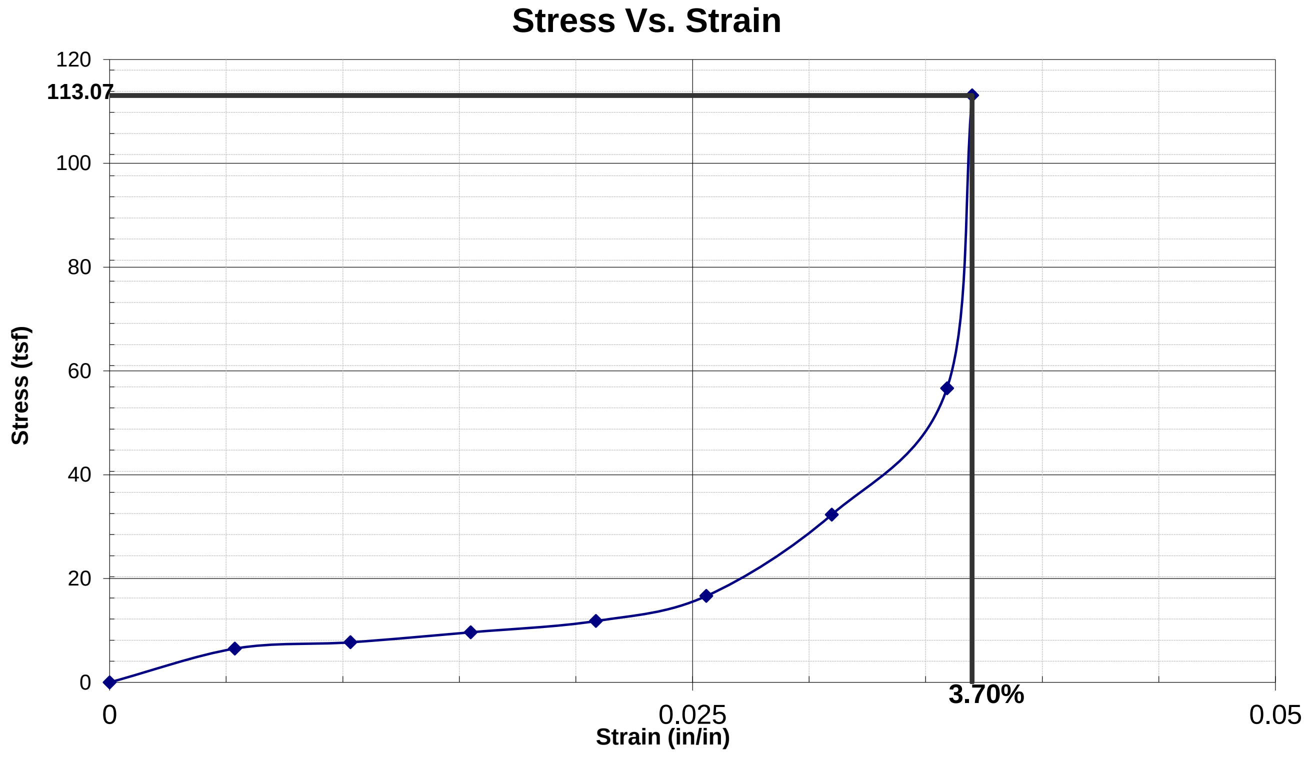
Unconfined Compression Test for Rock Cores

Date: 1/5/2017
Tested by: Yamma Ershadi
Client: Ensite USA

Project: Potomac River Crossing
Project No: 512713-2
Location: Washington County, MD

Average Initial Height (in):	4.65
Average Initial Diameter (in):	1.95
Water Content %:	0.4
Wet Density (pcf):	172.6
Dry Density (pcf):	172.0
LL - PL = PI:	NP - NP = NP

Boring : GO-3R
Sample Number: RC-42
Sample Depth: 192.5-193.0
Rock Description: Soft Gray Shale



Unconfined Compression Strength q_u (tsf):	**113**
Height to Diameter Ratio:	**2.4**
Percent Strain at Failure :	**3.70%**
Average Rate of Strain to Failure (% Strain/min):	**0.92%**

This test was performed according to ASTM D7012 - 14.
Compressive Strength of Intact Rock Core Specimens.

Failure Picture

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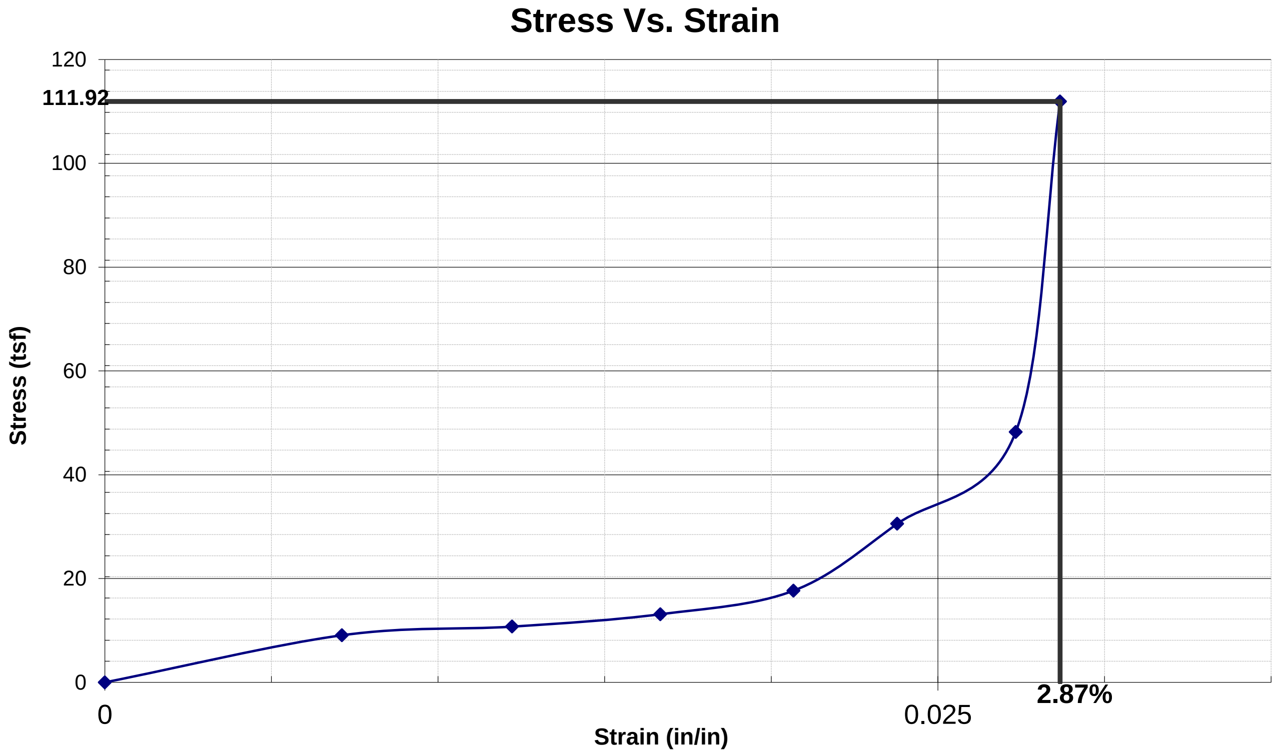
Unconfined Compression Test for Rock Cores

Date:	1/5/2017	Project:	Potomac River Crossing
Tested by:	Yamma Ershadi	Project No:	512713-2
Client:	Ensite USA	Location:	Washington County, MD

Average Initial Height (in):	4.50	Boring :	GO-3R
Average Initial Diameter (in):	1.96	Sample Number:	RC-47
Water Content %:	0.4	Sample Depth:	221.0-221.5
Wet Density (pcf):	173.5	Rock Description:	Soft gray Shale
Dry Density (pcf):	172.7		
LL - PL = PI:	NP - NP = NP		



Unconfined Compression Strength q_u (tsf):	**112**
Height to Diameter Ratio:	**2.3**
Percent Strain at Failure :	**2.87%**
Average Rate of Strain to Failure (% Strain/min):	**0.82%**

This test was performed according to ASTM D7012 - 14.
Compressive Strength of Intact Rock Core Specimens.

Failure Picture

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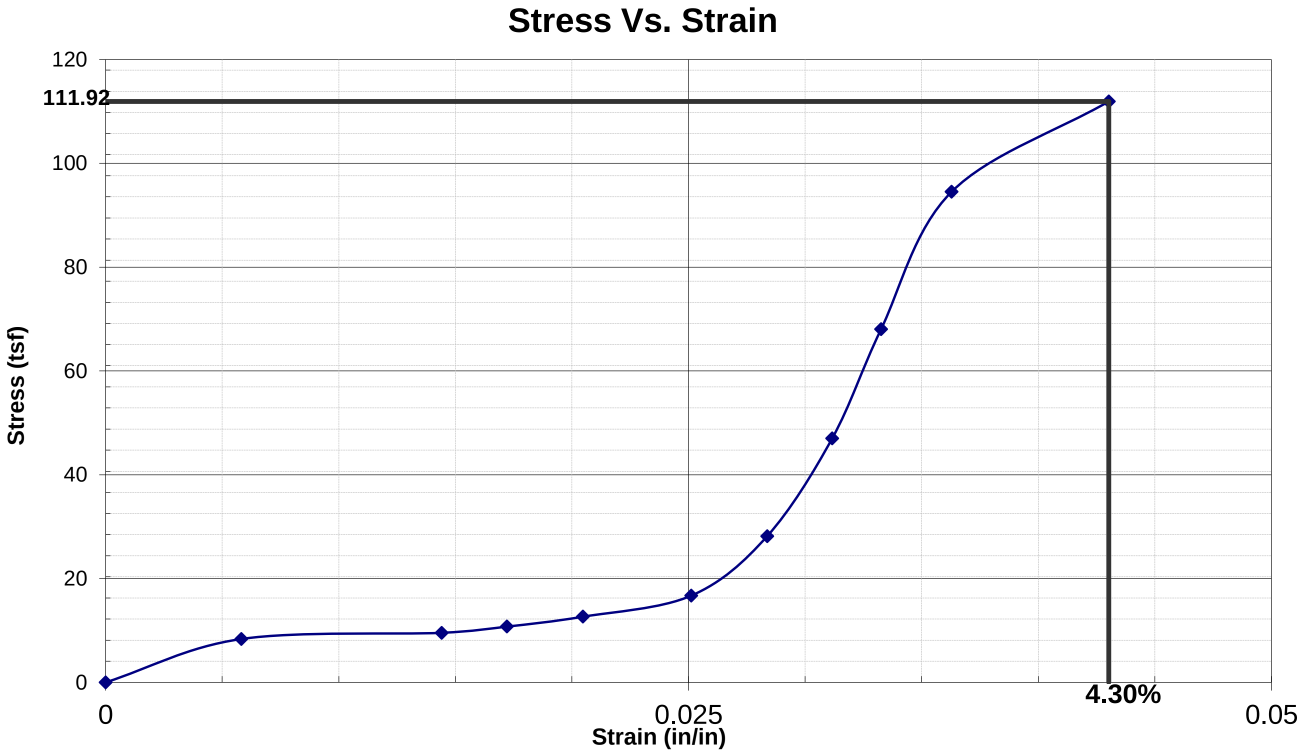
Unconfined Compression Test for Rock Cores

Date:	1/5/2017	Project:	Potomac River Crossing
Tested by:	Yamma Ershadi	Project No:	512713-2
Client:	Ensite USA	Location:	Washington County, MD

Average Initial Height (in):	4.30	Boring :	GO-3R
Average Initial Diameter (in):	1.96	Sample Number:	RC-57
Water Content %:	0.3	Sample Depth:	271.0-271.5
Wet Density (pcf):	171.8		
Dry Density (pcf):	171.2	Rock Description:	Soft gray Shale
LL - PL = PI:	NP - NP = NP		



Unconfined Compression Strength q_u (tsf):	**112**
Height to Diameter Ratio:	**2.2**
Percent Strain at Failure :	**4.30%**
Average Rate of Strain to Failure (% Strain/min):	**0.86%**

This test was performed according to ASTM D7012 - 14.
Compressive Strength of Intact Rock Core Specimens.

Failure Picture

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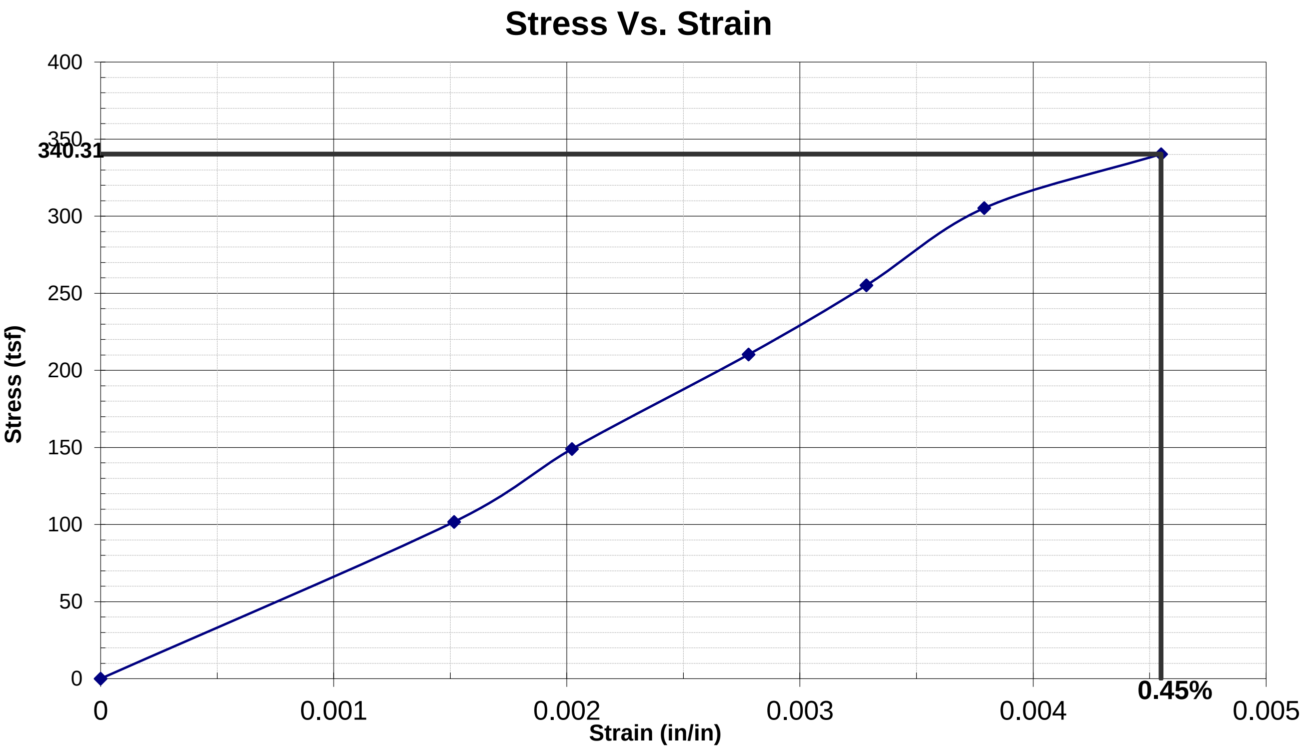
Unconfined Compression Test for Rock Cores

Date:	1/30/2017	Project:	Easter Panhandle Expansion
Tested by:	YE	Project No:	0512713-2
Client:	Ensite	Location:	Hancock, MD

Average Initial Height (in):	3.96	Boring :	GO-7
Average Initial Diameter (in):	1.98	Sample Number:	RC-11
Water Content %:	0.8	Sample Depth:	19.9' - 20.4'
Wet Density (pcf):	159.9	Rock Description:	Medium hard gray Limestone
Dry Density (pcf):	158.7		
LL - PL = PI:	NP - NP = NP		



Unconfined Compression Strength q_u (tsf):	**340**
Unconfined Compression Strength q_u (psf):	**680624**
Unconfined Compression Strength q_u (psi):	**4727**
Height to Diameter Ratio:	**2.0**
Percent Strain at Failure :	**0.45%**
Average Rate of Strain to Failure (% Strain/min):	**0.15%**
Time to Failure (min):	**3.0**

This test was performed according to ASTM D7012 - 14 Method C. Compressive Strength of Intact Rock Core Specimens.

