Geotechnical Engineering Report

Pedestrian Bridge for Green Circle Jubal Early Drive Trail Extension
City of Winchester, VA

October 10, 2019
Terracon Project No. EY195020

Prepared for:
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Prepared by:
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October 10, 2019

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Re: Geotechnical Engineering Services
Pedestrian Bridge for Green Circle Jubal Early Drive Trail Extension
Harvest Drive and W. Jubal Early Drive
City of Winchester, VA
Terracon Project No. EY195020

Dear Mr. Farrell:

We have completed the Geotechnical Engineering services for the above referenced project. This study was performed in general accordance with Terracon Proposal No. PEY195020 dated April 4, 2019. This report presents the findings of the subsurface exploration and provides geotechnical recommendations concerning earthwork and the design and construction of foundations and floor slabs for the proposed project.

We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report or if we may be of further service, please contact us.

Sincerely,
Terracon Consultants, Inc.

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Note: This report was originally delivered in a web-based format. For more interactive features, please view your project online at client.terracon.com.

ATTACHMENTS

SITE LOCATION AND EXPLORATION PLANS
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Note: Refer to each individual Attachment for a listing of contents.

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INTRODUCTION

The eastern terminus of Green Circle Trail in Winchester, Virginia is located at the Abrams Crossing Shopping plaza, approximately 200 feet northwest of W. Jubal Early Drive and Harvest Drive intersection. The City of Winchester plans to extend the trail further east along W. Jubal Early Drive to approximately 200 feet southeast of Route 11 (Valley Drive) intersection. The entire trail extension project includes the construction of an asphalt paved trail, a pre-fabricated pedestrian bridge, and a wooden boardwalk. The pedestrian bridge will be located immediately adjacent to the vehicular bridge on W. Jubal Early drive span over an existing box culvert that carries Abrams Creek.

The geotechnical engineering Scope of Services for this study is limited to the proposed pedestrian bridge.

This report presents the results of our subsurface exploration and geotechnical engineering services performed for the proposed pedestrian bridge. This report will provide information and geotechnical engineering recommendations relative to:

- Subsurface soil conditions
- Groundwater conditions
- Site preparation and earthwork
- Excavation considerations
- Foundation design and construction
- Seismic site classification
- Lateral earth pressures
- Construction considerations

Maps showing the site and boring locations are shown in the **Site Location** and **Exploration Plan** sections, respectively. The procedures used for performing the field exploration and laboratory testing are presented in the **Exploration and Testing Procedures** section. The results of the laboratory testing performed on soil samples obtained from the site during the field exploration are included on the boring logs and as separate graphs in the **Exploration Results** section. The **Supporting Information** section presents the additional information regarding field exploration and soil classification.

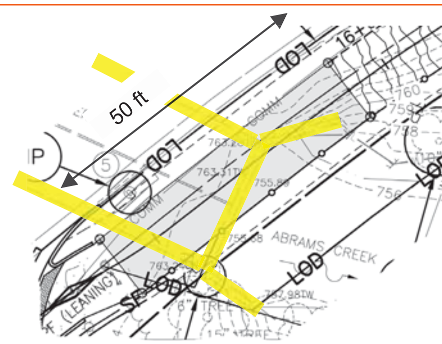
SITE CONDITIONS

The following description of site conditions is derived from our site visit in association with the field exploration and our review of publicly available geologic and topographic maps.

Item	Description
Site Location	The project is located in City of Winchester, Virginia along the southern shoulder of W. Jubal Early Drive near the intersection with Harvest Drive. Refer to Site Location Plan. The Latitude and Longitude for the approximate center of the site are 38.170169°N, 78.182619°W, respectively.
Existing Improvements	Existing construction at and in general vicinity of the site include: ■ W Jubal Early Drive – a four-lane road with concrete curbs, ■ a twin 12 ft x 6 ft concrete box culvert, and ■ a concrete-paved entrance to an adjacent property.
Current Ground Cover	Unpaved grass shoulder, asphalt roadway pavement, concrete pavement
Existing Topography (from USGS)	The roadway and shoulder of the road is relatively flat at an approximate Elevation of EL 766 feet. Beyond the shoulder, the ground surface slopes towards the Abrams Creek.
Geology	Based on the Geologic Map of Winchester Quadrangle, Frederick County, Virginia[1], the project site is underlain by Deposits of the Piedmont physiographic province. The site is mapped to be within the Conococheague formation dominantly with limestone and significant dolostone and sandstone beds located within upper part. Soils encountered generally consisted of localized deposits of clay, and clayey sand with interbedded gravel in colors of gray and brown. City of Winchester is known to have geology prone to karst features. Abrams Creek is a tributary of Opequon Creek and both originate from natural springs. [1] Orndorff, R.C., Weary, D.J., and Parker, R.A., 2004, *Geologic map of the Winchester Quadrangle, Frederick County, Virginia*: U.S. Geological Survey, 2003.

PROJECT DESCRIPTION

Our understanding of the project is based on our discussions with A. Morton Thomas and Associates, Inc. (AMT) during proposal and project planning. Aspects of the project that are assumed are highlighted as shown below and our final understanding of the project conditions are presented as follows:

Item	Description
Project Description	The project includes the construction of a pedestrian bridge over Abrams Creek, which is part of a 10- to 12-ft wide asphalt-paved and wooden boardwalk trail along the south side of the West Jubal Early Drive from approximately 120 ft northwest of Harvest Drive intersection to approximately 200 ft southeast of Valley Avenue.
Proposed Structures	The project consists of a 50-ft long, single-span, pre-fabricated pedestrian bridge (shaded in gray). The pedestrian bridge will span over the existing twin box culvert (highlighted in yellow) and support pedestrians, cyclists and light duty vehicles such as golf carts. The pedestrian bridge will not be designed to support emergency vehicles. 
Grading	We anticipate that the top of the bridge abutments will match the existing grades of the W Jubal Early drive.
Maximum Loads	Based on our experience with similar pre-fabricated bridges with steel railing and 4-inch thick concrete deck, we assume the total loads on each of the bridge abutments will be approximately 100 to 150 kips including live load, dead load, seismic load, and wind loads. We have assumed that per VDOT guidelines, structural design of the project will performed using Load and Resistance Factor Design (LRFD).
Pavements	Pavements associated with the trail are beyond the scope of this report.

GEOTECHNICAL CHARACTERIZATION

The planned field investigation included the advancement of two test borings, one near each bridge abutment, to a depth of 15 feet.

During the field exploration, Boring B-1 encountered both spoon and auger refusals on an obstruction (apparent boulder) at a depth of 1.1 feet. Boring B-1A was offset 8 feet west of B-1 and advanced to a depth of 28 feet to penetrate a thick layer of soft soils, before terminating with spoon refusal. B-2 was advanced to the planned depth of 15 feet.

We have developed a general characterization of the subsurface conditions based upon our review of the subsurface exploration and laboratory data, and our understanding of the geologic setting. This characterization forms the basis of our geotechnical calculations and evaluation of site preparation and foundation options. Conditions encountered at each exploration location are indicated on the individual boring logs (B-1 through B-2) presented in the **Exploration Results** section. Stratification boundaries on the boring logs represent the approximate location of changes in soil types; in situ, the transition between materials may be gradual. The **Exploration and Testing Procedures** section presents the procedures followed during our field exploration.

As part of our analyses, we identified the following layers within the subsurface profile.

Layer	Layer Name	General Description
F	**Existing Fill**	Gray-brown to light brown Silty SAND (SM) with gravel, brown Lean CLAY (CL) with gravel, and brown Fat CLAY (CH) with gravel. Relative densities of the sands vary between very loose to medium dense and the consistencies of the clays vary between stiff to hard.
1	**Fine-grained soils**	Red-brown Lean CLAY (CL) and Fat CLAY (CH) with varying amounts of sands. Consistency varies between medium stiff to very stiff. The soils are generally moist.
2	**Coarse-grained soils**	Light brown Silty SAND (SM) and red-brown Clayey SAND (SC) with varying amounts of fines. Relative density varies from very loose to loose. The soils are generally moist.

All the soils encountered within the field exploration are generally moist.

Groundwater

Groundwater was not encountered in any of the borings drilled; Cave-in depths measured in the boreholes after completion are presented below.

TEST BORING LOCATION	Observed Groundwater Levels	Observed Cave-in depths
	Depth[1] (ft.)	Depth[1] (ft.)
B-1A	Not Encountered	13
B-2	Not Encountered	11

1. Depth below existing ground surface.

Fluctuations in perched or groundwater levels should be expected with variations in conditions such as precipitation, evaporation, construction activity, etc. We have assumed the design groundwater elevation at five feet below the ground surface based on the site conditions.

GEOTECHNICAL OVERVIEW

The following sections present design recommendations, construction considerations, and limitations of the report.

The **Seismic Considerations** section presents seismic site class recommendation.

The **Earthwork** section includes recommendations related to site preparation, fill materials and placement, and site grading and drainage.

The **Foundations** section includes a discussion of anticipated loading conditions, possible foundation systems, along with their advantages and disadvantages, and the recommended foundation alternatives. This section also includes engineering parameters to be used in the design of the foundation and key considerations for design, including frost heave, shrink/swell, karstic ground etc.

The **Lateral Earth Pressure** section provides guidance for the proper calculation of earth and hydrostatic pressures on the bridge abutments. It also includes guidance for proper backfilling behind abutment walls.

The **Construction Considerations** section discusses key issues that the contractor should anticipate or consider during construction, including topics such as excavation support, construction dewatering, observation and testing, existing utilities, and foundation installation.