Failure Picture

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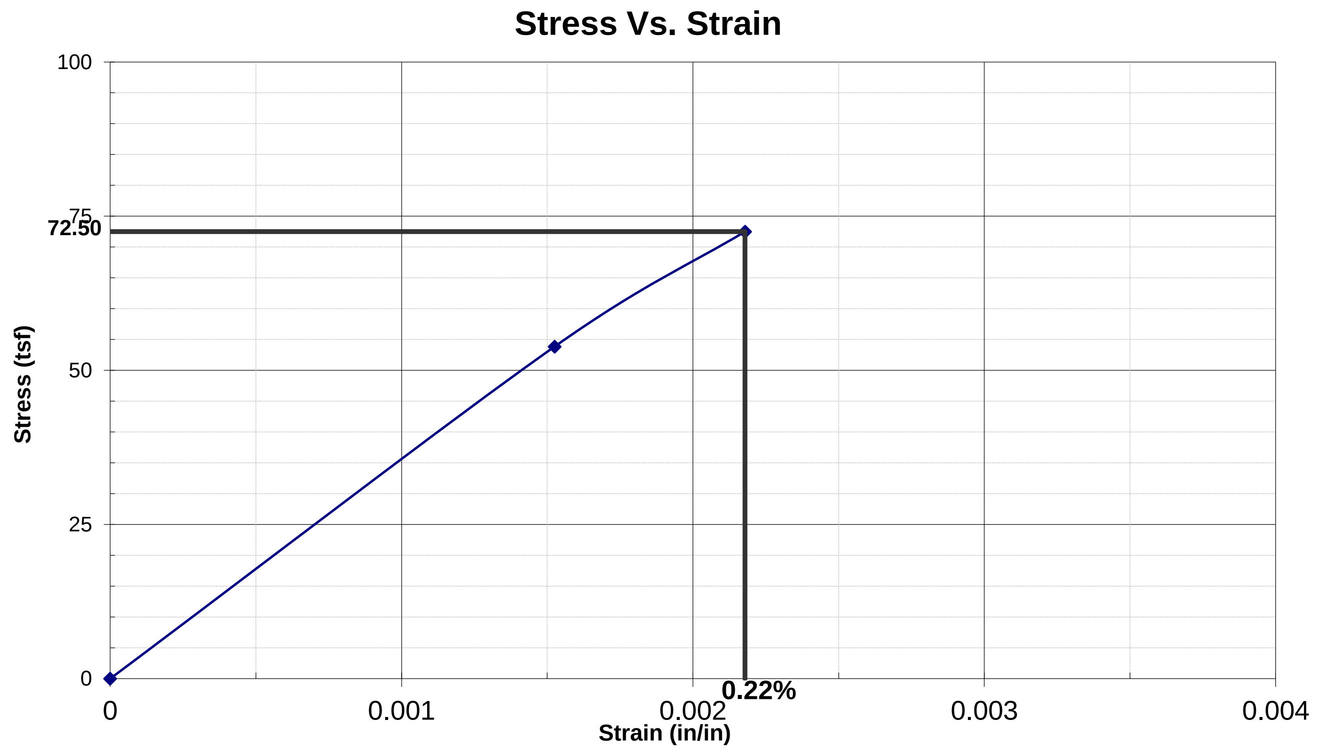
Unconfined Compression Test for Rock Cores

Date:	1/30/2017	Project:	Easter Panhandle Expansion
Tested by:	YE	Project No:	0512713-2
Client:	Ensite	Location:	Hancock, MD

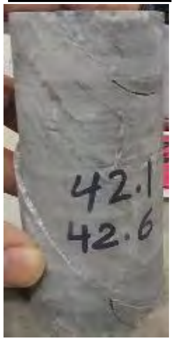
Average Initial Height (in):	4.59	Boring :	GO-7
Average Initial Diameter (in):	1.98	Sample Number:	RC-16
Water Content %:	0.5	Sample Depth:	42.1' - 42.6'
Wet Density (pcf):	159.0	Rock Description:	Soft gray Shale
Dry Density (pcf):	158.3		
LL - PL = PI:	NP - NP = NP		



Unconfined Compression Strength q_u (tsf):	**73**
Unconfined Compression Strength q_u (psf):	**145007**
Unconfined Compression Strength q_u (psi):	**1007**
Height to Diameter Ratio:	**2.3**
Percent Strain at Failure :	**0.22%**
Average Rate of Strain to Failure (% Strain/min):	**0.22%**
Time to Failure (min):	**1.0**

This test was performed according to ASTM D7012 - 14 Method C. Compressive Strength of Intact Rock Core Specimens.

Failure Picture



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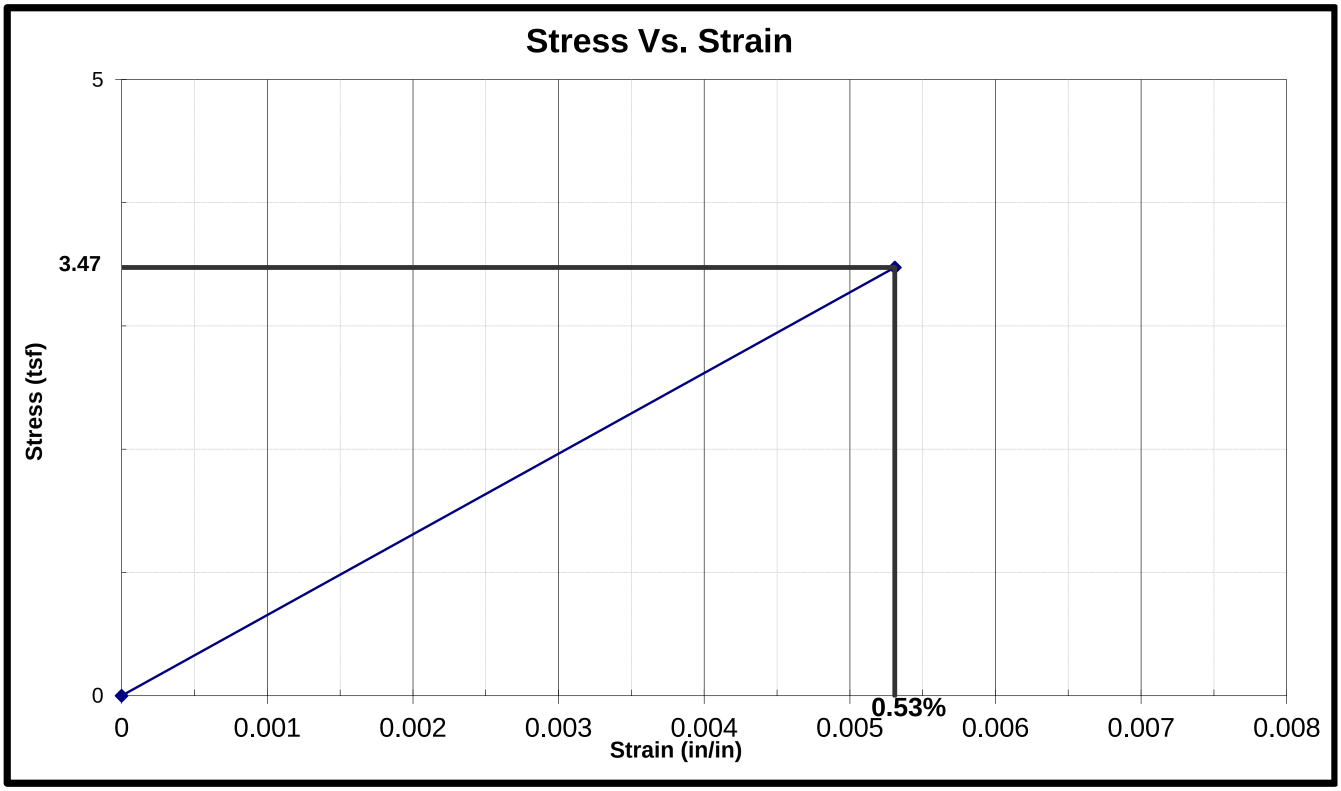
Unconfined Compression Test for Rock Cores

Date: 1/30/2017
Tested by: YE
Client: Ensite

Project: Easter Panhandle Expansion
Project No: 0512713-2
Location: Hancock, MD

Average Initial Height (in):	5.65
Average Initial Diameter (in):	1.98
Water Content %:	0.2
Wet Density (pcf):	144.3
Dry Density (pcf):	144.0
LL - PL = PI:	NP - NP = NP

Boring : GO-7
Sample Number: RC-22
Sample Depth: 90.5' - 91.0'
Rock Description: Extremely soft gray Shale



Unconfined Compression Strength q_u (tsf):	**3**
Unconfined Compression Strength q_u (psf):	**6950**
Unconfined Compression Strength q_u (psi):	**48**
Height to Diameter Ratio:	**2.9**
Percent Strain at Failure :	**0.53%**
Average Rate of Strain to Failure (% Strain/min):	**1.06%**
Time to Failure (min):	**0.5**

This test was performed according to ASTM D7012 - 14 Method C.
Compressive Strength of Intact Rock Core Specimens.
The H/D Ratio was not conforming to this method.

Failure Picture

2930 Eskridge Road
Fairfax, VA 22031

Phone (703) 698-9300
Fax (703) 560-7931

Unconfined Compression Test for Rock Cores

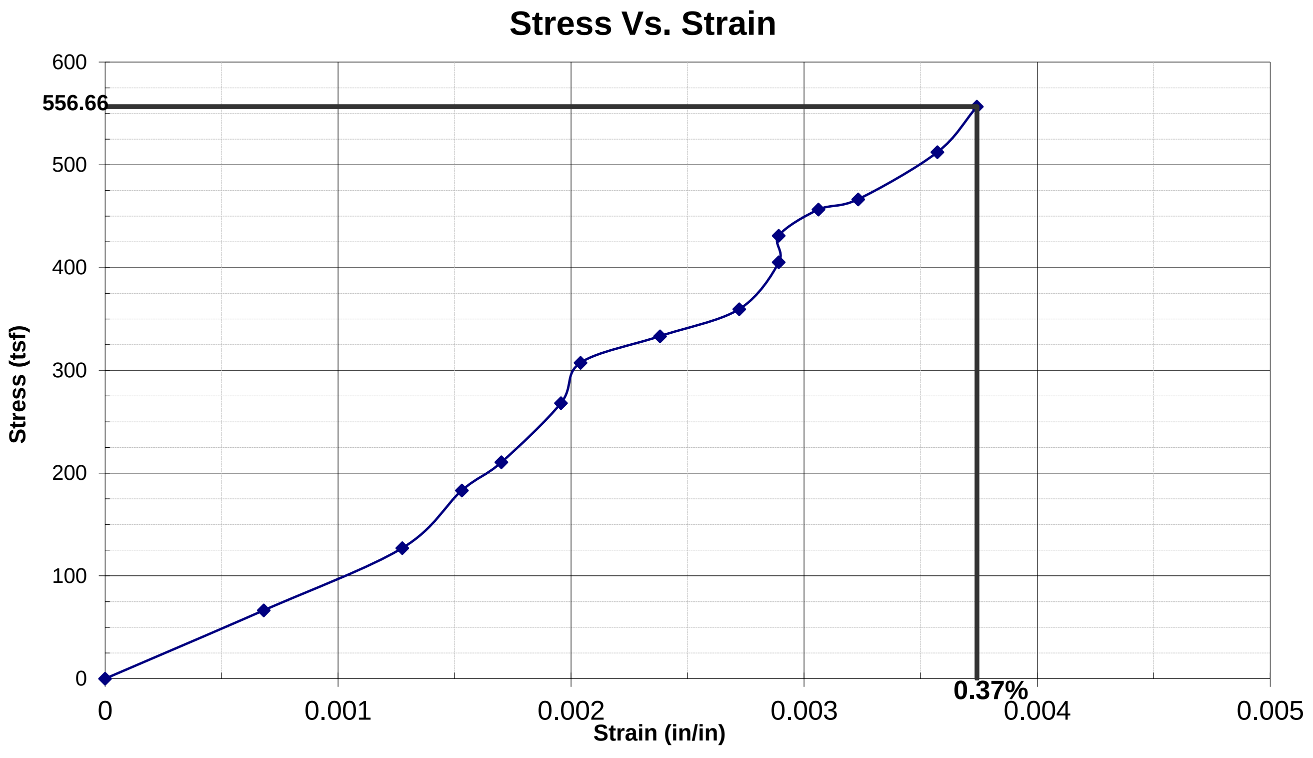
Date: 1/30/2017
Tested by: YE
Client: Ensite

Project: Easter Panhandle Expansion
Project No: 0512713-2
Location: Hancock, MD

Average Initial Height (in):	5.88
Average Initial Diameter (in):	1.99
Water Content %:	0.7
Wet Density (pcf):	169.6
Dry Density (pcf):	168.5
LL - PL = PI:	NP - NP = NP

Boring : GO-7
Sample Number: RC-33
Sample Depth: 131.5' - 132.0'

Rock Description: Moderately hard gray Limestone



Unconfined Compression Strength q_u (tsf):	**557**
Unconfined Compression Strength q_u (psf):	**1113319**
Unconfined Compression Strength q_u (psi):	**7731**
Height to Diameter Ratio:	**3.0**
Percent Strain at Failure :	**0.37%**
Average Rate of Strain to Failure (% Strain/min):	**0.05%**
Time to Failure (min):	**7.0**

This test was performed according to ASTM D7012 - 14 Method C.
Compressive Strength of Intact Rock Core Specimens.
The H/D Ratio was not conforming to this method.

Failure Picture

Slake Durability Test Results

Slake Durability of Shales and Similar Weak Rocks - ASTM D4644

Client	Professional Service Industries,	Boring	GO-3R
Client Project	Proposed Eastern Panhandle Ex	Depth	28.4'-29.4'
Project No.	38212	Sample	RC-9
		Lab Sample No.	38212005

Visual Description: Red Siltstone

Initial Water Content

Drum ID	B
Drum + Wet Shale, gm	1786.7
Drum + Dry Shale, gm	1778.1
Drum Wt., gm	1221.4
Water Content, %	2%
Initial Dry Shale Weight, gm	556.7
Water Temerature Before Cycle 1, *C	22
Water Temerature After Cycle 1, *C	21.7
Average Temp during Cycle 1, *C	21.85
Drum + Dry Shale after Cycle 1, gm	1768.9
Dry Shale after Cycle 1	547.5
Slake Durability Index (First cycle)	**98.3%**
Water Temerature Before Cycle 2, *C	18.2
Water Temerature After Cycle 2, *C	18.1
Average Temp during Cycle 2, *C	18.15
Drum + Dry Shale after Cycle 2, gm	1763
Dry Shale after Cycle 2	541.6
Slake Durability Index (Second cycle)	**97.3%**

Type II—Retained specimen consist of large and small fragments.

Initial Photograph

Final Photograph

Input Validation: MAK Reviewed By: ALO Date Tested: 1/6/2017

Slake Durability of Shales and Similar Weak Rocks - ASTM D4644

Client	Professional Service Industries,	Boring	GO-3R
Client Project	Proposed Eastern Panhandle Ex	Depth	141'-142'
Project No.	38212	Sample	RC-31
		Lab Sample No.	38212006

Visual Description: Gray Phyllite/Slate

Initial Water Content

Drum ID	A
Drum + Wet Shale, gm	1769.9
Drum + Dry Shale, gm	1765.4
Drum Wt., gm	1222.2
Water Content, %	1%
Initial Dry Shale Weight, gm	543.2
Water Temerature Before Cycle 1, *C	19.2
Water Temerature After Cycle 1, *C	19.9
Average Temp during Cycle 1, *C	19.55
Drum + Dry Shale after Cycle 1, gm	1761.4
Dry Shale after Cycle 1	539.2
Slake Durability Index (First cycle)	**99.3%**
Water Temerature Before Cycle 2, *C	22.5
Water Temerature After Cycle 2, *C	22.2
Average Temp during Cycle 2, *C	22.35
Drum + Dry Shale after Cycle 2, gm	1757.3
Dry Shale after Cycle 2	535.1
Slake Durability Index (Second cycle)	**98.5%**

Type II—Retained specimen consist of large and small fragments.

Initial Photograph

Final Photograph

Input Validation: MAK Reviewed By: ALO Date Tested: 1/4/2017

Slake Durability of Shales and Similar Weak Rocks - ASTM D4644

Client	Professional Service Industries,	Boring	GO-3R
Client Project	Proposed Eastern Panhandle Ex	Depth	177.5'-178.5'
Project No.	38212	Sample	RC-39
		Lab Sample No.	38212007

Visual Description: Gray Phyllite/Slate

Initial Water Content

Drum ID	B
Drum + Wet Shale, gm	1788.1
Drum + Dry Shale, gm	1784.7
Drum Wt., gm	1221.6
Water Content, %	1%
Initial Dry Shale Weight, gm	563.1
Water Temerature Before Cycle 1, *C	20.1
Water Temerature After Cycle 1, *C	20.6
Average Temp during Cycle 1, *C	20.35
Drum + Dry Shale after Cycle 1, gm	1782
Dry Shale after Cycle 1	560.4
Slake Durability Index (First cycle)	**99.5%**
Water Temerature Before Cycle 2, *C	23.2
Water Temerature After Cycle 2, *C	22.4
Average Temp during Cycle 2, *C	22.8
Drum + Dry Shale after Cycle 2, gm	1778.7
Dry Shale after Cycle 2	557.1
Slake Durability Index (Second cycle)	**98.9%**

Type II—Retained specimen consist of large and small fragments.

Initial Photograph

Final Photograph

Input Validation: MAK Reviewed By: ALO Date Tested: 1/4/2017

Slake Durability of Shales and Similar Weak Rocks - ASTM D4644

Client	Professional Service Industries,	Boring	G0-3R
Client Project	Proposed Eastern Panhandle Ex	Depth	228'-229'
Project No.	38212	Sample	RC-49
		Lab Sample No.	38212008

Visual Description: Gray Phyllite/Slate

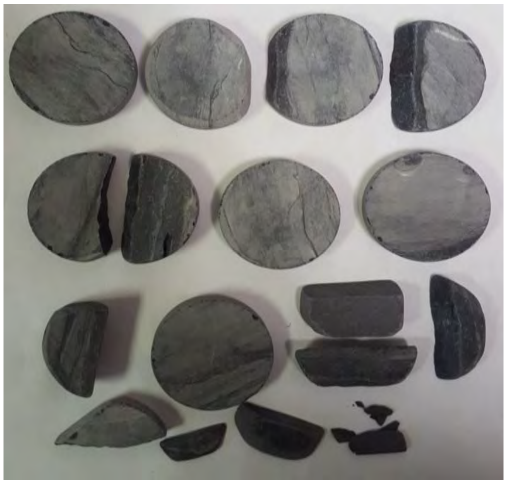
Initial Water Content

Drum ID	C
Drum + Wet Shale, gm	1821.6
Drum + Dry Shale, gm	1820.2
Drum Wt., gm	1234.7
Water Content, %	0%
Initial Dry Shale Weight, gm	585.5
Water Temerature Before Cycle 1, *C	20
Water Temerature After Cycle 1, *C	20.2
Average Temp during Cycle 1, *C	20.1
Drum + Dry Shale after Cycle 1, gm	1818.5
Dry Shale after Cycle 1	583.8
Slake Durability Index (First cycle)	**99.7%**
Water Temerature Before Cycle 2, *C	23.4
Water Temerature After Cycle 2, *C	23
Average Temp during Cycle 2, *C	23.2
Drum + Dry Shale after Cycle 2, gm	1817.1
Dry Shale after Cycle 2	582.4
Slake Durability Index (Second cycle)	**99.5%**

Type II—Retained specimen consist of large and small fragments.

Initial Photograph

Final Photograph



Input Validation: MAK Reviewed By: ALO Date Tested: 1/4/2017

Slake Durability of Shales and Similar Weak Rocks - ASTM D4644

Client	Professional Service Industries,	Boring	GO-3R
Client Project	Proposed Eastern Panhandle Ex	Depth	273'-274'
Project No.	38212	Sample	RC-58
		Lab Sample No.	38212009

Visual Description: Gray Phyllite/Slate

Initial Water Content

Drum ID	D
Drum + Wet Shale, gm	1796.5
Drum + Dry Shale, gm	1794.9
Drum Wt., gm	1238.4
Water Content, %	0.3%
Initial Dry Shale Weight, gm	556.5
Water Temerature Before Cycle 1, *C	20.7
Water Temerature After Cycle 1, *C	20.7
Average Temp during Cycle 1, *C	20.7
Drum + Dry Shale after Cycle 1, gm	1793.4
Dry Shale after Cycle 1	555
Slake Durability Index (First cycle)	**99.7%**
Water Temerature Before Cycle 2, *C	23.8
Water Temerature After Cycle 2, *C	23.2
Average Temp during Cycle 2, *C	23.5
Drum + Dry Shale after Cycle 2, gm	1792.1
Dry Shale after Cycle 2	553.7
Slake Durability Index (Second cycle)	**99.5%**

Type II—Retained specimen consist of large and small fragments.

Initial Photograph

Final Photograph

Input Validation: MAK Reviewed By: ALO Date Tested: 1/4/2017

Slake Durability of Shales and Similar Weak Rocks - ASTM D4644

Client	Professional Service Industries,	Boring	GO-7
Client Project	Proposed Eastern Panhandle Ex	Depth	14.9' - 15.9'
Project No.	38298	Sample	RC-3
		Lab Sample No.	38298001

Visual Description: Gray Siltstone

Initial Water Content

Drum ID	D
Drum + Wet Shale, gm	1755.2
Drum + Dry Shale, gm	1704.7
Drum Wt., gm	1238.4
Water Content, %	11%
Initial Dry Shale Weight, gm	466.3
Water Temerature Before Cycle 1, *C	20.1
Water Temerature After Cycle 1, *C	20
Average Temp during Cycle 1, *C	20.05
Drum + Dry Shale after Cycle 1, gm	1685.4
Dry Shale after Cycle 1	447
Slake Durability Index (First cycle)	**95.9%**
Water Temerature Before Cycle 2, *C	17.3
Water Temerature After Cycle 2, *C	17.5
Average Temp during Cycle 2, *C	17.4
Drum + Dry Shale after Cycle 2, gm	1661.2
Dry Shale after Cycle 2	422.8
Slake Durability Index (Second cycle)	**90.7%**

Type II—Retained specimen consist of large and small fragments.

Initial Photograph

Final Photograph

Input Validation: MAK Reviewed By: ALO Date Tested: 2/3/2017

Slake Durability of Shales and Similar Weak Rocks - ASTM D4644

Client	Professional Service Industries,	Boring	GO-7
Client Project	Proposed Eastern Panhandle Ex	Depth	54.0' - 55.0'
Project No.	38298	Sample	RC-11
		Lab Sample No.	38298002

Visual Description: Tan Siltstone with Interbedded Black Shale

Initial Water Content

Drum ID	A
Drum + Wet Shale, gm	1749.5
Drum + Dry Shale, gm	1720.1
Drum Wt., gm	1222.2
Water Content, %	6%
Initial Dry Shale Weight, gm	497.9
Water Temerature Before Cycle 1, *C	20.4
Water Temerature After Cycle 1, *C	20.1
Average Temp during Cycle 1, *C	20.25
Drum + Dry Shale after Cycle 1, gm	1716.6
Dry Shale after Cycle 1	494.4
Slake Durability Index (First cycle)	**99.3%**
Water Temerature Before Cycle 2, *C	17.6
Water Temerature After Cycle 2, *C	17.7
Average Temp during Cycle 2, *C	17.65
Drum + Dry Shale after Cycle 2, gm	1714.9
Dry Shale after Cycle 2	492.7
Slake Durability Index (Second cycle)	**99.0%**

Type II—Retained specimen consist of large and small fragments.

Initial Photograph

Final Photograph

Input Validation: MAK Reviewed By: ALO Date Tested: 2/3/2017

Slake Durability of Shales and Similar Weak Rocks - ASTM D4644

Client	Professional Service Industries,	Boring	GO-7
Client Project	Proposed Eastern Panhandle Ex	Depth	81.5' - 82.5'
Project No.	38298	Sample	RC-16
		Lab Sample No.	38298003

Visual Description: Gray Limestone

Initial Water Content

Drum ID	B
Drum + Wet Shale, gm	1761.2
Drum + Dry Shale, gm	1746.4
Drum Wt., gm	1221.6
Water Content, %	3%
Initial Dry Shale Weight, gm	524.8
Water Temerature Before Cycle 1, *C	20.1
Water Temerature After Cycle 1, *C	20
Average Temp during Cycle 1, *C	20.05
Drum + Dry Shale after Cycle 1, gm	1741
Dry Shale after Cycle 1	519.4
Slake Durability Index (First cycle)	**99.0%**
Water Temerature Before Cycle 2, *C	17.8
Water Temerature After Cycle 2, *C	17.9
Average Temp during Cycle 2, *C	17.85
Drum + Dry Shale after Cycle 2, gm	1737.9
Dry Shale after Cycle 2	516.3
Slake Durability Index (Second cycle)	**98.4%**

Type II—Retained specimen consist of large and small fragments.

Initial Photograph

Final Photograph

Input Validation: MAK Reviewed By: ALO Date Tested: 2/3/2017

Slake Durability of Shales and Similar Weak Rocks - ASTM D4644

Client	Professional Service Industries,	Boring	GO-7
Client Project	Proposed Eastern Panhandle Ex	Depth	129.0' - 130.0'
Project No.	38298	Sample	RC-26
		Lab Sample No.	38298004

Visual Description: Gray Shale

Initial Water Content

Drum ID	C
Drum + Wet Shale, gm	1749.7
Drum + Dry Shale, gm	1741.5
Drum Wt., gm	1234.5
Water Content, %	2%
Initial Dry Shale Weight, gm	507
Water Temerature Before Cycle 1, *C	20.5
Water Temerature After Cycle 1, *C	20.1
Average Temp during Cycle 1, *C	20.3
Drum + Dry Shale after Cycle 1, gm	1737.6
Dry Shale after Cycle 1	503.1
Slake Durability Index (First cycle)	**99.2%**
Water Temerature Before Cycle 2, *C	17.6
Water Temerature After Cycle 2, *C	17.8
Average Temp during Cycle 2, *C	17.7
Drum + Dry Shale after Cycle 2, gm	1736.1
Dry Shale after Cycle 2	501.6
Slake Durability Index (Second cycle)	**98.9%**

Type II—Retained specimen consist of large and small fragments.

Initial Photograph

Final Photograph

Input Validation: MAK   Reviewed By: ALO   Date Tested: 2/3/2017

Soil Resistivity Test Results

Corrosivity Testing

Client	Professional Service Industries, Inc. (PSI)
Client Project	Proposed Eastern Panhandle Expansion I-68 Crossing Wash Co MD
Project No.	38212

						pH AASHTO T289			Soil Resistivity AASHTO T-288		
Lab Sample ID	**Boring**	**Depth**	**Sample**	**Sample Received**	**Matrix**	**Result**	Date Tested	Tested By	**Result, Ohm-cm**	Date Tested	Tested By
38212001	GO-3R	0.0'-2.0'	ST-1	12/30/2016	Soil	4.8	1/9/2017	TX	28500	1/9/2017	TX

Input Validation: TX

Reviewed By: ALO

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Report of
Subsurface Exploration and
Geotechnical Engineering Services

Proposed Point-of-Delivery Facility,
6493 – Eastern Panhandle Expansion,
Morgan County, West Virginia

Prepared for

EnSiteUSA
109 Fieldview Drive, P.O. Box 1007
Versailles, KY 40383

Prepared by

Professional Service Industries, Inc.
2930 Eskridge Road
Fairfax, Virginia 22031
Phone: (703) 698-4414

February 2, 2017

PSI Project No. 0512713

Lubomir Peytchev

Lubomir D. Peytchev
Project Manager

2/2/17

KARL E. SUTER
18356

Karl E Suter

Karl Suter, P.E.
Chief Engineer

psi Information To Build On
Engineering • Consulting • Testing

Geotechnical

February 2, 2017

EnSiteUSA
109 Fieldview Drive
P.O. Box 1007
Versailles, KY 40383

Attention: Jacob Shams, P.E.

Reference: Report for Geotechnical Exploration and Assessment
Proposed Point-of-Delivery(POD) Facility
6493 – Eastern Panhandle Expansion
Morgan County, West Virginia
PSI Project Number: 0512713

Dear Mr. Shams:

Thank you for choosing Professional Service Industries, Inc. (PSI) as your geotechnical consultant for the proposed Point-of-Delivery(POD) Facility for Eastern Panhandle Expansion in Morgan County, West Virginia. This facility is planned as part of the Eastern Panhandle Expansion (Pipeline) Project.

As per your authorization, we have completed a subsurface exploration for this project. The findings of the exploration and our recommendations for the proposed development are discussed in the accompanying report. As requested, one electronic and three original hard copies of the report will be provided to you.

The soil samples obtained during this exploration will be retained in our laboratory for sixty days. Should there be any questions, please do not hesitate to contact our office. PSI would be pleased to continue providing geotechnical services throughout the implementation of the project, and we look forward to working with you and your organization on this and future projects.

Respectfully submitted,
Professional Service Industries, Inc.

Lubomir D. Peytchev

Karl E Sut

Lubomir D. Peytchev
Project Manager

Karl Suter, P.E
Chief Engineer

TABLE OF CONTENTS

APPENDICES

1 EXECUTIVE SUMMARY

PSI has completed the geotechnical assessment for Point-of-Delivery(POD) Facility for Eastern Panhandle Expansion in Morgan County, West Virginia. One soil test boring GO-6 was drilled to a depth of 40 feet below the existing grade.

The test boring encountered 3 inches of topsoil over the existing native soils described as loose to very dense silty sands over weathered shale and limestone bedrock to the boring termination depth of 40 feet.

As of the preparation of this report, limited structural loading information was provided. We anticipate the maximum load on the column to be less than 20 kips.

Existing residual soils will likely be encountered at the foundation bearing elevation. Based on the review of the test borings and taking into account the assumed structural loads, the proposed POD Facility can be supported on the existing medium dense to dense residual soils. Shallow foundations can be proportioned using an allowable bearing pressure value of 3,000 pounds per square foot (psf).

Groundwater was encountered in the boring GO-6 while drilling operations were performed, at approximately 29.5 feet below ground surface. As such, groundwater will not likely be encountered during the construction of the shallow foundations. However, there may be the groundwater seepage from the surface run off. In such case, the sump pumps can be used for temporary dewatering.

Recommendations relative to earthwork and foundation design are detailed in the report. The owner/designer should not rely solely upon the executive summary and must read and evaluate the entire contents of this report, prior to utilizing our engineering recommendations in the preparation of design and construction documents.

2 PROJECT INFORMATION

2.1 PROPOSAL AND PROJECT AUTHORIZATION

This report presents the findings and recommendations related to the geotechnical exploration program performed by Professional Service Industries, Inc. (PSI) for the proposed POD Facility in Morgan County, West Virginia. These services were planned and performed in general accordance with scope and services outlined in PSI Proposal No 0512-182348 and Change Order CO#1 dated July 28, 2016.

2.2 PROJECT DESCRIPTION

Initial project information was provided by Mr. Jacob Shams with EnSiteUSA. We also reviewed the RFP document titled, "Potomac River Crossing, Additional Investigation, for Eastern Panhandle Expansion, Washington County, Maryland and Morgan County West Virginia, dated July 26, 2016. The project involves the construction of a Point-of-Delivery Facility, which will support the pipes above grade. Based on the drawings provided to us, the pipes will be supported on isolated concrete columns that will extend approximately 3 feet, above the finished grade. We anticipate very minimal cut and fill grading activities of less than 1 foot.

As of the preparation of this report, limited structural loading information was provided. We anticipate the maximum load on the column to be less than 20 kips.

If any of the noted information is incorrect or has changed, please inform PSI so that we may review the geotechnical data and amend the recommendations presented in this report, if deemed appropriate.

2.3 PURPOSE AND SCOPE OF WORK

The scope of services for this study included a site reconnaissance of the project area and the assessment of subsurface conditions through field exploration and laboratory testing. The study included an assessment of the site and subsurface conditions relative to the proposed development, engineering studies and the preparation of this report. The subsurface exploration was developed to provide the following:

- Geologic review of the project site.
- Subsurface conditions encountered including pertinent soil properties including water levels and drainage.
- Soil data review and analysis as it relates to the proposed site development.
- Civil site recommendations for site preparation, placement and compaction of fill.
- Structural recommendations to support foundation and construction.
- Comments relating to observed geotechnical conditions such as soft material or groundwater which could impact development.