The **General Comments** section presents the report limitations.

SEISMIC CONSIDERATIONS

The seismic design requirements for bridges and other structures are based on the specifications provided in "LRFD Seismic Analysis and Design of Bridges" reference manual developed by Federal Highway Administration (FHWA). Site Classification is required to determine the Seismic Design Category for a structure. The Site Classification is based on the upper 100 feet of the site profile defined by a weighted average value of either shear wave velocity, standard penetration resistance, or undrained shear strength in accordance with Table 2-3 of FHWA-NHI-15-004 and the International Building Code (IBC) as listed in the table below.

Site Class	v_s (ft/sec)	N or N_{ch}	s_u (psf)
A. Hard rock	>5,000	NA	NA
B. Rock	2,500 to 5,000	NA	NA
C. Very dense soil and soft rock	1,200 to 2,500	>50	>2,000
D. Stiff soil	600 to 1,200	15 to 50	1,000 to 2,000
E. Soft clay soil [1]	<600	<15	<1,000
F. Soils requiring site response analysis	• Soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils, quick and highly sensitive clays, and collapsible weakly cemented soils [2]. • Peats and/or highly organic clays (H>10 ft 3 m), where H=thickness of soil • Very high plasticity clays (H>25 ft with PI>75 • Very thick soft/medium stiff clays (H>120 ft with su<1,000 psf)		

1. Any profile with more than 10 feet of soil having the following characteristics:
   - Plasticity index, PI > 20
   - Moisture content, w > 40 percent
   - Undrained shear strength, su < 500 psf
2. Subject to exceptions stated in Section 20.3.1 in Chapter 20 of ASCE 7.

Based on the soil properties encountered at the site and as described on the exploration logs and results, it is our opinion that the Seismic Site Classification is D. Subsurface explorations at the site were extended to a maximum depth of 28 feet. The site properties below the boring depth to 100 feet were estimated based on our experience and knowledge of geologic conditions of the general area. If a more precise seismic site classification is desired, additional deeper borings or geophysical testing may be performed to confirm the conditions below the deepest current boring depth.

EARTHWORK

All earthwork procedures should conform to Section 303 of the VDOT Road and Bridge Specifications. Earthwork is anticipated to include site preparation such as clearing and grubbing, excavations, and fill placement. The following sections provide recommendations for use in the preparation of specifications for the work.

Site Preparation

Any subsurface utilities and abandoned subsurface structures should be excavated and removed or relocated within and extending laterally at least five feet beyond the limits of the proposed pedestrian bridge. In addition, asphalt and gravel fill encountered within the proposed development should also be stripped within, as well as five feet beyond, the limits of the proposed pedestrian bridge after the removal and relocation of utilities. All areas proposed for cut and fill should be cleared, grubbed, and stripped of existing pavement, topsoil, or any other deleterious material within the proposed limits of pedestrian bridge on the approved plans for this project. Care should be maintained to avoid damaging the existing concrete culvert box while excavating within the proposed limits.

We anticipate that smaller conventional earth-moving equipment will be more suitable for this project due to narrow work space for the excavation.

The Geotechnical Engineer of Record (GEOR) or the GEOR’s authorized representative should evaluate the suitability of the fill subgrades prior to fill placement. In the backfill behind the abutment, subgrade evaluation techniques could include a combination of probing with a penetrometer, drilling hand augers, or observing test pits excavations.

Karst Risk

The geologic setting present at the site is known to produce karst landforms and sinkholes, and the associated risks and challenges are inherent in this setting and cannot be eliminated. However, we note that our borings did not disclose any obvious signs of significant karst activity at the site nor was any obvious indications observed at the ground surface in the immediate vicinity of our boring locations. Any construction in karst topography is accompanied by some risk for future internal soil erosion and ground subsidence (sinkhole development) that could affect the stability of the proposed soil supported structures.

Existing Fill

During the site exploration, existing fill soils were encountered in both borings to depths varying from 7 to 13.5 feet below the existing grades, as seen on the boring logs in **Exploration Results** section. We have no records to indicate the extent of existing fill and whether it was placed in a controlled manner and properly compacted. As such, the fill would be treated as undocumented fill.

Fill Material Types

Fill required to achieve design grade should be classified as structural fill, structural backfill, general fill, and crushed aggregate fill. Earthen materials used for structural and general fill should meet the following material property requirements:

Soil Type [1]	USCS Classification	Additional Requirements	Acceptable Location for Placement
Structural Fill [2]	GW, GP, GM, SW, SP, SM	LL<40 and PI<15 Free of lumps or particles larger than 3 inches	Support of structures and construction of slopes (extending to at least 10 ft from structures)
Structural Backfill	GW, GP, SW, SP	LL<40 and PI<15 <15% passing the No. 200 sieve Free of lumps or particles larger than 3 inches	Abutment wingwall backfill
Crushed aggregate fill	GP	Meeting AASHTO No. 57 aggregate requirements	Drainage blanket

1. A sample of each fill material type should be submitted to the GEOR for evaluation.
2. Coarse-grained soils should contain at least 15% silt or clay "fines". Use of sand without fines risks the creation of perched water near the excavation subgrade as surface water infiltrating the sand becomes trapped above less permeable sandy silt and silty sand.
3. Fine-grained material from on- or off-site borrow sources that classifies as SC, CL or ML should be tested to ensure that the material has a liquid limit and plasticity index less than or equal to those required; and a maximum of 70% passing a U.S. Standard No. 200 sieve.

Based on the soils encountered in test borings and soil laboratory test results, it is expected that the sands of layer 2 excavated at the site will be suitable for re-use as fill based on classification provided the material is free of debris and deleterious materials.

All borrow material, whether on-site or imported from an off-site source, should be tested for suitability and quality prior to its use as fill or backfill. The following tests should be performed to evaluate imported fill material:

Determination of Moisture Content of Soils	ASTM D2216
Particle Size Analysis of Soils	ASTM D422
Atterberg Limits	ASTM D4318
Standard Proctor Test (where applicable)	ASTM D698

Fill Compaction Requirements

Structural and general fill should meet the following compaction requirements.

Item	Requirement
Maximum Lift Thickness	8-inch thick, loose, horizontal layers when heavy, self-propelled compaction equipment is used 4-inch thick, loose, horizontal layers when hand-operated equipment (e.g., jumping jack or plate compactor) is used 12-inch-thick loose, horizontal layers and compacted with two passes of compaction equipment for crushed aggregate fill
Minimum Compaction Requirements [1, 2]	<u>Structural Fill/Structural Backfill:</u> 95% of maximum dry density
Water Content Range [1]	Within the range of +/- 2 percentage points of the optimum moisture content as determined by ASTM D698 at the time of placement and compaction

1. Maximum density and optimum water content as determined by the standard Proctor test (ASTM D698).
2. High plasticity cohesive fill should not be compacted to more than 100% of standard Proctor maximum dry density.

To ensure proper compaction efforts, field density determinations should be performed in accordance with specifications set forth in ASTM D6938 (nuclear method) or D1556 (sand cone method).

Granular soils (i.e., SM or more granular soils) should be compacted with a smooth drum vibratory roller or rubber-tired compactors.

The GEOR, or the GEOR's authorized representative, should complete all required testing and in-situ evaluation to ensure that these materials meet the requirements stated in this section. Soils may be wet or dry of the optimum moisture required for compaction; therefore, scarifying and drying by spreading and aerating or the use of a water truck during construction and prior to their reuse as compacted structural fill or backfill should be expected.

Grading and Drainage

All grades must provide effective drainage away from the bridge during and after construction and should be maintained throughout the life of the structure.

Exposed ground should be sloped and maintained to drain away from the bridge, if feasible. After bridge construction and landscaping have been completed, final grades should be verified to document effective drainage has been achieved. Grades around the structure should also be periodically inspected and adjusted, as necessary, as part of the structure's maintenance program.

FOUNDATIONS

Based on the site plans prepared by AMT dated November 2018, the 50-ft long and 14-ft wide pedestrian bridge will be a pre-fabricated bridge. Based on our experience with similar pedestrian bridges, we have assumed that it will be steel truss bridge with a deck consisting of either wooden planks, fiberglass-reinforced epoxy, or a 4-inch thick concrete slab. We anticipate that the axial bridge loads will be between 100 and 150 kips, depending on the deck selected.

We evaluated compressive shallow foundations to support the pedestrian bridge. However, due to the existing undocumented fill encountered in both borings at depths of 7 feet (B-1) and 13.5 feet (B-2), and very loose to soft soils throughout the site, we do not consider shallow foundations feasible to support the pedestrian bridge abutments. However, shallow foundations may be feasible to support the wing walls as recommended in this section.

Several deep foundation options such as helical piles, auger cast-in place piles, micropiles and driven piles were considered. Deep foundations provide support for the bridge without the need for removing the existing fills or soft soils beneath the abutments.

Advantages and disadvantages of various deep foundation systems considered for this project are presented below.