- Determination of the Seismic Site Class and seismic design parameters per IBC 2015 based on the SPT N-values obtained during field exploration.

The scope of our services did not include an environmental assessment for determining the presence or absence of wetlands, or hazardous or toxic materials in the soil, bedrock, groundwater, or air, on or below or around this site. Any statement in this report or on the boring logs regarding odors, colors, unusual or unexpected items or conditions are strictly for the information of our client.

PSI did not provide nor was it requested to provide any service to investigate or detect the presence of moisture, mold or other biological contaminants in or around any structure, or any service that was designed or intended to prevent or lower the risk of the occurrence of the amplification of the same. Client acknowledges that mold is ubiquitous to the environment with mold amplification occurring when building materials are impacted by moisture. Client further acknowledges that site conditions are outside of PSI's control, and that mold amplification will likely occur, or continue to occur, in the presence of moisture. As such, PSI cannot and shall not be held responsible for the occurrence or recurrence of mold amplification.

2.4 SUBSURFACE EXPLORATION

PSI subcontracted Connelly Drilling Inc. to provide drilling services for the exploration program at the site. One soil test boring designated as GO-6 was drilled to a depth of 40 feet, below the existing grade.

The boring was completed with a track-mounted drill rig with HSA in conformance with ASTM standards. Standard Penetration Testing (SPT) and split-spoon sampling of overburden soils was performed at 2.5 foot intervals for the first 10 feet and at 5-foot intervals thereafter to the termination depths to evaluate the strength and relative consistency of the soils encountered. Below auger refusal depth, rock coring was performed using NQ coring equipment. All recovered soil and rock samples were visually classified by a PSI geotechnical engineer and a graphical log developed for each boring. Boring depth and depth at which auger refusal was encountered are summarized in Table 1 below.

Table 1 – Summary of Boring Depths

Boring	Approximate Termination Depth (feet)	Ground Surface Elevation (feet, NAVD)	Approximate Depth/Elevation of Top of Weathered Rock	Approximate Depth/Elevation of Auger Refusal
GO-6	40	590	23 feet, EL ±567MSL	30 feet, EL ±560MSL

The boring log included in the Appendix shows approximate depths and visual descriptions of overburden soil, underlying rock materials encountered, soil SPT test results, rock core recovery and quality designation (RQD) values, and measurements of groundwater depth where encountered. The total length of recovered rock core, divided by the length of the run, is referred to as rock core recovery and is expressed as a percentage. The Rock Quality Designation (RQD) is a measure of the rock mass quality and is defined as the total length of sound, intact rock core pieces 4 inches or more in length divided by the length of the rock core run, also expressed as a percentage. The rock core recovery and RQD values are indicated on the Boring Log included with this report.

The location of the boring is shown on the Boring Location Plan, in Appendix **B**. The findings of the PSI boring are presented on the Test Boring Log included in Appendix **C**.

2.5 LABORATORY TESTING

A PSI geotechnical engineer visually-manually classified the soil samples obtained for this geotechnical report in general accordance with the Unified Soil Classification System (USCS) (ASTM D2487 and D2488). Selected samples were tested for natural water content (ASTM D2216), Atterberg limits tests (ASTM D4318), grain size analyses (ASTM D6913), Soil Unconsolidated Undrained Compressive Strength Test (ASTM D2850), Slake Durability Test (ASTM D4644), Soil Resistivity Test (AASHTO T-288) and Soil pH Test (AASHTO T-289).

Table 2 – Overburden Soil Classification Test Results

Boring	Sample No.	Sample Depth (feet)	USCS Classification [1]	Moisture Content (%)	Atterberg Limits			Grain-Size Distribution		
					Liquid Limit	Plastic Limit	Plasticity Index	Gravel (%)	Sand (%)	Fines (%)
GO-6	S-1	0.0 – 1.5	SM	16						
GO-6	ST-1	0.0 – 2.0	SM	16	37	25	12	7.9	63.4	28.7
GO-6	ST-2	2.0 – 4.0	SM	16						
GO-6	S-2	2.5 – 4.0	SM	19						
GO-6	S-3	5.0 – 6.5	SM	19	41	27	14			34.5
GO-6	S-4	8.5 – 10.0	SM	14						
GO-6	S-5	13.5 – 15.0	SM	13						
GO-6	S-6	18.5 – 20.0	GM	13						
GO-6	S-7	23.5 – 25.0	GM	5	26	19	7			13.3

(1) For USCS Soil Classification definitions, refer to the General Notes in Attachment
(2) ST – Shelby Tube soil sample
(3) S – Split spoon soil sample

Table 3 – Rock Recovery and RQD Field Coring Test Results

Boring	Depth (feet)	Elevation (feet)	Run Length (feet)	Recovery (%)	RQD (%)	Hardness (Moh,s)
GO-6	30 - 35	560 - 555	5	100	25	3
GO-6	35- 40	555 – 550	5	70	0	3

Table 4 – Rock Unconfined Compressive Strength Test Results

Boring	Approximate Sample Depth (feet)	Rock Classification	Unit Weight (pcf)	Unconfined Compressive Strength	
				(psi)	(tsf)
Rock Compressive Strength Tests were not performed as the available rock core samples had cracks and fissures					

Table 5 – Soil, Unconsolidated Undrained Triaxial Compression Test Results

Boring	Approximate Sample Depth (feet)	Water Content (%)	Dry Unit Weight (pcf)	Soil Classification	Confining Stress	Shear Strength, S_u	
					(psi)	(psf)	(tsf)
GO-6	0.0 – 4.0	17.9	112.5	SM	6.9	2966	1.5
GO-6	0.0 – 4.0	14.4	121.4	SM	13.9	4190	2.1
GO-6	0.0 – 4.0	18.4	112.8	SM	27.8	3643	1.8

The **durability** of the shale is a measurement of its deterioration over time interaction with the water weathering properties. The durability of the shale was determined on a selected sample of shales per Slake Durability of Shales and Similar Weak Rocks, ASTM D-4644 Standard.

Table 6 – Slake Durability Test Results

Boring	Approximate Sample Depth (feet)	Rock Classification	Slake Durability Index First Cycle (%)	Slake Durability Index Second Cycle (%)
GO-6	31.5 – 32.5	Shale	98.9	98.3

One representative soil sample was selected by PSI for soil resistivity testing. Table 7 below presents a summary of the test results. A detailed report is included in the Appendix D.

Table 7 – Soil Resistivity Test Results

Location	GO-6
Depth (Foot)	0.0’ to 4.0’
pH - AASHTO T289	4.4
Soil Resistivity – AASHTO T-288	18000 Ohm-cm

3 SITE AND SUBSURFACE CONDITIONS

3.1 SITE LOCATION AND DESCRIPTION

The proposed project site is located on the east side of the Hancock Road (WV SR 522), 0.9 mile south of the intersection of Hancock Road with River Road (WV SR 1) in Morgan County, West Virginia. Based on Google Earth, the existing grade within the limits of the proposed Tie-in-Facility is relatively level and varies from EL. 590 to EL. 592 feet. The surface cover within the limits of the proposed facility consists of grass, brush and young forest. The location of the site is shown on the Boring Location Plan attached as **Appendix B**.

3.2 AREA GEOLOGY

The site is geologically located in the Ridge and Valley Province. A study of the area geology from the available literature shows that the site is underlain by Marcellus Formation and Needmore Shale of Devonian age. The Marcellus Formation in general consists of gray, thin-bedded shale and argillaceous limestone.

3.3 SUBSURFACE CONDITIONS

The stratification of the subsurface conditions at the soil test boring location is described in this section. The log of the boring is provided in Appendix C.

A brief summary of subsurface stratigraphy as encountered at the boring is presented below. The soil is classified per the Unified Soil Classification System (ASTM D-2487):

<u>Surficial Materials:</u> Approximately 3 inches of surficial topsoil were encountered at the ground surface of Boring GO-6.

<u>Alluvium:</u> Alluvium was observed with thickness up to 2 feet consisting of loose to medium dense silty sand (SM) with shale fragments, gravel, and roots. The SPT N-value in this layer was 9 blows per foot of penetration (BPF).

<u>Residuum:</u> Residual soil classified as medium dense to very dense silty SAND (SM) was encountered to depth of approximately 19 feet below existing surface grade at the test boring. The residual soil was approximately 17 feet thick in the test boring location. SPT N-values in this layer ranged from approximately 19 to 84 BPF and 50 blows per 4-inches of penetration.

<u>Weathered Rock:</u> Typically consisting of weathered shale, weathered rock was encountered at the test boring location. The weathered rock samples consisted of soft shale with limestone floaters. SPT N-values were typically in excess of 50 BPF. Auger refusal was encountered within the weathered rock at depth of 19 feet below existing grades.

<u>Bedrock:</u> Bedrock materials encountered below the auger refusal depths consisted primarily of Shale and Limestone. Voids were not encountered in Boring GO-6. Core recoveries were 70 and 100 percent. RQD values ranged from 0 to 25 percent.

The above subsurface descriptions are of a generalized nature provided to highlight the major strata encountered. The boring log included in the Appendix should be reviewed for specific information at the boring location. The stratification lines shown on the boring log represent the conditions only at the actual boring location. The stratification lines represent the approximate boundaries between subsurface materials and the actual transition may be gradual.

3.4 GROUNDWATER CONDITIONS

During drilling groundwater was encountered in the test boring GO-6 at approximate depth of 29.5 feet below the exisnig grade. Water level at the test boring location is shown on the boring log provided in Appendix C.

The groundwater observations presented in this report and the attached boring log reflect those observed at the time of our field activities. We recommend that the Contractor determine the actual groundwater levels at the time of construction to determine groundwater impact on the proposed construction procedure.

4 GEOTECHNICAL ASSESMENT AND RECOMMENDATIONS

The following recommendations are based on the information available on the proposed construction, the data obtained from the boring, and our experience with soils and subsurface conditions similar to those encountered at this site. Because the borings represent a very small statistical sampling of the subsurface materials, conditions encountered during construction may be substantially different from those encountered in our borings. In these instances, adjustments to the design and construction may be necessary depending on the actual conditions encountered.

As indicated earlier, very minimal cut and fill (less than a foot) is anticipated within the proposed construction limits of the POD Facility. Based on the review of the test boring, competent residual soils will likely be encountered at the design foundation bearing level of the isolated columns, assumed to be about 5 feet below grade and thus below the frost penetration depth, which is 36 inches below grade. If deleterious or incompetent soils are encountered the design foundation bearing level of the columns, then such soils shall be undercut to the competent stratum and replaced with compacted structural fill or lean concrete, placed up to the bottom of the design foundation level.

4.1 SEISMIC CONSIDERATIONS

The project site is located within a municipality that employs the International Building Code (IBC), 2015 edition. As part of this code, the design of structures must consider dynamic forces resulting from seismic events. These forces are dependent upon the magnitude of the earthquake event as well as the properties of the soils that underlie the site.

Part of the IBC code procedure to evaluate seismic forces requires the evaluation of the Seismic Site Class, which categorizes the site based upon the characteristics of the subsurface profile within the upper 100 feet of the ground surface.

To define the Seismic Site Class for this project, and in accordance with your requested level of assessment, we have interpreted the results of our soil test borings drilled within the project site per Section 1613.5 of the code. Material properties were estimated below the depth of the borings based upon data available in published geologic reports as well as our experience with subsurface conditions in the general site area.

Based upon our assessment, it is our opinion that the subsurface conditions within the areas of the site planned for building construction are consistent with the characteristics of **Site Class C** as defined in Table 1613.5.2 of the building code.

The associated IBC probabilistic ground motion values for latitude 39.678878° and longitude -78.194106° obtained from the *Java Ground Motion Parameter Calculator – Version 5.1.0* on the USGS Earthquake Hazards Program – Seismic Design for Buildings web page (http://earthquake.usgs.gov/designmaps/us/application.php) are as follows:

Table 8: Seismic Design Parameters*

Period (seconds)	Mapped MCE Spectral Response Acceleration** (g)		Site Coefficients		Adjusted MCE Spectral Response Acceleration (g)		Design Spectral Response Acceleration (g)	
0.2	S_s	0.130	F_a	1.2	SM_S	0.156	SD_S	0.104
1.0	S_1	0.053	F_v	1.7	SM_1	0.091	SD_1	0.061

* 2% Probability of exceedance in 50 years.
** At B-C interface (i.e. top of bedrock).
MCE= Maximum Considered Earthquake

The Site Coefficients, F_a and F_v presented in the above table were also obtained from the USGS calculator, but can be interpolated from IBC Tables 1613.5.3(1) and 1613.5.3(2) as a function of the site classification and mapped spectral response acceleration at the short (S_s) and 1 second (S_1) periods.

For Seismic Design Category designations of C, D, E or F, which are contingent on the structure “Occupancy Category”, the Code also requires an assessment of liquefaction, slope stability and surface rupture due to faulting or lateral spreading. Detailed evaluations of these factors were beyond the scope of this study. However, the following table presents a qualitative assessment of these issues considering the site class, the subsurface soil properties, the groundwater elevation and probabilistic ground motions.

Table 9: Seismic Hazards

Hazard	Relative Risk	Comments
Liquefaction	Low	The materials below the foundation bearng level are dense, and the seismicity is low.
Slope Stability	Low	The site is relatively level and does not incorporate significant cut or fill slopes
Surface Rupture	Low	The site is not underlain by a mapped Holocene-aged fault

4.2 SITE PREPARATION AND EARTHWORK

We anticipate site preparation and earthwork for the proposed Tie-in-Facility to consist primarily of foundation excavation and backfilling.

- Utilities, if any, encountered within the proposed column pad footprints should be removed or relocated. The utility excavations shall be backfilled and compacted as per the fill requirements provided in the subsequent paragraphs.

- All loose or wet soils or any debris encountered at the footing subgrade elevation shall be undercut and replaced with structural fill.

- Material satisfactory for structural fill may include clean soil or bankrun sand and gravel (SW, SP, SM, GW, and GM). CL, ML, GC, and SC material can be used in engineered fills, subject to the following limitations:

Maximum Dry Density (per ASTM D698)	≥ 105 pcf
Liquid Limit	≤ 40
Plasticity Index	≤ 20

Organic soils and high plasticity clays and silts (CH, MH, OL, OH, PT) should not be used as engineered fill. The fill materials should be free from topsoil and debris, have less than 3 percent organics and should not contain rock fragments having a major dimension greater than 3 inches. The use of the excavated fill soils for controlled structural fill will be subject to approval of the Geotechnical Engineer of Record and moisture adjustments at the time of construction, and the plasticity and maximum dry density requirement specified in this section.

The onsite existing fill material can be reused as a structural fill provided it meets the above indicated requirements.

- Fill placement should be in loose horizontal lifts no greater than 8 inches thick compacted uniformly with the proper equipment.

- Fill required to support the footings and the slab-on-grade should be compacted to at least 98 percent of the maximum dry density as per ASTM D698 (Standard Proctor) test method. The moisture content of the fill should be within plus or minus two (±2) percentage points of the optimum moisture content.

For proper site preparation, the earthwork should be performed under the observation of and to the satisfaction of the Geotechnical Engineer of Record or his authorized representative.

It will be important to maintain positive site drainage during construction. Stormwater runoff should be diverted around the excavated areas. The site should be graded at all times such that water is not allowed to pond. If any surface soils become wet due to rains, they should be removed or dried prior to further site work operations and/or fill placement.

4.3 FOUNDATION DISCUSSION

The isolated columns supporting the pipes of POD Facility can be supported on isolated spread foundations bearing on the underlying competent residual soils. The bottom of the column foundations should be below the frost penetration depth, which is assumed to be 36 inches, below the existing grade. This is consistent with the design drawings provided to us showing the bearing level being about 5 feet below grade.

Spread foundations can be proportioned using a net allowable soil bearing pressure of 3,000 pounds per square foot (psf). Utilizing this allowable bearing pressure, we estimated the total settlement to be less than 1 inch with differential settlement being less than ½ inch over a horizontal distance of 25 feet.

Because of possible variations in subsurface conditions and related bearing capacity, all footing excavations and trenches should be observed and approved by the Geotechnical Engineer of Record or his qualified representative. Water and possibly some loose soil may collect in the footing excavations as a result of surface precipitation and near ground surface seepage. Therefore:

- Water, loose soil and soil softened by water should be removed from the bottom of the footing excavations before placing concrete.

- Footing excavations should not be left open for long periods. If the concrete cannot be placed due to inclement weather conditions or any other unforeseen circumstances, the bottom of the footing excavations and trenches should be protected by undercutting 3 inches and placing a 3-inch thick lean-mix concrete (2,000 psi) work mat immediately upon approval and before reinforcing steel is placed.

Where unsuitable bearing conditions are encountered as determined by the PSI Geotechnical Engineer or designated representative, these soils should be undercut and replaced with controlled structural fill. If backfilled up to the design bearing elevation, the over-excavation should extend laterally from all foundation edges a minimum of one half the depth of the undercut. The backfill should consist of the materials described earlier in this section. If the overexcavation is filled with concrete or flowable fill, the widening of the excavation will not be required. Backfill around and above the footing should satisfy the controlled fill requirements described in Section 4.1 ‘Site Preparation and Earthwork’.

Column footings should have minimum widths of 24 inches, regardless of the actual bearing pressure.

4.4 CONSTRUCTION DEWATERING

During our investigation groundwater was encountered in the Boring GO-6 at 29.5 feet. As such, groundwater may not be encountered during the foundation excavation. However, additional water may be introduced into excavations due to surface runoff and local precipitation during construction. Our past experience indicates that the foundation and subgrade bearing soils encountered on-site will soften considerably when exposed to free water. The contractor should keep excavations dry to prevent the softening of these materials. Methods such as sloping, ditching, and berming should be used to control surface water at the site.

Groundwater at this site can be handled by using sump pumps and pits may be utilized to direct and remove the water both during and after construction.

For the purposes of managing water that may enter an excavation, we recommend that collection pits with pumps be used to remove the water from the excavation. The sump pits should be backfilled with open graded stone (AASHTO #57 recommended) and should be surrounded by a properly graded filter medium. The purpose of the filter medium is to prevent clogging of the drainage system by the infiltration of fine-grained soils.

Pumping from the sump pits should be done with care to prevent the loss of soil fines, development of soil boils, or instability of slopes. We must emphasize that dewatering requirements will be dictated by groundwater conditions at the time of construction and may require more aggressive techniques than pumping from a sump pit. The contractor should use a technique or combination of techniques which achieve the desired results under actual field conditions.

5 CONSTRUCTION CONSIDERATIONS

To assess that the in-situ soil conditions or those conditions developed during the construction are as anticipated during the design stage, construction control, continuous observation and testing are recommended as follows:

- Structural fill placement, if any, should be monitored by a qualified soils technician working under the supervision of the geotechnical engineer of record.

- All footing excavations should be carried out under the observation of the geotechnical engineer of record or authorized representative.

5.1 EXCAVATION AND SAFETY

In Federal Register, Volume 54, No. 209 (October 1989), the United States Department of Labor, Occupational Safety and Health Administration (OSHA) amended its “Construction Standards for Excavations, 29 CFR, Part 1926, Subpart P”. This document was issued to better allow for the safety of workers entering trenches or excavations. It is mandated by this federal regulation that excavations, whether they be utility trenches, basement excavations or footing excavations, be constructed in accordance with the new OSHA guidelines. It is our understanding that these regulations are being strictly enforced and if they are not closely followed, the owner and the Contractor could be liable for substantial penalties.

The Contractor is solely responsible for designing and constructing stable, temporary excavations and should shore, slope, or bench the sides of the excavations as required to maintain stability of both the excavation sides and bottom. The Contractor's “responsible person”, as defined in 29 CFR Part 1926, should evaluate the soil exposed in the excavations as part of the Contractor’s safety procedures. In no case should slope height, slope inclination, or excavation depth, including utility trench excavation depth, exceed those specified in all local, state, and federal safety regulations.

We are providing this information solely as a service to our client. PSI does not assume responsibility for construction site safety or the Contractor’s or other parties' compliance with local, state, and federal safety or other regulations.

6 RECOMMENDED ADDITIONAL SERVICES

Additional foundation engineering, testing, and consulting services recommended for this project are summarized below:

- Footing Evaluations: It is recommended that footing for this project be evaluated by PSI. The purpose of these evaluations will be to verify that the design soil bearing pressure is available and that subgrade areas are properly prepared.

- Earthwork & Compaction Testing: It is recommended that an experienced engineering technician witness the required filling operations and take sufficient in-place density tests to verify that the specified degree of compaction has been achieved. Soil engineering judgments will be involved and should be made by the geotechnical engineer of record with information provided by the engineering technician.

- Soils Laboratory Testing: Testing to aid in the classification and verification of use of the on-site soils for structural fill and/or embankment material should be performed by PSI. Testing includes, but is not limited to, Atterberg Limits, Grain Size Analysis, California Bearing Ratio, Standard Moisture Density Relationship, and Moisture Content.

7 REPORT LIMITATIONS

The recommendations submitted in this report are based upon the available subsurface information obtained by PSI and design details furnished by **EnSiteUSA** for the proposed project. If there are any revisions to the plans for this project or if deviations from the subsurface conditions noted in this report are encountered during construction, PSI should be notified immediately to determine whether the recommendations provided herein must be changed. If PSI is not retained to perform these functions, we will not be responsible for the impact of those conditions on the geotechnical recommendations for the project.

PSI warrants that the findings, recommendations, specifications, or professional advice contained herein have been made in accordance with generally accepted professional geotechnical engineering practices in the local area at the date of this report. No other warranties are implied or expressed.

No entity can be as familiar with the design concepts inherent in these recommendations as PSI. Accordingly, only observations by PSI can permit PSI to finalize its recommendations and enhance the likelihood of the design concept being adequately considered during implementation of its recommendations.

After the plans and specifications are more complete, PSI should be retained and provided the opportunity to review the final design plans and specifications to check that our engineering recommendations have been properly incorporated into the design documents. At that time, it may be necessary to submit supplementary recommendations. This report has been prepared for the exclusive use of **EnSiteUSA** and its clients for the specific application to construction of the proposed **POD Facility Project,** located in Morgan County, West Virginia.

******

APPENDIX A: IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL REPORT

Important Information about Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

While you cannot eliminate all such risks, you can manage them. The following information is provided to help.

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

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

Read the Full Report

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

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

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

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

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

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

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

Subsurface Conditions Can Change

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

Most Geotechnical Findings Are Professional Opinions

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

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual

subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

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

Do Not Redraw the Engineer's Logs

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

Give Contractors a Complete Report and Guidance

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

Read Responsibility Provisions Closely

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

Geoenvironmental Concerns Are Not Covered

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

Obtain Professional Assistance To Deal with Mold

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

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

Membership in ASFE/THE BEST PEOPLE ON EARTH exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE-member geotechnical engineer for more information.

ASFE
THE BEST PEOPLE ON EARTH

8811 Colesville Road/Suite G106, Silver Spring, MD 20910
Telephone: 301/565-2733 Facsimile: 301/589-2017
e-mail: info@asfe.org www.asfe.org

APPENDIX B – VICINITY MAP AND BORING LOCATION PLAN

LEGEND:

● B-1 - PROPOSED BORING

(10 FT) - BORING DEPTH

NOTES:

1. ALL BORINGS WERE ADVANCED WITH HOLLOW-STEM AUGERS.
2. SPT SAMPLING WAS BE PERFORMED IN ALL BORINGS.
3. BORING DEPTHS ARE AS SHOWN
4. BORING SPOILS WERE USED TO BACKFILL THE BORE HOLES.

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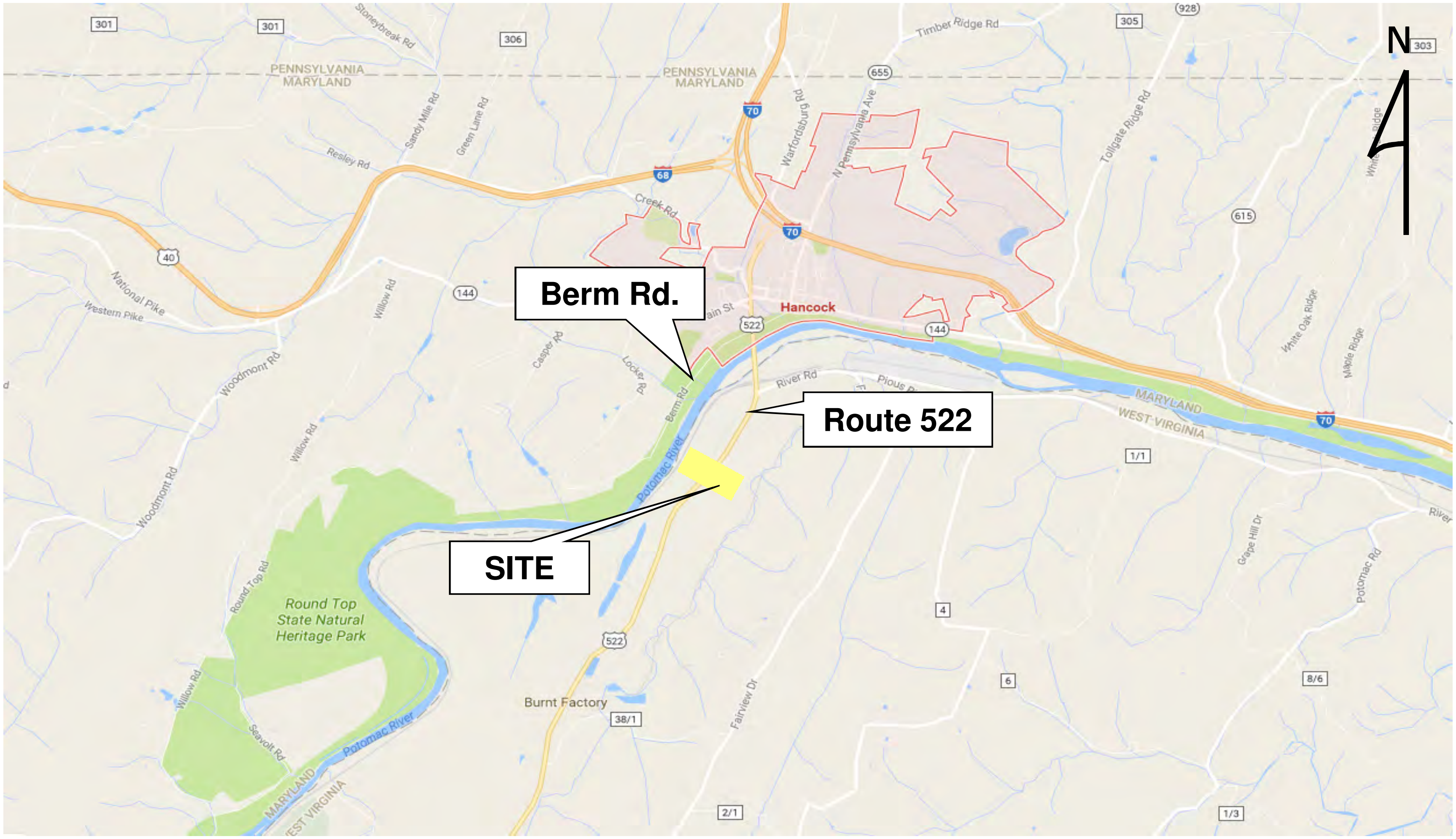
REVISIONS

BORING LOCATION PLAN (FIGURE 1B)
6493- EASTERN PANHANDLE EXPANSION POTOMAC RIVER CROSSING

WASHINGTON COUNTY, MD AND MORGAN COUNTY, WV		January 11, 2017
L.D.P.	N.T.S.	0512713-2

Map Source: Google



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REVISIONS

Site Vicinity Map (Figure 1A)
6493- Eastern Panhandle Expansion Potomac River Crossing

Morgan County, WV		January 11, 2017
L.D.P.	Not Drawn To Scale	0512713-2

APPENDIX C: BORING LOGS

DATE STARTED: 12/14/16
DATE COMPLETED: 12/14/16
COMPLETION DEPTH 40.0 ft
BENCHMARK: N/A
ELEVATION: 590 ft
LATITUDE: 39.678878°
LONGITUDE: 78.194106°
STATION: N/A **OFFSET:** N/A
REMARKS:

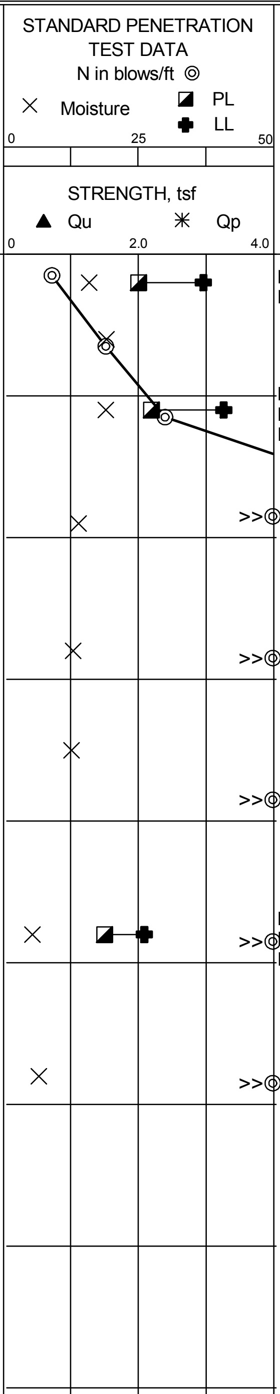
DRILL COMPANY: Connelly Drilling, Inc.
DRILLER: Kevin Kersh **LOGGED BY:** Gunner Ingram
DRILL RIG: CME 550 ATV
DRILLING METHOD: Hollow Stem Auger
SAMPLING METHOD: 2-in SS1.874-in Core Standard
HAMMER TYPE: Automatic
EFFICIENCY N/A
REVIEWED BY: Lubomir Peytchev

BORING GO-6

Water		
▽ While Drilling	29.5	feet
▼ Upon Completion	N/A	feet
▼ Delay	N/A	feet

BORING LOCATION:

Elevation (feet)	Depth, (feet)	Sample No.	Recovery (inches)	MATERIAL DESCRIPTION	USCS Classification	SPT Blows per 6-inch (SS) RQD & Recovery % (NX)	Moisture, %	Additional Remarks
	0	1	10	Approximately 4 inches of Topsoil Loose to medium dense, moist, brown, **silty SAND (USCS SM)** with shale fragments, gravel, roots.	Top Soil SM	2-3-6 N=9	16	LL = 37 PL = 25
		2	18	Medium dense to very dense mottled brown **silty SAND (USCS SM)** with gravel, shale fragments **Residuum**		7-6-13 N=19	19	
585	5	3	12			10-13-17 N=30	19	LL = 41 PL = 27 Fines=34.5%
580	10	4	18		SM	19-36-48 N=84	14	
575	15	5	10			25-50/4"	13	
							13	
570	20	6	4	Weathered Rock, SHALE and LIMESTONE, sampled as very hard, moist, dark gray, black **silty GRAVEL (USCS GM) with sand**		50/4"		
565	25	7	4		GM	50/4"	5	LL = 26 PL = 19 Fines=13.3%
		8	4 ▽			50/4"	7	
560	30	9	60	Interbedded, weathered, medium bedded to thin bedded, gray, dark gray , black fine grained to medium grained, very soft to soft SHALE and hard LIMESTONE, dip of 30 to 45 degrees, (RQD from 25% to 0 %), Devonian [Marcellus Formation and Needmore Shale]	Shale and Limestone	RQD=25 Rec=100%		
555	35	10	42			RQD=0 Rec=70%		
550	40			Bottom of test boring at 40 feet				



Professional Service Industries, Inc.
2930 Eskridge Rd
Fairfax, VA 22031
Telephone: (703) 698-9300

PROJECT NO.: 0512713-1
PROJECT: 6493-Eastern Panhandle Expansion
LOCATION: Potomac River Crossing
Morgan County
West Virginia

The stratification lines represent approximate boundaries. The transition may be gradual.