Foundation Systems	Advantages	Disadvantages	Suitability
Driven piles	■ Can be pre-fabricated off-site and efficiently installed once on-site ■ Superior structural strength – fewer piles needed	■ Installation very noise ■ high vibrations ■ potential damage to existing adjacent structures (like twin culvert)	Not Suitable

Foundation Systems	Advantages	Disadvantages	Suitability
Helical piles	■ Quick installation ■ Suitable for sites with restricted access and low headroom ■ Can be constructed adjacent to existing structures such as culverts without any damage ■ Can be used in areas where high noise and vibration levels are not permitted ■ Can be installed within soils ranging from stiff clay to loose sand ■ Can be installed in locations with shallow groundwater table	■ Installation can be difficult within very dense or gravelly soils as encountered within top 5 feet at this site.	Not Suitable
Micropiles	■ Feasible in restricted access locations ■ Can be used in areas where high noise and vibration levels are not permitted ■ Minimal impact to existing adjacent structures (such as the twin box culvert at this site) ■ Especially suitable for karst areas	■ Relatively expensive	Potentially Suitable
Auger Cast-in Place piles	■ Quick installation ■ Can be used in areas where high noise and vibration levels are not permitted ■ Can be constructed adjacent to existing structures such as culverts without any construction damage	■ Installation generates spoils ■ Installation requires rigorous materials testing and monitoring ■ Requires larger equipment than helical piles and micropiles ■ High mobilization costs	Suitable
Steel H-piles installed in pre-drilled holes	■ Can be used in areas where high noise and vibration levels are not permitted ■ Minimal impact to existing adjacent structures (such as the twin box culvert at this site). ■ High vertical and lateral load carrying capacities.	■ Relatively expensive. ■ High mobilization costs ■ Requires larger equipment than helical piles and micropiles ■ High mobilization costs	Suitable

The final selection should be based on costs and other physical limitations. All deep foundation options listed below are anticipated to have total and differential settlements of less than ½-inch.

Based on the encountered subsurface conditions, laboratory test results, and field penetration test results, engineering properties have been estimated for the soil conditions at the site, as listed below;

Properties for West Abutment based on Boring B-1A

Layer	Top Depth[1] Bottom Depth (feet)	p-y Modulus, k – Static Loading (lb/in^3)	Strain Factor ε_{50}	Effective Unit Weight (pcf)	Angle of Internal Friction ϕ	Cohesion c (psf)
F	0 7	See Note 2	-	115	-	-
Coarse-grained	7 13.5		-	120	28°	-
Fine-grained	13.5 23.5		See Note 3	115	-	1,500
Coarse-grained	23.5 28		-	115	28°	-
Apparent Rock[4]	28 30		-	140	38°	-

1. Depth below ground surface elevation.
2. Allow LPILE to choose parameters based on friction angle/cohesion value provided in the table.
3. Allow LPILE to choose parameters based on cohesion value provided in the table.
4. Rock was not cored to confirm the type of material encountered, material properties are provided per our site knowledge. Allow LPILE to choose parameters based on friction angle/cohesion value provided in the table.

Properties for East Abutment based on Boring B-2

Layer	Top Depth[1] Bottom Depth (feet)	p-y Modulus, k – Static Loading (lb/in^3)	Strain Factor ε_{50}	Effective Unit Weight (pcf)	Angle of Internal Friction ϕ	Cohesion c (psf)
F	0 4	See Note 3	See Note 4	115	-	-
F	4 13.5		-	120	-	-
Fine-grained	13.5 20		See Note 4	115	-	3,000

1. Depth below ground surface elevation.
2. Allow LPILE to choose parameters based on friction angle/cohesion value provided in the table.
3. Allow LPILE to choose parameters based on cohesion value provided in the table.

Auger Cast-In Place Foundation

The proposed pedestrian bridge may also be supported on Auger Cast-in-Place (ACIP) piles. ACIPs are constructed by drilling a hole with continuous flight augers. The shaft will be filled with pressurized grout or concrete as the augers are withdrawn, which causes additional densification of the surrounding soil. The piles should be designed using Service Load Design (SLD) procedures contained in Chapter 5 and 6 of FHWA Geotechnical Circular No. 8, "Augered Cast-in-Place and Continuous Flight Auger Piles".

ACIP piles for the abutments should extend to a minimum of approximately five feet into competent material consisting of relatively dense sands and/or gravels. We expect 22-inch diameter ACIP piles extending to depths ranging from 30 to 35 feet could achieve an ultimate axial design capacity of 200 kips per pile. Approximately two piles are required at each abutment to develop sufficient axial capacities to support the bridge abutment.

Properties for West Abutment based on Boring B-1A

Layer	Top Depth[1] / Bottom Depth (feet)	Ultimate/Nominal[2]	
		Unit Skin Friction (psf)	Unit End Bearing Capacity (psf)
Existing Fill	0 / 7	N/A	N/A
Coarse-grained	7 / 13.5	300	23,000
Fine-grained	13.5 / 23.5	900	13,500
Coarse-grained	23.5 / 28	800	53,000
Apparent Rock[3]	28 / 30	1,400	300,000

1. Depth below ground surface elevation.
2. The upper 2 feet of the pile should be neglected in pile design.
3. Rock was not cored to confirm the type of material encountered, material properties are provided per our site knowledge.

Properties for East Abutment based on Boring B-2

Layer	Top Depth[1] / Bottom Depth (feet)	Ultimate/Nominal	
		Unit Skin Friction (psf)	Unit End Bearing Capacity (psf)
Existing Fill	0 / 4	N/A	N/A
Existing Fill	4 / 13.5	N/A	N/A
Fine-grained	13.5 / 20	1,100	27,000

1. Depth below ground surface elevation.
2. The upper 2 feet of the pile should be neglected in pile design.

Auger cast-in place piles may be designed utilizing the above provided ultimate unit skin friction and ultimate end bearing values with appropriate resistance factors. Based on the Load Resistance Factored Design (LRFD) specified in AASHTO LRFD[1] bridge specifications a resistance factor of 0.45 should be used to calculate the maximum side resistance and a resistance factor of 0.4 should be used for calculating the maximum tip resistance.

Reduction in pile capacity for consideration of group action are unnecessary, provided piles are spaced no closer than three times the diameter of the pile.

LRFD resistance factor should be applied to ultimate capacities unless a pile load-testing program is performed. A representative of the Geotechnical Engineer must be present during all pile installation and load testing activities to record and document construction of each pile.

Micropile Foundation

Micropile foundations can be another alternative to ACIP foundation systems. Micropiles are a deep foundation type consisting of small (generally 5.5- to 12-inch diameter) holes advanced into the subsurface materials using a variety of methods, with steel rod and/or casing reinforcement, and neat cement grout. However, micropiles are more expensive than any alternate foundation systems under typical installation conditions. Therefore, the use of micropiles is usually limited to limited site access conditions.

[1] *Section 10.5.5.2.4 – Drilled Shafts, Section 10: Foundations, AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS, 2012.*

We recommend that the micropile contractor construct the micropile using permanent steel casing at least through the zone of soft soils, in dense materials, the micropile can be advanced with temporary casing or open-hole drilling techniques. Micropiles should be designed constructed in general accordance with the procedures provided by the Federal Highway Administration (FHWA) in "Micropile Design and Construction" (Publication No. FHWA NHI-05-039) by a specialty geotechnical design-build firm experienced in Micropile design and construction.

We expect 5.5-inch diameter micropiles, extending to depths ranging from 30 to 35 feet, could achieve an ultimate axial design capacity of 60 kips per pile. If rock is encountered, micropiles should be socketed into at least 2 feet of rock. Approximately 3 piles are required at each abutment to develop sufficient axial capacities to support the bridge abutment.

Properties for West Abutment based on Boring B-1A

Layer	Top Depth[1] / Bottom Depth (feet)	Ultimate/Nominal[2] Unit Side Resistance (psf)
Existing Fill	0 7	N/A
Coarse-grained	7 13.5	2,000
Fine-grained	13.5 23.5	1,000
Coarse-grained	23.5 28	2,000
Apparent Rock[3]	28 30	31,000

1. Depth below ground surface elevation.
2. The upper 2 feet of the pile should be neglected in pile design.
3. Rock was not cored to confirm the type of material encountered, material properties are provided per our site knowledge.

Properties for East Abutment based on Boring B-2

Layer	Top Depth[1] / Bottom Depth (feet)	Ultimate/Nominal[2] Unit Side Resistance (psf)
Existing Fill	0 4	N/A
Existing Fill	4 13.5	N/A
Fine-grained	13.5 20	2,000

1. Depth below ground surface elevation.
2. The upper 2 feet of the pile should be neglected in pile design.

Micropiles may be designed utilizing the above provided ultimate unit skin friction and ultimate end bearing values with appropriate resistance factors. Based on the Load Resistance Factored Design (LRFD) specified in AASHTO LRFD[2] bridge specifications a resistance factor of 0.55 should be used to calculate the maximum side resistance and a resistance factor of 0.5 should be used for calculating the maximum tip resistance.

H-Piles in Pre-Drilled holes

H-piles installed in pre-drilled holes are another deep foundation alternative to support the bridge abutments. We recommend that steel H-piles be installed in pre-drilled holes drilled to the top of rock. The annular space between the steel H-piles and the rock/soil around it should be backfilled with concrete having a compressive strength of at least 4,000 pounds per square inch (psi).

Parameter	Value
Steel Pile size	HP 8x36
Nominal/ultimate axial capacity of pile	100 kips
Minimum diameter of rock socket	18 inches
Minimum length of rock socket	2 ft

If rock is encountered, H-piles should be socketed into rock to develop the ultimate capacity through end bearing.

Abutment wing walls on Shallow Foundations

Based on the site plans provided by AMT, it is our assumption that a wing wall at the east end of the bridge will be required. The wing wall will be a maximum of about 8 feet in height and will be located at the toe of the existing slope near Abrams Creek. We have not performed soil borings at the toe of the slope to verify the soil information. Based on boring B-2 if soft soils are encountered at the footing elevation, wing walls can be supported on deep foundations consisting of piles. If rock is encountered at shallow depths then the wing walls can be supported on shallow wall footings bearing on rock or very dense natural soils. Shallow footings designed and constructed to bear on subgrades may be proportioned using a maximum net allowable bearing pressure of 2,000 pounds per square foot (psf).

If wing walls are supported on shallow foundations, control Joints should be added between the Abutment and the wing wall for rotational support. Joints are not required of the wing walls are supported on deep foundations.

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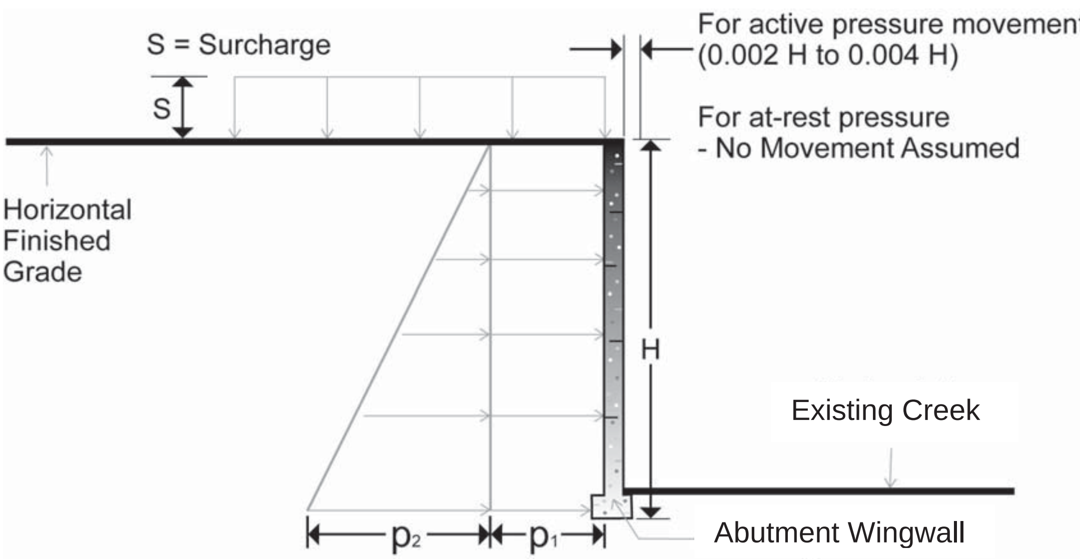
[2] *Section 10.5.5.2.5 – Micropiles, Section 10: Foundations, AASHTO LRFD BRIDGE DESIGN SPECIFICATIONS, 2012.*

These recommendations are based on the consideration that all foundation bearing subgrades will be witnessed and approved as suitable by a qualified geotechnical engineer at the time of construction.

LATERAL EARTH PRESSURES BEHIND ABUTMENTS

If fill is placed to construct the approach grade, abutment wingwalls may be required. The paragraphs below provide guidance on the proper design of wingwalls.

Earth pressures will be influenced by structural design of the abutment wing wall, conditions of wall restraint, methods of construction and/or compaction and the strength of the materials being restrained. Two wall restraint conditions are shown. Active earth pressure is commonly used for design of free-standing cantilever retaining walls and assumes wall movement. The "at-rest" condition is used where the movement at the top of the wall will be restrained. The recommended design lateral earth pressures do not include a factor of safety and do not provide for possible hydrostatic pressure on the walls.



Earth Pressure Coefficients for Structural Backfill

Earth Pressure Conditions	Coefficient of Lateral Earth Pressure	Equivalent Fluid Density (pcf)	Pressure due to Vertical Surcharge, p_1 (psf)	Earth Pressure, p_2 (psf)
At-Rest (Ko)	0.5	60	(0.5)S	(60)H

The recommended earth pressure coefficients above are based on the following assumptions and considerations:

- For active earth pressure, wall must rotate about base, with top lateral movements of about 0.002H to 0.004H, where H is wall height
- A uniform vertical surcharge pressure, S, is acting behind the wall
- In-situ soil or soil backfill total unit weight is a maximum of 120 pcf
- Backfill should be placed in accordance with **Earthwork** section
- Hand-operated/walk behind compaction equipment must be used within a distance equal to the height of the wall or 5 ft, whichever is greater
- A drainage system will be installed so that no hydrostatic pressures is acting on wall. No dynamic loading will be acting on the wall
- Ground behind and in front of the wingwalls will be level

Backfill placed against structures should consist of structural backfill as defined above. The structural backfill must extend out and up from the base of the wall at an angle of at least 45 degrees from vertical for the active case, with a minimum width of four feet. To calculate the resistance to sliding, a value of 0.35 should be used as the *ultimate* coefficient of friction between the foundation and the underlying soil.

The foundations for the wingwalls should bear below anticipated scour depths.

CONSTRUCTION CONSIDERATIONS

Earthwork and Construction Dewatering

Excavations for the proposed structure are anticipated to be accomplished with smaller-sized conventional construction equipment. Upon completion of embankment filling and grading, care should be taken to maintain the subgrade water content prior to construction of abutment. The site should also be graded to prevent ponding of surface water on the prepared subgrades or in excavations. Water collecting over or adjacent to construction areas should be removed. If the subgrade freezes, desiccates, saturates, or is disturbed, the affected material should be removed, or the materials should be scarified, moisture conditioned, and recompacted prior to foundation construction. The contractor should be responsible for reworking of subgrades and compacted structural fill that were initially considered suitable but were later disturbed by equipment and/or weather.

Removal of Existing Utilities

Existing buried utilities should be removed entirely from the bridge abutment footprints and relocated, as necessary.

As a minimum, excavations should be performed in accordance with OSHA 29 CFR, Part 1926, Subpart P, “Excavations” and its appendices, and in accordance with any applicable local, and/or state regulations.

Construction site safety is the sole responsibility of the contractor who controls the means, methods, and sequencing of construction operations. Under no circumstances shall the information provided herein be interpreted to mean Terracon is assuming responsibility for construction site safety, or the contractor's activities; such responsibility shall neither be implied nor inferred.

Construction Observation and Testing

The earthwork efforts should be monitored under the direction of the GEOR. Monitoring should include documentation of adequate removal of existing fill, asphalt, vegetation and topsoil, within the proposed bridge limits.

Each lift of compacted fill should be tested, evaluated, and reworked, as necessary, until approved by the GEOR prior to placement of additional lifts. Each lift of fill should be tested for density and water content at a frequency required by VDOT.

In areas of foundation excavations, the bearing subgrade should be evaluated under the direction of the GEOR. If unanticipated conditions are encountered, the GEOR should prescribe mitigation options.

In addition to the documentation of the essential parameters necessary for construction, the continuation of the GEOR into the construction phase of the project provides the continuity to maintain the GEOR’s evaluation of subsurface conditions, including assessing variations and associated design changes.

Auger Cast-in Place Piles

While advancing the auger to the required depth, it is essential that the auger flights remain filled with soil so that the stability of the hole is maintained. The rig should have adequate torque to maintain the proper rate of penetration.

Some obstructions may be encountered above required pile tip elevations due to obstructions in the fill, and it may be necessary to relocate piles or provide alternate piles. Piles within four diameters of each other should not be installed within 8 hours to allow the initial set of the grout to take place.

The minimum auger center shaft size should be 2-1/2 inches to limit head loss of grout. In addition, the grout should have fluidifier to inhibit early set up, decrease bleeding, eliminate shrinkage, and increase fluidity. The grout pump should be a calibrated high-pressure positive displacement pump equipped with a stroke counter to allow measurement of grout volume per pile. The contractor should establish an accurate method of determining the auger depth at all times, and pressure gauges to allow determination of grout pressure. The auger should have a bottom discharge bit with a discharge opening below the cutter blades.

The CFA pile contractor shall be solely responsible for evaluating the need for, design, and monitoring of measures to prevent damage to existing adjacent culvert or underground utilities, on or of the right-of-way. Construction methods and a detailed work plan and specifications should be prepared by the pile contractor.

Micropiles

Micropiles should be installed by a qualified contractor, experienced in all aspects of micropile design and construction. The micropile contractor should furnish all necessary plant, materials, skilled labor, and supervision to install the micropiles. The micropile contractor should demonstrate a minimum of five years of experience and have experience on at least three projects of similar scope and size in the previous five years.

The micropile contractor should select the drilling methods and grouting procedures used for the installation of the micropiles, subject to the approval of the GEOR. The drilling equipment and methods should be suitable for drilling into the underlying existing fill and bedrock until achieving the required design capacity. Due to the soil conditions encountered during the subsurface exploration, we recommend performing drilling through at least the soft/loose soils under the protection of permanent casing. The micropile contractor should keep complete and accurate records of drilling resistance, drilling fluid level, drilling fluid losses, description of cutting materials, etc.

After drilling, the hole should be flushed with water and/or air to remove drill cuttings and/or other loose debris. Grouting should be performed using a stable, homogeneous, neat cement grout or a sand-cement grout with a minimum 28-day unconfined compressive strength of 4,000 psi. The grout should be injected using a tremie tube from the lowest point of the drill hole until clean, pure grout flows from the top of the micropile. The entire pile should be grouted to the design cut-off level.

The micropile contractor should provide systems and equipment to measure the grout quality and quantity during the grouting operations. Specific gravity measurement should be made for each grout batch using a mud balance. All cement should be Portland cement conforming to ASTM C-150 (AASHTO M85) Type I, Type II, or Type III. The use of Type V cement, for sulfate resistance should not be necessary.