GENERAL NOTES

SAMPLE IDENTIFICATION

The Unified Soil Classification System (USCS), AASHTO 1988 and ASTM designations D2487 and D-2488 are used to identify the encountered materials unless otherwise noted. Coarse-grained soils are defined as having more than 50% of their dry weight retained on a #200 sieve (0.075mm); they are described as: boulders, cobbles, gravel or sand. Fine-grained soils have less than 50% of their dry weight retained on a #200 sieve; they are defined as silts or clay depending on their Atterberg Limit attributes. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size.

DRILLING AND SAMPLING SYMBOLS

SFA: Solid Flight Auger - typically 4" diameter flights, except where noted.
HSA: Hollow Stem Auger - typically 3¼" or 4¼ I.D. openings, except where noted.
M.R.: Mud Rotary - Uses a rotary head with Bentonite or Polymer Slurry
R.C.: Diamond Bit Core Sampler
H.A.: Hand Auger
P.A.: Power Auger - Handheld motorized auger

☒ SS: Split-Spoon - 1 3/8" I.D., 2" O.D., except where noted.
■ ST: Shelby Tube - 3" O.D., except where noted.
▯ RC: Rock Core
⬇ TC: Texas Cone
✋ BS: Bulk Sample
▨ PM: Pressuremeter
CPT-U: Cone Penetrometer Testing with Pore-Pressure Readings

SOIL PROPERTY SYMBOLS

N: Standard "N" penetration: Blows per foot of a 140 pound hammer falling 30 inches on a 2-inch O.D. Split-Spoon.
N_{60}: A "N" penetration value corrected to an equivalent 60% hammer energy transfer efficiency (ETR)
Q_u: Unconfined compressive strength, TSF
Q_p: Pocket penetrometer value, unconfined compressive strength, TSF
w%: Moisture/water content, %
LL: Liquid Limit, %
PL: Plastic Limit, %
PI: Plasticity Index = (LL-PL),%
DD: Dry unit weight, pcf
▼, ▽, ▼: Apparent groundwater level at time noted

RELATIVE DENSITY OF COARSE-GRAINED SOILS

Relative Density	N - Blows/foot
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	50 - 80
Extremely Dense	80+

ANGULARITY OF COARSE-GRAINED PARTICLES

Description	Criteria
Angular:	Particles have sharp edges and relatively plane sides with unpolished surfaces
Subangular:	Particles are similar to angular description, but have rounded edges
Subrounded:	Particles have nearly plane sides, but have well-rounded corners and edges
Rounded:	Particles have smoothly curved sides and no edges

GRAIN-SIZE TERMINOLOGY

Component	Size Range
Boulders:	Over 300 mm (>12 in.)
Cobbles:	75 mm to 300 mm (3 in. to 12 in.)
Coarse-Grained Gravel:	19 mm to 75 mm (¾ in. to 3 in.)
Fine-Grained Gravel:	4.75 mm to 19 mm (No.4 to ¾ in.)
Coarse-Grained Sand:	2 mm to 4.75 mm (No.10 to No.4)
Medium-Grained Sand:	0.42 mm to 2 mm (No.40 to No.10)
Fine-Grained Sand:	0.075 mm to 0.42 mm (No. 200 to No.40)
Silt:	0.005 mm to 0.075 mm
Clay:	<0.005 mm

PARTICLE SHAPE

Description	Criteria
Flat:	Particles with width/thickness ratio > 3
Elongated:	Particles with length/width ratio > 3
Flat & Elongated:	Particles meet criteria for both flat and elongated

RELATIVE PROPORTIONS OF FINES

Descriptive Term	% Dry Weight
Trace:	< 5%
With:	5% to 12%
Modifier:	>12%

GENERAL NOTES

(Continued)

CONSISTENCY OF FINE-GRAINED SOILS

Q_U - TSF	N - Blows/foot	Consistency
0 - 0.25	0 - 2	Very Soft
0.25 - 0.50	2 - 4	Soft
0.50 - 1.00	4 - 8	Firm (Medium Stiff)
1.00 - 2.00	8 - 15	Stiff
2.00 - 4.00	15 - 30	Very Stiff
4.00 - 8.00	30 - 50	Hard
8.00+	50+	Very Hard

MOISTURE CONDITION DESCRIPTION

Description	Criteria
Dry:	Absence of moisture, dusty, dry to the touch
Moist:	Damp but no visible water
Wet:	Visible free water, usually soil is below water table

RELATIVE PROPORTIONS OF SAND AND GRAVEL

Descriptive Term	% Dry Weight
Trace:	< 15%
With:	15% to 30%
Modifier:	>30%

STRUCTURE DESCRIPTION

Description	Criteria
Stratified:	Alternating layers of varying material or color with layers at least ¼-inch (6 mm) thick
Laminated:	Alternating layers of varying material or color with layers less than ¼-inch (6 mm) thick
Fissured:	Breaks along definite planes of fracture with little resistance to fracturing
Slickensided:	Fracture planes appear polished or glossy, sometimes striated

Description	Criteria
Blocky:	Cohesive soil that can be broken down into small angular lumps which resist further breakdown
Lensed:	Inclusion of small pockets of different soils
Layer:	Inclusion greater than 3 inches thick (75 mm)
Seam:	Inclusion 1/8-inch to 3 inches (3 to 75 mm) thick extending through the sample
Parting:	Inclusion less than 1/8-inch (3 mm) thick

SCALE OF RELATIVE ROCK HARDNESS

Q_U - TSF	Consistency
2.5 - 10	Extremely Soft
10 - 50	Very Soft
50 - 250	Soft
250 - 525	Medium Hard
525 - 1,050	Moderately Hard
1,050 - 2,600	Hard
>2,600	Very Hard

ROCK BEDDING THICKNESSES

Description	Criteria
Very Thick Bedded	Greater than 3-foot (>1.0 m)
Thick Bedded	1-foot to 3-foot (0.3 m to 1.0 m)
Medium Bedded	4-inch to 1-foot (0.1 m to 0.3 m)
Thin Bedded	1¼-inch to 4-inch (30 mm to 100 mm)
Very Thin Bedded	½-inch to 1¼-inch (10 mm to 30 mm)
Thickly Laminated	1/8-inch to ½-inch (3 mm to 10 mm)
Thinly Laminated	1/8-inch or less "paper thin" (<3 mm)

ROCK VOIDS

Voids	Void Diameter
Pit	<6 mm (<0.25 in)
Vug	6 mm to 50 mm (0.25 in to 2 in)
Cavity	50 mm to 600 mm (2 in to 24 in)
Cave	>600 mm (>24 in)

GRAIN-SIZED TERMINOLOGY

(Typically Sedimentary Rock)

Component	Size Range
Very Coarse Grained	>4.76 mm
Coarse Grained	2.0 mm - 4.76 mm
Medium Grained	0.42 mm - 2.0 mm
Fine Grained	0.075 mm - 0.42 mm
Very Fine Grained	<0.075 mm

ROCK QUALITY DESCRIPTION

Rock Mass Description	RQD Value
Excellent	90 -100
Good	75 - 90
Fair	50 - 75
Poor	25 -50
Very Poor	Less than 25

DEGREE OF WEATHERING

Slightly Weathered:	Rock generally fresh, joints stained and discoloration extends into rock up to 25 mm (1 in), open joints may contain clay, core rings under hammer impact.
Weathered:	Rock mass is decomposed 50% or less, significant portions of the rock show discoloration and weathering effects, cores cannot be broken by hand or scraped by knife.
Highly Weathered:	Rock mass is more than 50% decomposed, complete discoloration of rock fabric, core may be extremely broken and gives clunk sound when struck by hammer, may be shaved with a knife.

SOIL CLASSIFICATION CHART

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

MAJOR DIVISIONS			SYMBOLS		TYPICAL DESCRIPTIONS
			GRAPH	LETTER	
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)	●	GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
			○	GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)	○	GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
			▨	GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)	⁘	SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
			░	SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)	░	SM	SILTY SANDS, SAND - SILT MIXTURES
			▨	SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	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, SILTY CLAYS, LEAN 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 SAND OR SILTY SOILS
			▨	CH	INORGANIC CLAYS OF HIGH PLASTICITY
			≈	OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			⁂	PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

APPENDIX D: LABORATORY TESTING RESULTS

Laboratory Summary Sheet

Sheet 1 of 1

Borehole	Approx. Depth	Liquid Limit	Plastic Limit	Plasticity Index	Qu (tsf)	%<#200 Sieve	Est. Specific Gravity	Water Content (%)	Dry Density (pcf)	Satur-ation (%)	Void Ratio
GO-6	1	37	25	12				16			
GO-6	3							19			
GO-6	5.5	41	27	14		34.5%		19			
GO-6	9.5							14			
GO-6	14							13			
GO-6	17.5							13			
GO-6	24	26	19	7		13.3%		5			
GO-6	29							7			

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Fairfax, VA 22031
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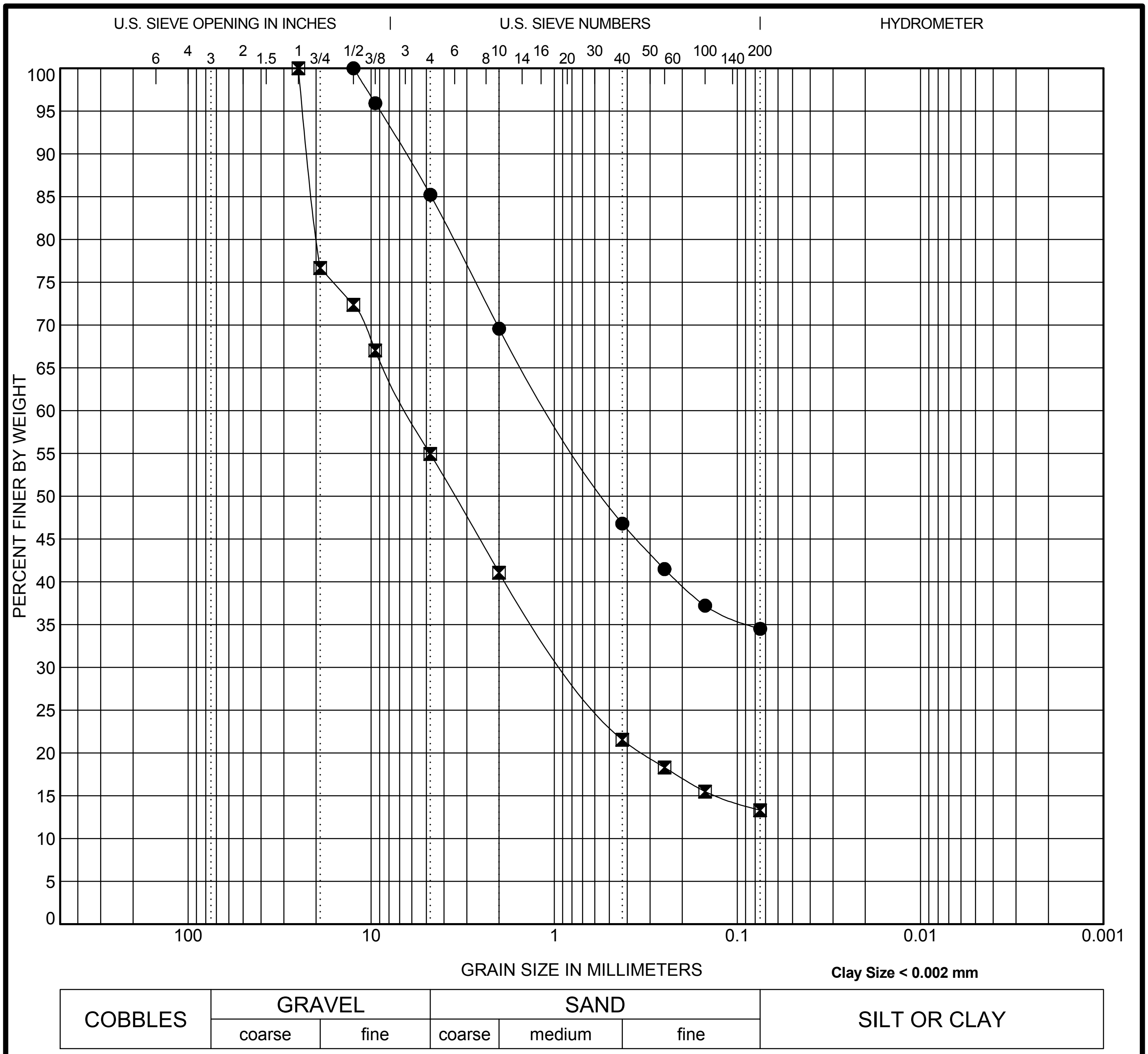
Fax: (703) 560-7931

Summary of Laboratory Results

PSI Job No.: 0512713-1
Project: 6493-Eastern Panhandle Expansion
Location: Potomac River Crossing
Washington County
Hancock, MD



Specimen Identification			Classification	LL	PL	PI	Cc	Cu
●	**GO-6**	**5.5**	Silty SAND (USGS SM)with gravel	**41**	**27**	**14**		
☒	**GO-6**	**24.0**	Silty GRAVEL (USGS GM)with sand	**26**	**19**	**7**		

Specimen Identification			D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**GO-6**	**5.5**	**12.5**	**1.043**			**14.8**	**50.7**	**34.5**	
☒	**GO-6**	**24.0**	**25**	**6.352**	**0.832**		**45.1**	**41.6**	**13.3**	

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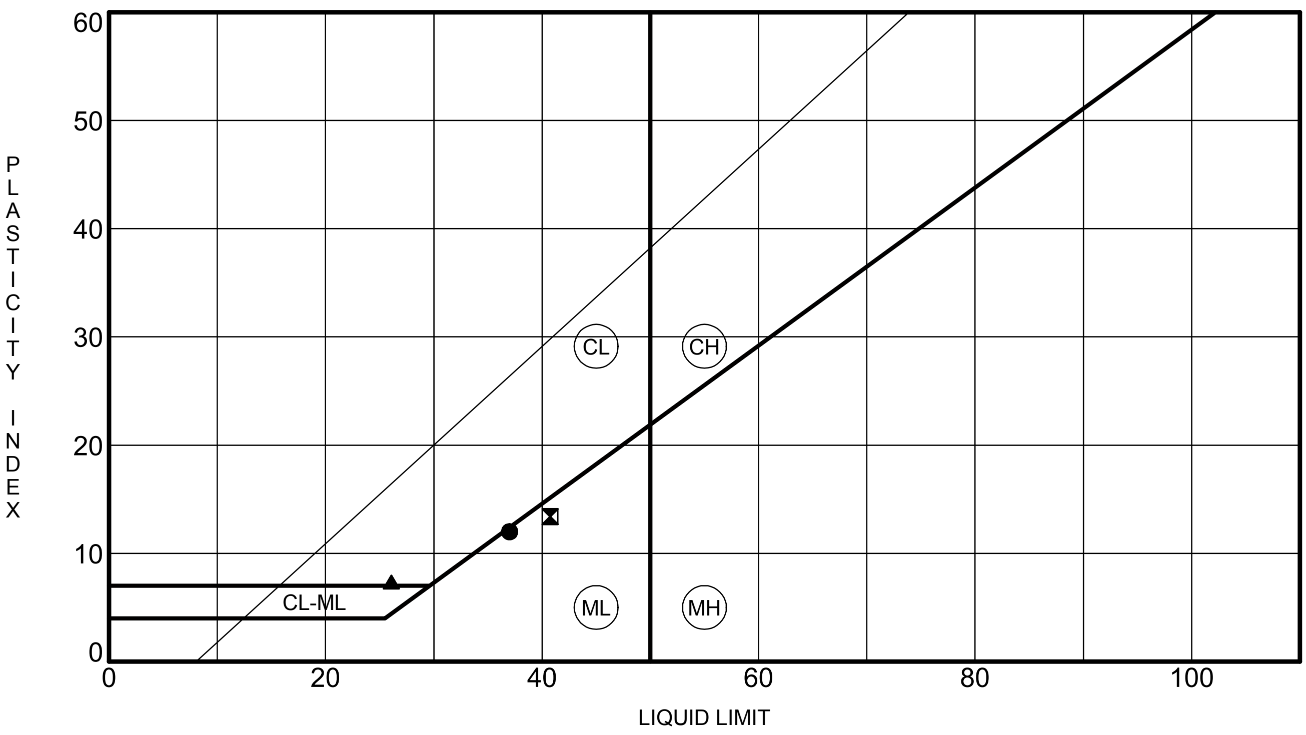
GRAIN SIZE DISTRIBUTION