The reinforcing steel may be installed before or after grouting of the micropiles and should consist of threaded bars in accordance with ASTM A-615 (AASHTO M31) Grade 75 or steel casing. Where reinforcing steel is installed after the grout is in place, the contractor should provide means for confirming that the bars are installed to the correct depths. For cases of tensile loading, bar couplers, if required, should develop the ultimate tensile stress of the bar, without any evidence of failure. Centralizers should be provided on bars at 10-ft center maximum spacing on central reinforcement and should permit the free flow of grout without misalignment of the reinforcement. Where casing is used as pile reinforcement, mill certificates or coupon tests should be provided to verify the specified properties of each lot of casing.

The sequence of pile installation should be such as to avoid interconnection or damage to piles in which grout has not achieved final set.

Full-time observation of micropile installations should be made by the GEOR to confirm the consistent installation of the micropile in accordance with the design and results of the load test program.

H-Piles

H-piles supporting the bridge abutments should be installed in pre-drilled holes. The holes should be cleaned to remove loose material and dewatered if water is present. The holes should be drilled to a depth of at least two-ft into bedrock. Piles socketed in bedrock do not need require dynamic pile testing. Piles should be seated at the bottom of the rock socket and verified that the pile tip is founded on rock. Plies should be backfilled with concrete as discussed under the section above for recommendations H-piles in Pre-Drilled Holes. The verticality of the piles should be confirmed prior to, and maintained during, concrete placement.

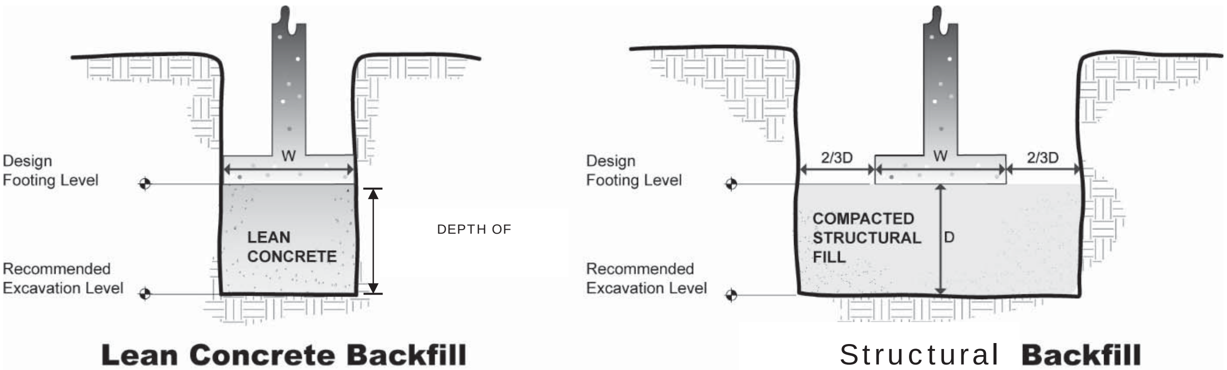
Shallow Foundations

The base of all abutment foundation excavations should be free of water and loose soil and rock prior to placing concrete. Concrete should be placed soon after excavating to reduce the potential for bearing surface disturbance. Should the soils at the bearing level become dry, disturbed or saturated, or frozen, the affected soil should be removed prior to placing concrete. A lean concrete mud-mat should be placed over the bearing soils if the excavations must remain open overnight. We recommend that a geotechnical engineer be retained to observe and confirm the suitability of the foundation bearing materials.

If uncontrolled fill and/or unsuitable bearing soils are encountered in foundation excavations, the excavation should be extended deeper to suitable soils and the footing should bear directly on these soils at the lower level. As an alternative, the footings could also bear on lean concrete or structural backfill extending down to the suitable soils. If structural fill is used, overexcavation should extend laterally beyond all edges of the footings at least eight inches per foot of overexcavation depth below footing base elevation. The overexcavation should then be backfilled up to the footing base elevation with structural fill placed in lifts of eight inches or less in loose thickness (four inches or less if using hand-guided compaction equipment) and compacted to at least 95 percent of the material's maximum dry density (per ASTM D 698).

Alternatively, lean concrete may be used to backfill the overexcavation. Excavation sides for this option can be near-vertical or as steep as possible while maintaining stable excavations sides.

Foundation excavation should not extend within 1(H):1(V) slope outward and downward from the edge of the existing box culvert and wingwall. Due to the proximity of the box culvert to the proposed construction, lean concrete fill option may be preferable because the width of excavation would be minimized. The overexcavation and backfill recommendations are shown in the following figure.



NOTE: Excavations in sketches shown vertical for clarity. Excavations should be sloped as necessary for safety.

GENERAL COMMENTS

Our analysis and opinions are based upon our understanding of the project, the geotechnical conditions in the area, and the data obtained from our site exploration. Natural variations will occur between exploration point locations or due to the modifying effects of construction or weather. The nature and extent of such variations may not become evident until during or after construction. Terracon should be retained as the Geotechnical Engineer, where noted in this report, to provide observation and testing services during pertinent construction phases. If variations appear, we can provide further evaluation and supplemental recommendations. If variations are noted in the absence of our observation and testing services on-site, we should be immediately notified so that we can provide evaluation and supplemental recommendations.

Our Scope of Services does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

Our services and any correspondence or collaboration through this system are intended for the sole benefit and exclusive use of our client for specific application to the project discussed and are accomplished in accordance with generally accepted geotechnical engineering practices with no third-party beneficiaries intended. Any third-party access to services or correspondence is solely for information purposes to support the services provided by Terracon to our client. Reliance upon the services and any work product is limited to our client, and is not intended for third parties. Any use or reliance of the provided information by third parties is done solely at their own risk. No warranties, either express or implied, are intended or made.

Site characteristics as provided are for design purposes and not to estimate excavation cost. Any use of our report in that regard is done at the sole risk of the excavating cost estimator as there may be variations on the site that are not apparent in the data that could significantly impact excavation cost. Any parties charged with estimating excavation costs should seek their own site characterization for specific purposes to obtain the specific level of detail necessary for costing. Site safety, and cost estimating including, excavation support, and dewatering requirements/design are the responsibility of others. If changes in the nature, design, or location of the project are planned, our conclusions and recommendations shall not be considered valid unless we review the changes and either verify or modify our conclusions in writing.

ATTACHMENTS

SITE LOCATION AND EXPLORATION PLANS

Site Location Plan
Exploration Plan

SITE LOCATION

Pedestrian Bridge for Green Circle Jubal Early Drive Trail Extension ■ City of Winchester, VA
October 10, 2019 ■ Terracon Project No. EY195020