Project: 6493-Eastern Panhandle Expansion
PSI Job No.: 0512713-1
Location: Potomac River Crossing
Washington County



	Boring	Depth (ft)	LL	PL	PI	Fines	Classification (*Visual)
●	**GO-6**	**1.0**	**37**	**25**	**12**		Sandy SILT (USGS ML)
⧗	**GO-6**	**5.5**	**41**	**27**	**14**	**34.5**	Sandy SILT (USGS ML)
▲	**GO-6**	**24.0**	**26**	**19**	**7**	**13.3**	Lean CLAY (USGS CL)

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ATTERBERG LIMIT RESULTS

PSI Job No.:	0512713-1
Project:	6493-Eastern Panhandle Expansion
Location:	Potomac River Crossing Washington County

PARTICLE-SIZE ANALYSIS OF SOILS - ASTM D422

Client	Professional Service Industries, Inc. (PSI)	Boring	GO-6
Client Project	Proposed Eastern Panhandle Expansion I-68 Crossing V	Depth	0.0'-4.0'
Project No.	38212	Sample	ST-1 & ST-2
		Lab Sample	38212002

Sample Color: **YELLOWISH BROWN**
USCS Group Name: **SILTY SAND**
USCS Group Symbol: **SM** USDA: **NA** AASHTO: **A-2-6 (0)**

MECHANICAL SIEVE

Total Sample	
Total Sample Wet Wt, gm (-3")	1324
Sample Split on Sieve	No. 4
Coarse Washed Dry Sample, gm	91
Wet Wt Passing Split, gm	1233
Dry Wt. Passing Split, gm	1064
Total Sample Dry Wt, gm	1155

Split Sample - Passing No. 4	
Tare No.	2073
Tare + WS., gm	839.29
Tare + DS., gm	744.86
Tare, gm	151.78
Water Content of Split Sample	**15.9%**
Wt. of DS., gm	593.08
Wt. of +#200 Sample, gm	408.01

Sieve Size	Nominal Opening, mm	Dry Wt, gm	Split Normalized % Retained	Split Normalized % Finer	Project Specifications
3"	75	0	0.0%	100.0%	
2-1/2"	63	0	0.0%	100.0%	
2"	50	0	0.0%	100.0%	
1-1/2"	37.5	0	0.0%	100.0%	
1"	25	0	0.0%	100.0%	
3/4"	19	0	0.0%	100.0%	
1/2"	12.5	0	0.0%	100.0%	
3/8"	9.5	8.3	0.7%	99.3%	
No. 4	4.75	82.72	7.2%	92.1%	
No. 10	2	140.74	21.9%	70.3%	
No. 20	0.85	128.35	19.9%	50.3%	
No. 40	0.425	63.37	9.8%	40.5%	
No. 60	0.25	32.25	5.0%	35.5%	
No. 140	0.106	33.12	5.1%	30.3%	
No. 200	0.075	10.18	1.6%	28.7%	

USCS SOIL CLASSIFICATION

Corrected For 100% Passing a 3" Sieve

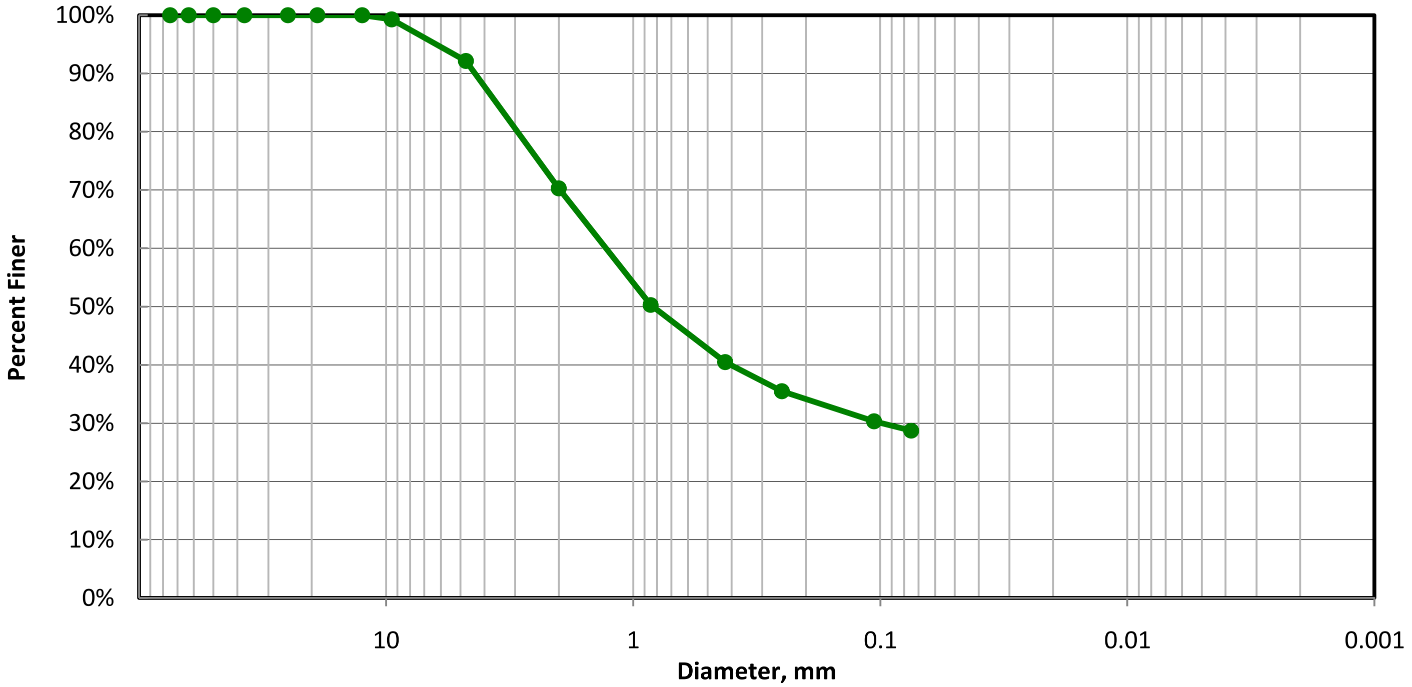
% Gravel (-3" & +#4)	**7.9**	Silt=NA Clay=NA	
Coarse=0; Fine=7.9		D60, mm	NA
% Sand (-#4 & +#200)	**63.4**	D30, mm	NA
Coarse=21.9; Medium=29.8; Fine=11.7		D10, mm	NA
% Fines (-#200)	**28.7**	Cc	NA
% Plus #200 (-3")	**71.3**	Cu	NA

USCS Description

SILTY SAND

USCS Group Symbol	**Atterberg Limits Group Symbol**
SM	**ML - SILT**

Auxiliary Information	Wt Ret, gm	% Retained	% Finer
12" Sieve - 300 mm	0	0.0	100.0
6" Sieve - 150 mm	0	0.0	100.0
3" Sieve - 75 mm	0	0.0	100.0



Input Validation MAK Reviewed By: ALO Date Tested 1/6/2017

LIQUID LIMIT, PLASTIC LIMIT, AND PLASTICITY INDEX OF SOILS
ASTM D 4318

Client	Professional Service Industries, Inc. (PSI)	Boring	GO-6
Client Project	Proposed Eastern Panhandle Expansion I-68 Crossing V	Depth	0.0'-4.0'
Project No.	38212	Sample	ST-1 & ST-2
		Lab Sample	38212002

Soil Description: YELLOWISH BROWN SILT
(-#40 Fraction)

AS-RECEIVED W.C.

Tare Number	2073
Wt. Tare & WS, gm	839.29
Wt. Tare & DS, gm	744.86
Wt. Tare, gm	151.78
Water Content, %	15.9

SAMPLE SUMMARY

C

Liquid Limit (LL), %	**37**
Plastic Limit (PL), %	**25**
Plasticity Index (PI)	**12**
USCS Group Symbol (*-#40 Fraction*)	**ML**
USCS Group Name (*-#40 Fraction*)	**SILT**
Sample Color:	**YELLOWISH BROWN**

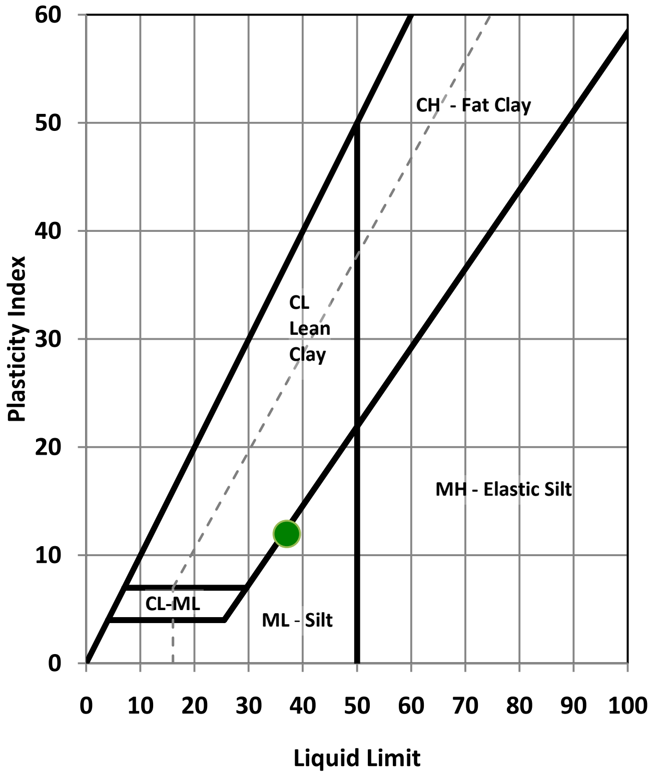
PLASTIC LIMIT

Points Run		3 Points	
Tare Number	709	466	499
Wt. Tare & WS, gm	19.79	17.34	17.54
Wt. Tare & DS, gm	18.34	16.03	16.21
Wt. Tare, gm	12.43	10.74	10.75
Water Content, %	24.5	24.8	24.4

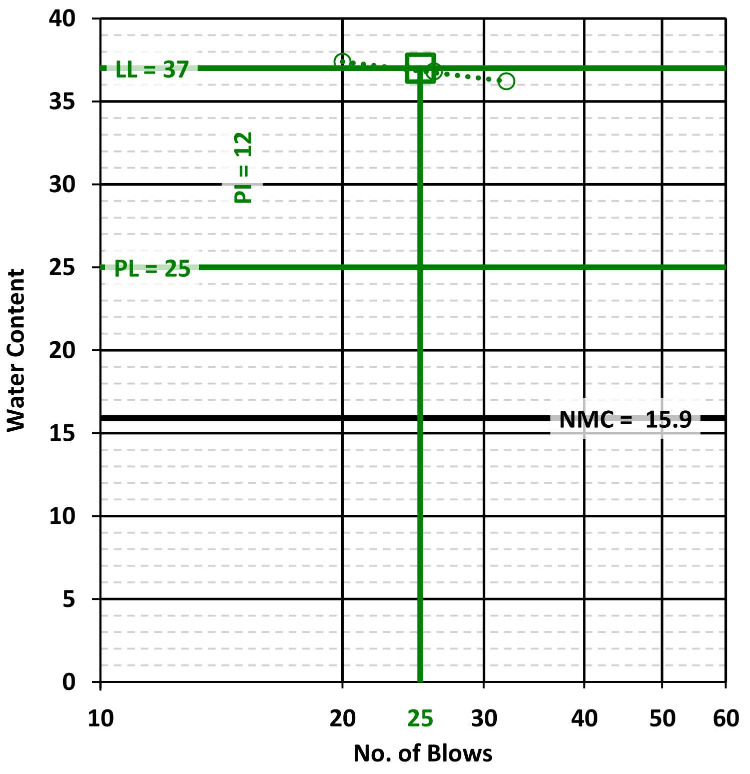
LIQUID LIMIT

		3 Points	
	509	498	444
	16.81	17.51	17.49
	15.15	15.68	15.69
	10.71	10.71	10.72
	37.4	36.8	36.2
# of Blows	20	26	32

PLASTICITY CHART



FLOW CURVE



Input Validation: MAK Reviewed By: ALO Date Tested: 1/7/2017

UNCONSOLIDATED UNDRAINED COMPRESSIVE STRENGTH OF COHESIVE SOIL - ASTM D 2850

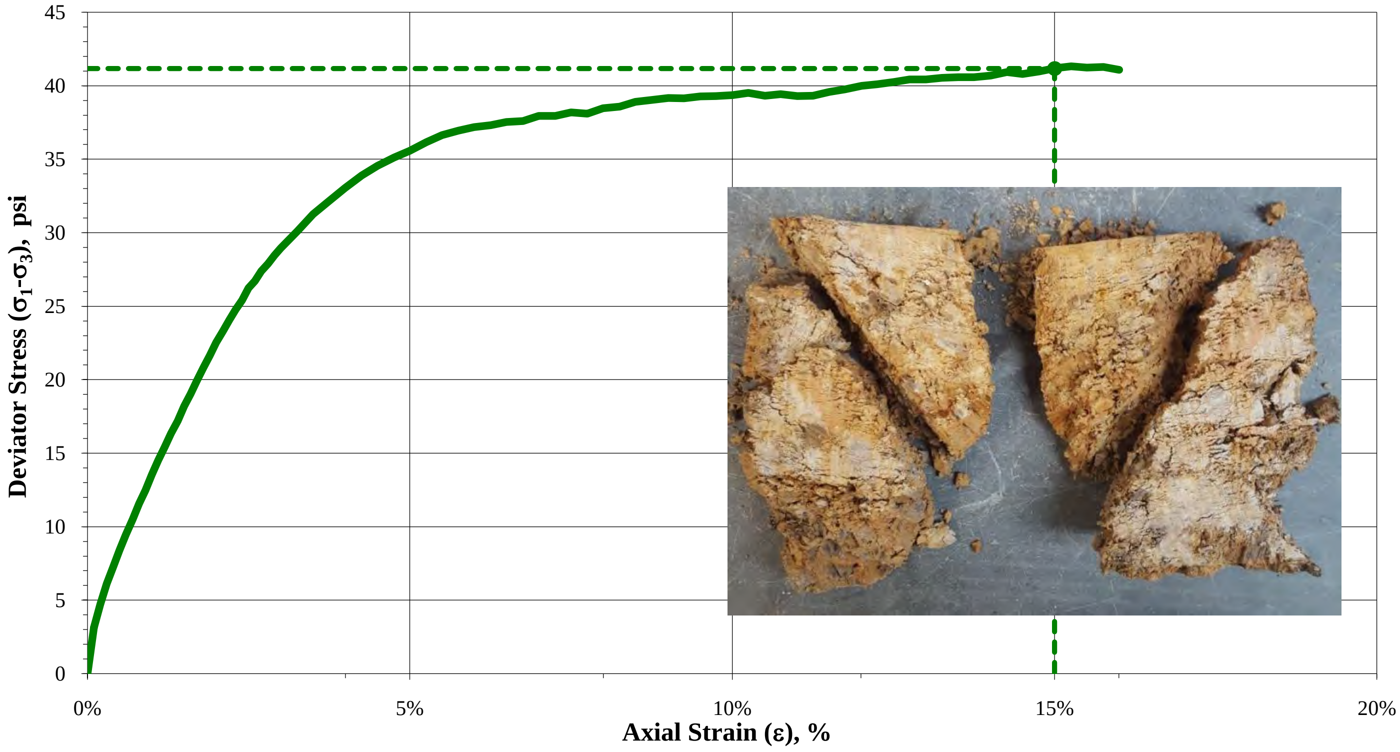
Sheet 3 of 3

Client	Professional Service Industries, Inc. (PS	Boring	GO-6	***Confining Stress (σ_3), psi***	**6.9**
Client No.	Proposed Eastern Panhandle Expansion	Depth	0.0'-4.0'	**UU Strength (q_u), psi**	**41.2**
Project No.	38212	Sample	ST-1 & ST-2	**Shear Strength (S_u), psi**	**20.6**
		Lab ID No.	38212002	**Strain at Failure (ε_f), %**	**15.0%**