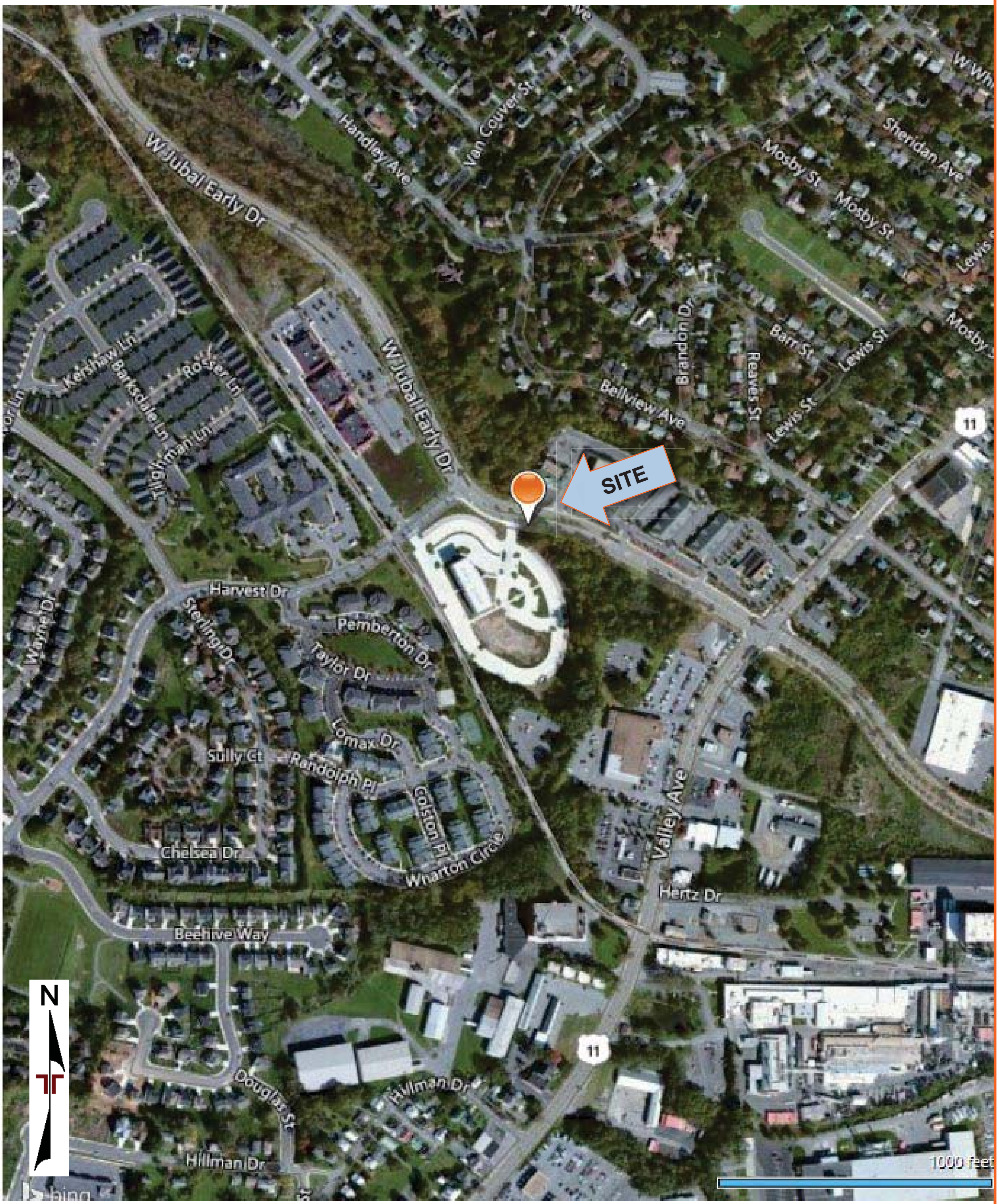


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

MAP PROVIDED BY MICROSOFT BING MAPS

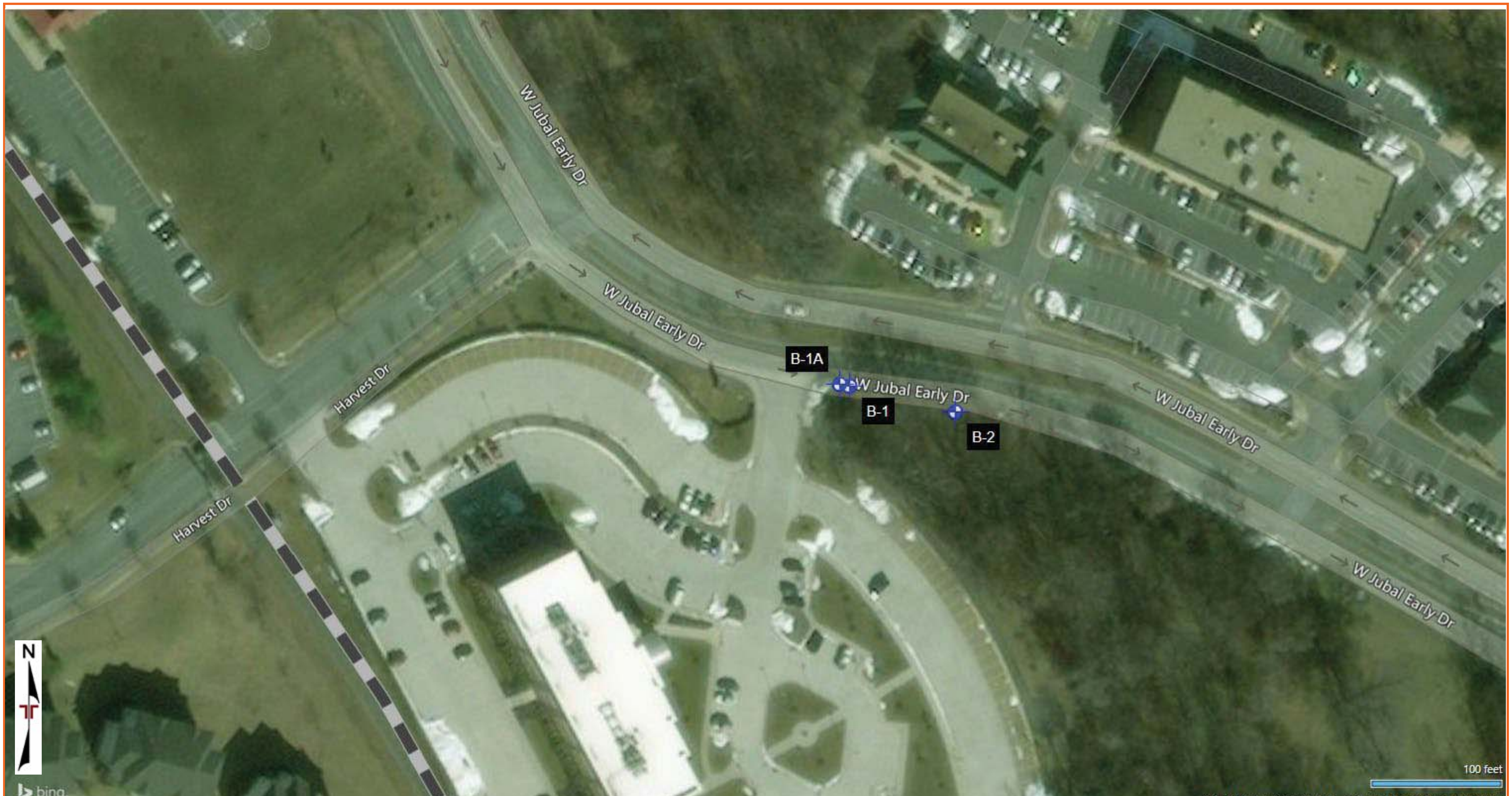


DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

MAP PROVIDED BY MICROSOFT BING MAPS

EXPLORATION AND TESTING PROCEDURES

Field Exploration

The following test boring program was performed:

Borings	Boring Depth (feet)	Planned Location
B-1	1.1	West Abutment
B-1A	28	West Abutment
B-2	15	East Abutment

Boring Layout and Elevations: Terracon personnel provided the boring layout. Coordinates were obtained with a handheld GPS unit (estimated horizontal accuracy of about ±10 feet) and approximate elevations were obtained by interpolation from the from local USGS topographic maps. If elevations and a more precise boring layout are desired, we recommend borings be surveyed following completion of fieldwork.

Subsurface Exploration Procedures: The drilling was performed on April 17th, 2019 by Recon Drilling, Inc. The borings were advanced using a truck-mounted, drill rig CME 45B using continuous flight augers (hollow stem). Five samples were obtained in the upper 10 feet of each boring and at intervals of 5 feet thereafter. In the split-barrel sampling procedure, a standard 2-inch outer diameter split-barrel sampling spoon was driven into the ground by a 140-pound automatic hammer falling a distance of 30 inches. The number of blows required to advance the sampling spoon the middle 12 inches of a continuous 24-inch penetration is recorded as the Standard Penetration Test (SPT) resistance value. The SPT resistance values, also referred to as N-values, are indicated on the boring logs at the test depths. We observed and recorded groundwater levels during drilling and sampling. All borings were backfilled with auger cuttings upon completion. Pavements were patched with Aquaphalt (a quick-setting; permanent asphaltic concrete), as appropriate.

The sampling depths, penetration distances, and other sampling information was recorded on the field boring logs. The samples were placed in appropriate containers and taken to our soil laboratory for testing and classification by a Geotechnical Engineer. Our exploration team prepared field boring logs as part of the drilling operations. These field logs included visual classifications of the materials encountered during drilling and our interpretation of the subsurface conditions between samples. Final boring logs were prepared from the field logs. The final boring logs represent the Geotechnical Engineer's interpretation of the field logs and include modifications based on observations and tests of the samples in our laboratory.

Laboratory Testing

The project engineer reviewed the field data and assigned laboratory tests to understand the engineering properties of the various soil strata, as necessary, for this project. Procedural standards noted below are for reference to methodology in general. In some cases, variations to methods were applied because of local practice or professional judgment. Standards noted below include reference to other, related standards. Such references are not necessarily applicable to describe the specific test performed.

- ASTM D2216 Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
- ASTM D4318 Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils
- ASTM D422 Standard Test Method for Particle-Size Analysis of Soils

The laboratory testing program often included examination of soil samples by an engineer. Based on the material's texture and plasticity, we described and classified the soil samples in accordance with the Unified Soil Classification System.

EXPLORATION RESULTS

Boring Logs (B-1, B-1A, and B-2)
Atterberg Limits
Grain Size Distribution

BORING LOG NO. B-1

PROJECT: Pedestrian Bridge for Green Circle Jubal Early Drive Trail Extension

CLIENT: A Morton Thomas & Assoc Inc, Richmond, VA

SITE: Harvest Drive and W. Jubal Early Drive, Winchester, VA

LOCATION: Latitude: 39.1702° Longitude: -78.1827°

GRAPHIC LOG	DEPTH	DESCRIPTION	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
	0.3	**ASPHALT**				0	50/1"				
	1.1	**AGGREGATE BASE COURSE (GM)** with sand, brown, moist									
		Auger refusal at 1.1 feet on apparent boulder. Offset and redrilled as B-1A.									
		Boring Terminated at 1.1 Feet									

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic
Logged by: Aditya Rayudu

Advancement Method:
3-1/4 ID HSA

Abandonment Method:
Boring backfilled with Auger Cuttings and surface Capped with Asphalt

Notes:

WATER LEVEL OBSERVATIONS

Groundwater not encountered

Terracon
20401 Seneca Meadows Pkwy
Germantown, MD

Boring Started: 04-17-2019	Boring Completed: 04-17-2019
Drill Rig: CME 45B	Driller: Recon Drilling
Project No.: EY195020	

BORING LOG NO. B-1A

PROJECT: Pedestrian Bridge for Green Circle Jubal Early Drive Trail Extension

CLIENT: A Morton Thomas & Assoc Inc, Richmond, VA

SITE: Harvest Drive and W. Jubal Early Drive, Winchester, VA

LOCATION
Latitude: 39.1702° Longitude: -78.1827°

DEPTH	Material Description	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
0.3	**ASPHALT**									
3.0	**FILL - AGGREGATE BASE COURSE (GM)**									
	FILL - SILTY SAND WITH GRAVEL (SM), fine grained, gray-brown, moist, medium dense			X	20	8-8-8-6 N=16				
7.0	very loose	5		X	18	3-3-1-2 N=4				
	SILTY SAND (SM), fine grained, light brown mottled with red, moist, loose			X	24	1-2-3-2 N=5				
	very loose	10		X	24	1-1-2-4 N=3				
13.5			caved in at 13 ft.							
	FAT CLAY (CH), with sand, red-brown, moist, medium stiff	15		X	18	1-2-3 N=5	27		61-21-40	75
23.5		20		X	18	2-2-3 N=5				
	CLAYEY SAND (SC), red-brown, moist, loose	25		X	18	1-2-3 N=5				

Stratification lines are approximate. In-situ, the transition may be gradual.
Offset 8 ft. West from B-1 on W Jubal Early Dr.