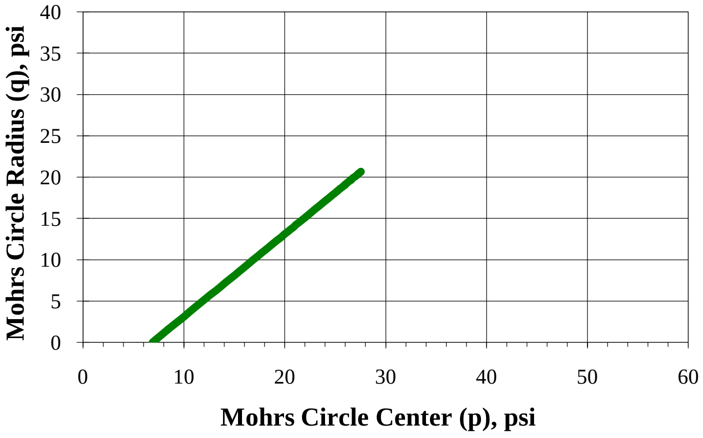
Visual Description: YELLOWISH BROWN SILTY SAND

Sample Condition Undisturbed

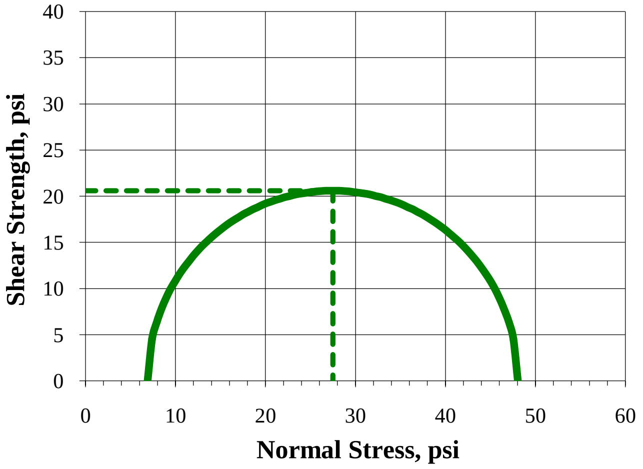
STRESS VS. STRAIN



STRESS PATHS



MOHRS CIRCLE



6.9 psi

UNCONSOLIDATED UNDRAINED COMPRESSIVE STRENGTH OF COHESIVE SOIL - ASTM D 2850

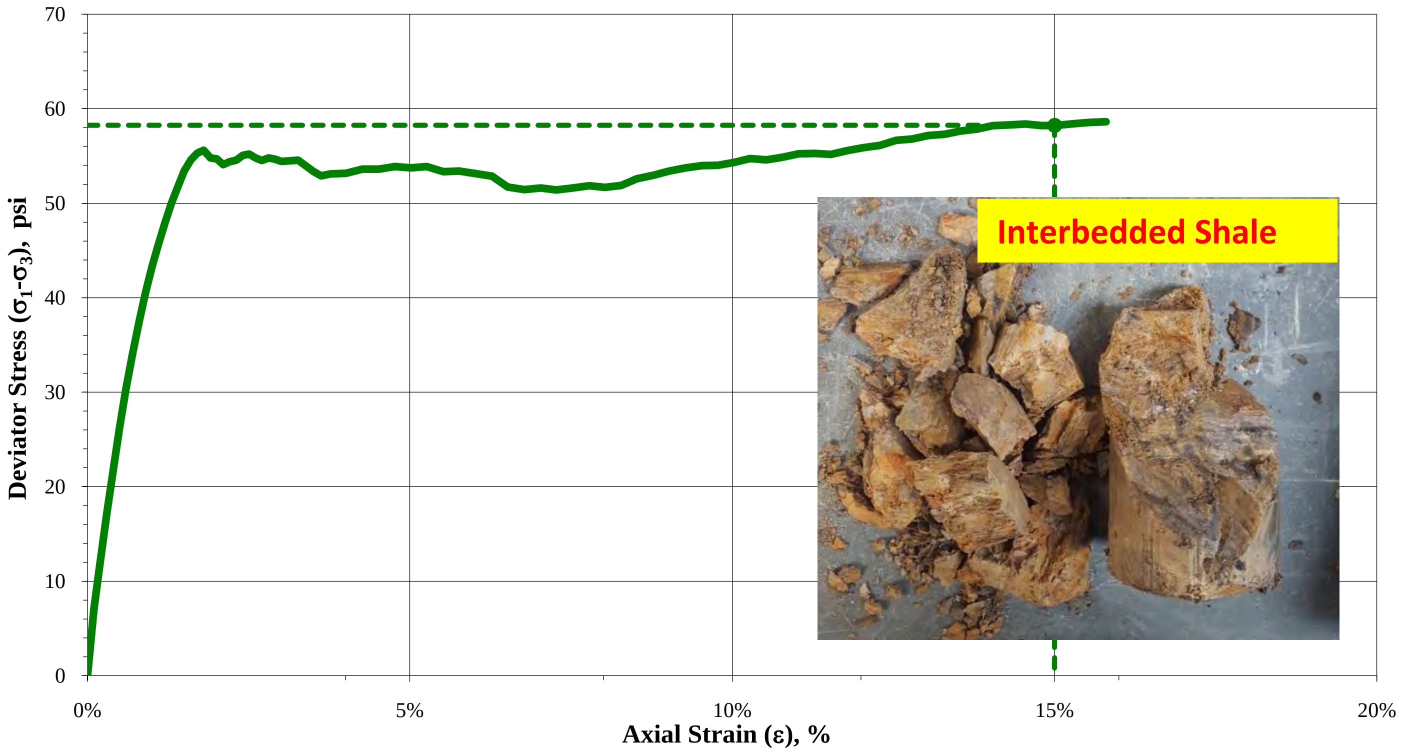
Sheet 3 of 3

Client	Professional Service Industries, Inc. (PS	Boring	GO-6	***Confining Stress (σ_3), psi***	**13.9**
Client No.	Proposed Eastern Panhandle Expansion	Depth	0.0'-4.0'	**UU Strength (q_u), psi**	**58.2**
Project No.	38212	Sample	ST-1 & ST-2	**Shear Strength (S_u), psi**	**29.1**
		Lab ID No.	38212002	**Strain at Failure (ε_f), %**	**15.0%**

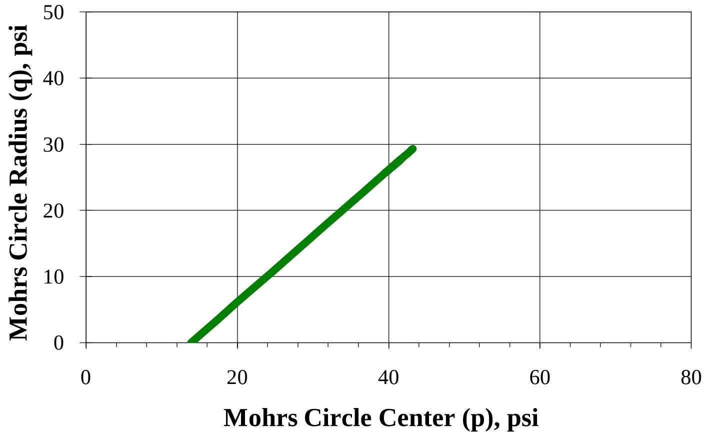
Visual Description: YELLOWISH BROWN SILTY SAND

Sample Condition Undisturbed

STRESS VS. STRAIN



STRESS PATHS



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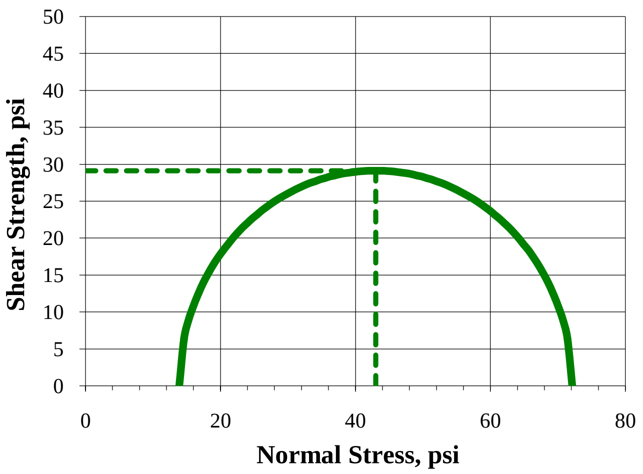


13.9 psi

UNCONSOLIDATED UNDRAINED COMPRESSIVE STRENGTH OF COHESIVE SOIL - ASTM D 2850

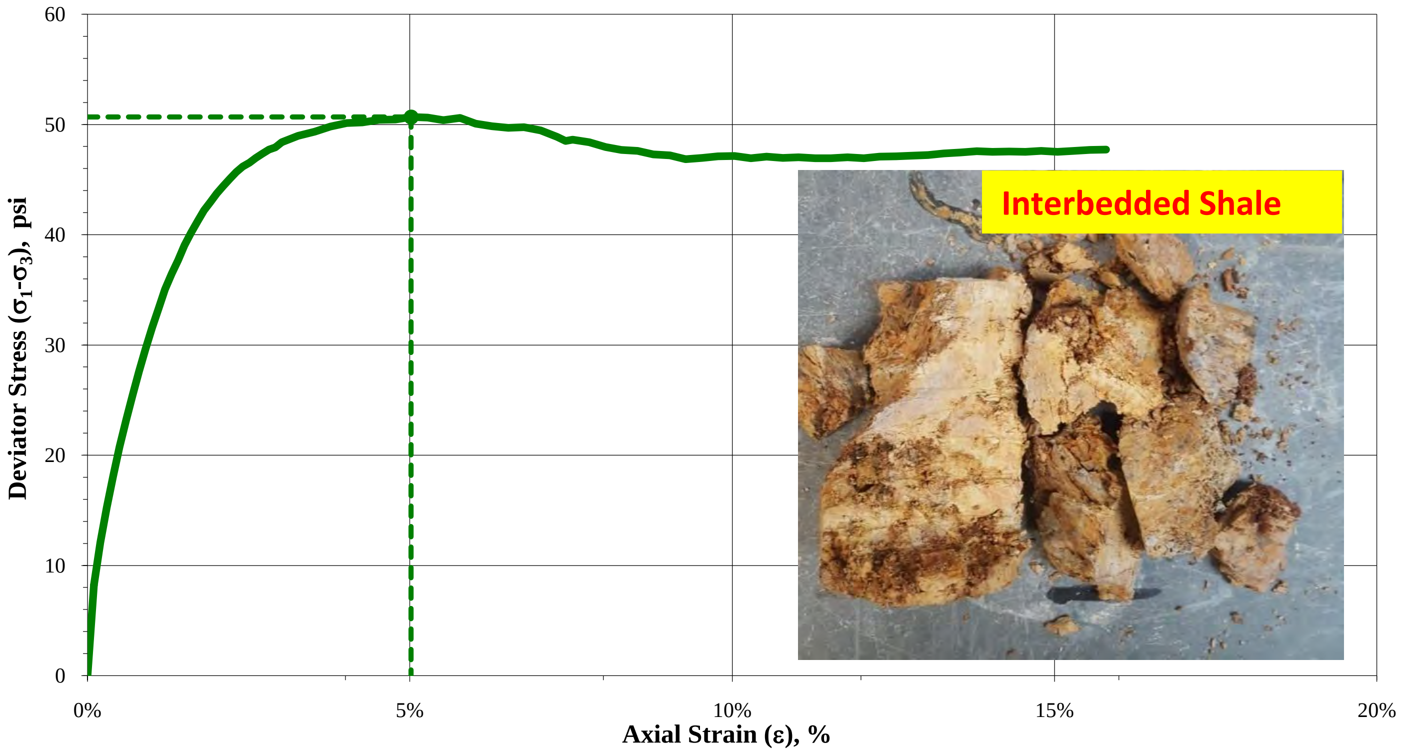
Sheet 3 of 3

Client	Professional Service Industries, Inc. (P	Boring	GO-6	*Confining Stress (σ_3), psi*	**27.8**
Client No.	Proposed Eastern Panhandle Expansion	Depth	0.0'-4.0'	**UU Strength (q_u), psi**	**50.7**
Project No.	38212	Sample	ST-1 & ST-2	**Shear Strength (S_u), psi**	**25.3**
		Lab ID No.	38212002	**Strain at Failure (ε_f), %**	**5.0%**

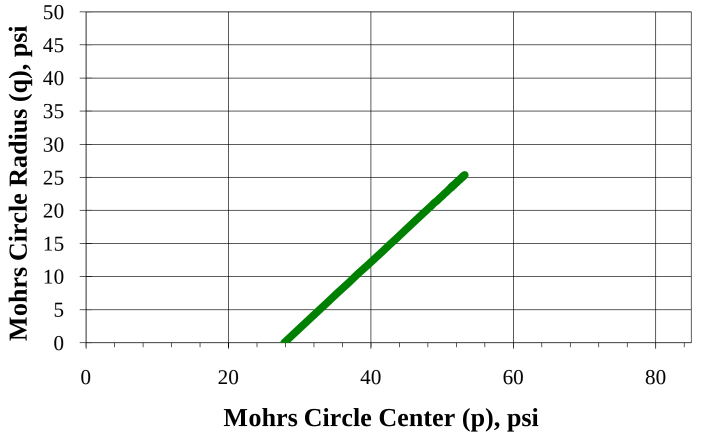
Visual Description: YELLOWISH BROWN SILTY SAND

Sample Condition Undisturbed

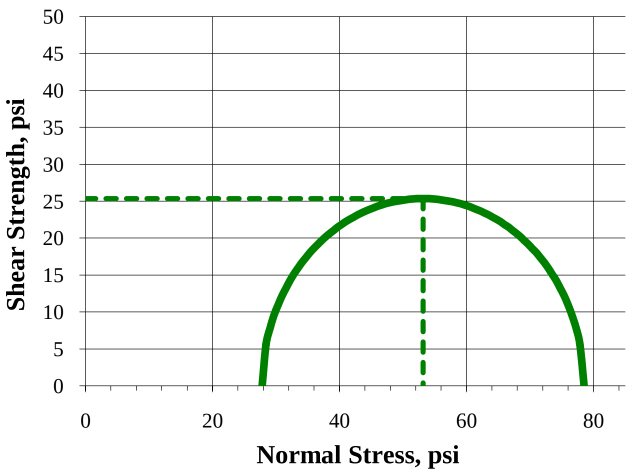
STRESS VS. STRAIN



STRESS PATHS



MOHRS CIRCLE



27.8 psi

Slake Durability Test Results

Slake Durability of Shales and Similar Weak Rocks - ASTM D4644

Client	Professional Service Industries,	Boring	GO-6
Client Project	Proposed Eastern Panhandle Ex	Depth	31.5'-32.5'
Project No.	38212	Sample	RC-9
		Lab Sample No.	38212004

Visual Description: Gray Phyllite/Slate

Initial Water Content

Drum ID	A
Drum + Wet Shale, gm	1733.9
Drum + Dry Shale, gm	1726.4
Drum Wt., gm	1221.9
Water Content, %	1%
Initial Dry Shale Weight, gm	504.5
Water Temerature Before Cycle 1, *C	21.9
Water Temerature After Cycle 1, *C	21.6
Average Temp during Cycle 1, *C	21.75
Drum + Dry Shale after Cycle 1, gm	1720.9
Dry Shale after Cycle 1	499
Slake Durability Index (First cycle)	**98.9%**
Water Temerature Before Cycle 2, *C	18.2
Water Temerature After Cycle 2, *C	18.3
Average Temp during Cycle 2, *C	18.25
Drum + Dry Shale after Cycle 2, gm	1717.8
Dry Shale after Cycle 2	495.9
Slake Durability Index (Second cycle)	**98.3%**

Type II—Retained specimen consist of large and small fragments.

Initial Photograph

Final Photograph

Input Validation: MAK Reviewed By: ALO Date Tested: 1/6/2017

Soil Resistivity Test Results

Corrosivity Testing

Client	Professional Service Industries, Inc. (PSI)
Client Project	Proposed Eastern Panhandle Expansion I-68 Crossing Wash Co MD
Project No.	38212

						pH AASHTO T289			Soil Resistivity AASHTO T-288		
Lab Sample ID	**Boring**	**Depth**	**Sample**	**Sample Received**	**Matrix**	**Result**	Date Tested	Tested By	**Result, Ohm-cm**	Date Tested	Tested By
38212002	GO-6	0.0'-4.0'	ST-1 & ST-2	12/30/2016	Soil	4.4	1/9/2017	TX	18000	1/9/2017	TX

Input Validation: TX

Reviewed By: ALO