Hammer Type: Automatic
Logged by: Aditya Rayudu

Advancement Method:
3-1/4 ID HSA

Abandonment Method:
Boring backfilled with Auger Cuttings and surface Capped with Asphalt

Notes:

WATER LEVEL OBSERVATIONS
Groundwater not encountered
caved in at 13 ft.

Terracon
20401 Seneca Meadows Pkwy
Germantown, MD

Boring Started: 04-17-2019	Boring Completed: 04-17-2019
Drill Rig: CME 45B	Driller: Recon Drilling
Project No.: EY195020	

BORING LOG NO. B-1A

PROJECT: Pedestrian Bridge for Green Circle Jubal Early Drive Trail Extension

CLIENT: A Morton Thomas & Assoc Inc Richmond, VA

SITE: Harvest Drive and W. Jubal Early Drive Winchester, VA

GRAPHIC LOG	LOCATION Latitude: 39.1702° Longitude: -78.1827° DEPTH	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
	CLAYEY SAND (SC), red-brown, moist, loose *(continued)*									
	28.0					50/0"				
	Spoon and Auger Refusal at 28 Feet									

Stratification lines are approximate. In-situ, the transition may be gradual.
Offset 8 ft. West from B-1 on W Jubal Early Dr.

Hammer Type: Automatic
Logged by: Aditya Rayudu

Advancement Method:
3-1/4 ID HSA

Abandonment Method:
Boring backfilled with Auger Cuttings and surface Capped with Asphalt

Notes:

WATER LEVEL OBSERVATIONS

Groundwater not encountered

caved in at 13 ft.

20401 Seneca Meadows Pkwy
Germantown, MD

Boring Started: 04-17-2019	Boring Completed: 04-17-2019
Drill Rig: CME 45B	Driller: Recon Drilling
Project No.: EY195020	

BORING LOG NO. B-2

PROJECT: Pedestrian Bridge for Green Circle Jubal Early Drive Trail Extension

CLIENT: A Morton Thomas & Assoc Inc, Richmond, VA

SITE: Harvest Drive and W. Jubal Early Drive, Winchester, VA

LOCATION: Latitude: 39.1702° Longitude: -78.1824°

DEPTH	Material Description	DEPTH (Ft.)	WATER LEVEL OBSERVATIONS	SAMPLE TYPE	RECOVERY (In.)	FIELD TEST RESULTS	WATER CONTENT (%)	DRY UNIT WEIGHT (pcf)	ATTERBERG LIMITS LL-PL-PI	PERCENT FINES
0.2	**TOPSOIL**									
2.0	**FILL - LEAN CLAY WITH GRAVEL (CL)**, brown, moist, stiff			X	19	2-4-6-6 N=10				
4.0	**FILL - FAT CLAY WITH GRAVEL (CH)**, brown, moist, hard			X	7	4-9-9-50 N=18				
	FILL - SILTY SAND WITH GRAVEL (SM), light brown, moist, loose	5		X	2	2-5-4-10 N=9				
	mottled red- dark brown, moist			X	24	4-3-4-5 N=7	23		40-28-12	19
	brown			X	16	1-3-3-2 N=6				
13.5		10	caved in at 11 ft.							
15.0	**LEAN CLAY (CL)**, red-brown, moist, very stiff	15		X	18	7-10-9 N=19				
	Boring Terminated at 15 Feet									

Stratification lines are approximate. In-situ, the transition may be gradual.

Hammer Type: Automatic
Logged by: Aditya Rayudu

Advancement Method:
3-1/4 ID HSA

Abandonment Method:
Boring backfilled with auger cuttings upon completion.

Notes:

WATER LEVEL OBSERVATIONS

Groundwater not encountered

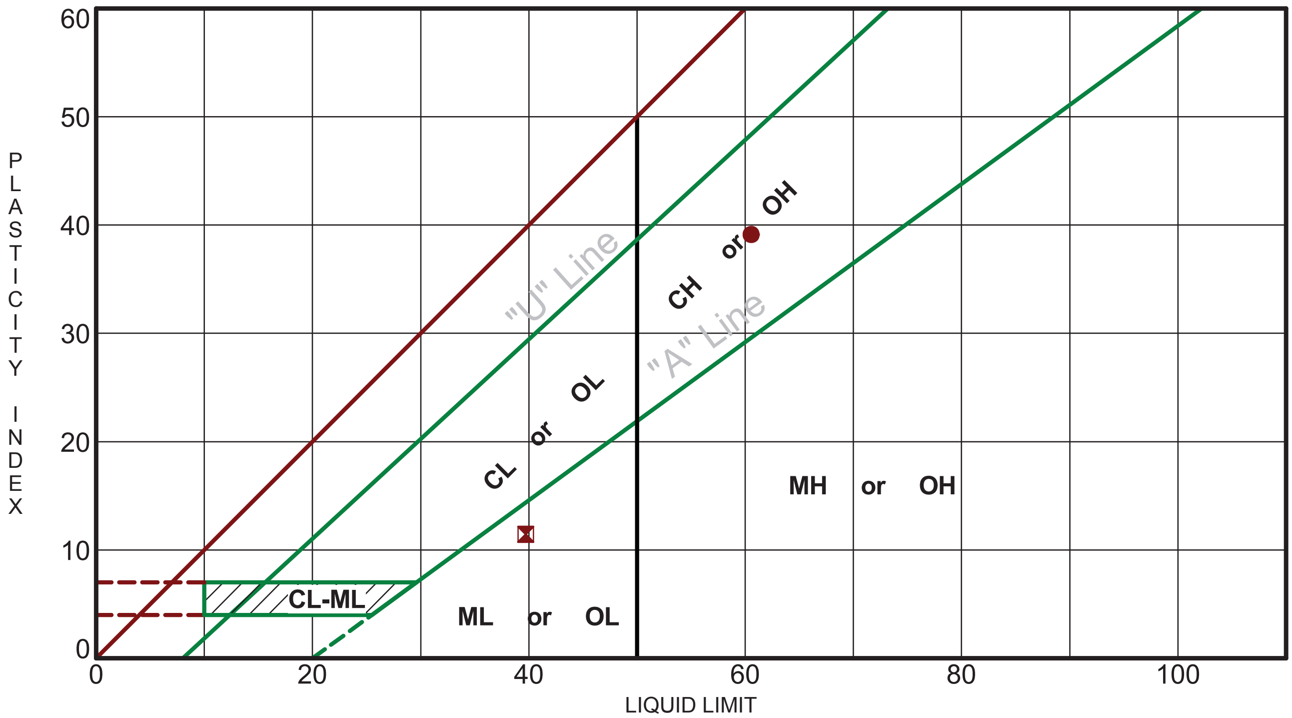
caved in at 11 ft.

Terracon
20401 Seneca Meadows Pkwy
Germantown, MD

Boring Started: 04-17-2019	Boring Completed: 04-17-2019
Drill Rig: CME 45B	Driller: Recon Drilling
Project No.: EY195020	

ATTERBERG LIMITS RESULTS

ASTM D4318

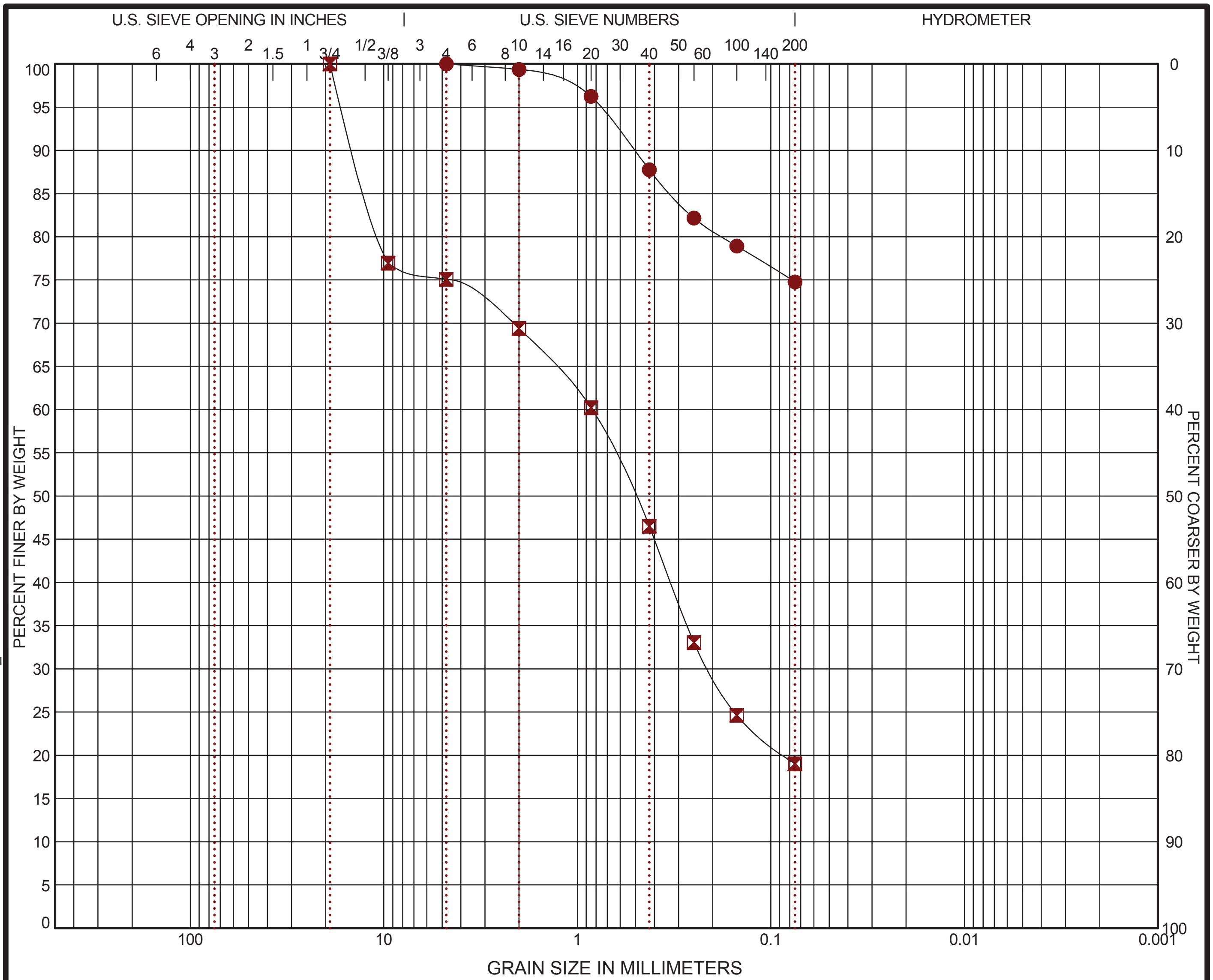


	Boring ID	Depth	LL	PL	PI	Fines	USCS	Description
●	**B-1A**	**13.5 - 15.5**	**61**	**21**	**40**	**75**	**CH**	**FAT CLAY with SAND**
⊠	**B-2**	**6 - 8**	**40**	**28**	**12**	**19**	**SM**	**SILTY SAND with GRAVEL**

PROJECT: Pedestrian Bridge for Green Circle Jubal Early Drive Trail Extension	Terracon 20401 Seneca Meadows Pkwy Germantown, MD	PROJECT NUMBER: EY195020
SITE: Harvest Drive and W. Jubal Early Drive Winchester, VA		CLIENT: A Morton Thomas & Assoc Inc Richmond, VA

GRAIN SIZE DISTRIBUTION

ASTM D422 / ASTM C136



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	BORING ID	DEPTH	% COBBLES	% GRAVEL	% SAND	% SILT	% FINES	% CLAY	USCS
●	**B-1A**	**13.5 - 15.5**	**0.0**	**0.0**	**25.2**		**74.8**		**CH**
⊠	**B-2**	**6 - 8**	**0.0**	**24.9**	**56.1**		**19.0**		**SM**

	GRAIN SIZE	
	●	⊠
D_{60}		**0.84**
D_{30}		**0.208**
D_{10}		
	COEFFICIENTS	
C_C		
C_U		

●		⊠			
Sieve	% Finer	Sieve	% Finer	Sieve	% Finer
#4	**100.0**	**3/4"**	**100.0**		
#10	**99.36**	**3/8"**	**76.97**		
#20	**96.24**	**#4**	**75.08**		
#40	**87.75**	**#10**	**69.39**		
#60	**82.16**	**#20**	**60.24**		
#100	**78.92**	**#40**	**46.52**		
#200	**74.78**	**#60**	**33.06**		
		#100	**24.62**		
		#200	**19.01**		

SOIL DESCRIPTION

● FAT CLAY with SAND (CH)

⊠ SILTY SAND with GRAVEL (SM)

REMARKS

●

⊠

PROJECT: Pedestrian Bridge for Green Circle
Jubal Early Drive Trail Extension

SITE: Harvest Drive and W. Jubal Early Drive
Winchester, VA

Terracon
20401 Seneca Meadows Pkwy
Germantown, MD

PROJECT NUMBER: EY195020

CLIENT: A Morton Thomas & Assoc Inc
Richmond, VA

SUPPORTING INFORMATION

General Notes
Unified Soil Classification System

GENERAL NOTES

DESCRIPTION OF SYMBOLS AND ABBREVIATIONS

SAMPLING		WATER LEVEL		FIELD TESTS	
	Standard Penetration Test		Water Initially Encountered	N	Standard Penetration Test Resistance (Blows/Ft.)
			Water Level After a Specified Period of Time	(HP)	Hand Penetrometer
			Water Level After a Specified Period of Time	(T)	Torvane
			Water levels indicated on the soil boring logs are the levels measured in the borehole at the times indicated. Groundwater level variations will occur over time. In low permeability soils, accurate determination of groundwater levels is not possible with short term water level observations.	(DCP)	Dynamic Cone Penetrometer
				(PID)	Photo-Ionization Detector
				(OVA)	Organic Vapor Analyzer

DESCRIPTIVE SOIL CLASSIFICATION

Soil classification is based on the Unified Soil Classification System. Coarse Grained Soils have more than 50% of their dry weight retained on a #200 sieve; their principal descriptors are: boulders, cobbles, gravel or sand. Fine Grained Soils have less than 50% of their dry weight retained on a #200 sieve; they are principally described as clays if they are plastic, and silts if they are slightly plastic or non-plastic. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size. In addition to gradation, coarse-grained soils are defined on the basis of their in-place relative density and fine-grained soils on the basis of their consistency.

LOCATION AND ELEVATION NOTES

Unless otherwise noted, Latitude and Longitude are approximately determined using a hand-held GPS device. The accuracy of such devices is variable. Surface elevation data annotated with +/- indicates that no actual topographical survey was conducted to confirm the surface elevation. Instead, the surface elevation was approximately determined from topographic maps of the area.

STRENGTH TERMS

RELATIVE DENSITY OF COARSE-GRAINED SOILS (More than 50% retained on No. 200 sieve.) Density determined by Standard Penetration Resistance		CONSISTENCY OF FINE-GRAINED SOILS (50% or more passing the No. 200 sieve.) Consistency determined by laboratory shear strength testing, field visual-manual procedures or standard penetration resistance		
Descriptive Term (Density)	**Standard Penetration or N-Value Blows/Ft.**	**Descriptive Term (Consistency)**	**Unconfined Compressive Strength Qu, (tsf)**	**Standard Penetration or N-Value Blows/Ft.**
Very Loose	0 - 3	Very Soft	less than 0.25	0 - 1
Loose	4 - 9	Soft	0.25 to 0.50	2 - 4
Medium Dense	10 - 29	Medium Stiff	0.50 to 1.00	4 - 8
Dense	30 - 50	Stiff	1.00 to 2.00	8 - 15
Very Dense	> 50	Very Stiff	2.00 to 4.00	15 - 30
		Hard	> 4.00	> 30

RELATIVE PROPORTIONS OF SAND AND GRAVEL

Descriptive Term(s) of other constituents	Percent of Dry Weight
Trace	< 15
With	15 - 29
Modifier	> 30

GRAIN SIZE TERMINOLOGY

Major Component of Sample	Particle Size
Boulders	Over 12 in. (300 mm)
Cobbles	12 in. to 3 in. (300mm to 75mm)
Gravel	3 in. to #4 sieve (75mm to 4.75 mm)
Sand	#4 to #200 sieve (4.75mm to 0.075mm
Silt or Clay	Passing #200 sieve (0.075mm)

RELATIVE PROPORTIONS OF FINES

Descriptive Term(s) of other constituents	Percent of Dry Weight
Trace	< 5
With	5 - 12
Modifier	> 12

PLASTICITY DESCRIPTION

Term	Plasticity Index
Non-plastic	0
Low	1 - 10
Medium	11 - 30
High	> 30

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UNIFIED SOIL CLASSIFICATION SYSTEM

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests [A]					Soil Classification: Group Symbol	Soil Classification: Group Name [B]
Coarse-Grained Soils: More than 50% retained on No. 200 sieve	**Gravels:** More than 50% of coarse fraction retained on No. 4 sieve	**Clean Gravels:** Less than 5% fines [C]	$Cu \geq 4$ and $1 \leq Cc \leq 3$ [E]		GW	Well-graded gravel [F]
			$Cu < 4$ and/or [$Cc<1$ or $Cc>3.0$] [E]		GP	Poorly graded gravel [F]
		Gravels with Fines: More than 12% fines [C]	Fines classify as ML or MH		GM	Silty gravel [F, G, H]
			Fines classify as CL or CH		GC	Clayey gravel [F, G, H]
	Sands: 50% or more of coarse fraction passes No. 4 sieve	**Clean Sands:** Less than 5% fines [D]	$Cu \geq 6$ and $1 \leq Cc \leq 3$ [E]		SW	Well-graded sand [I]
			$Cu < 6$ and/or [$Cc<1$ or $Cc>3.0$] [E]		SP	Poorly graded sand [I]
		Sands with Fines: More than 12% fines [D]	Fines classify as ML or MH		SM	Silty sand [G, H, I]
			Fines classify as CL or CH		SC	Clayey sand [G, H, I]
Fine-Grained Soils: 50% or more passes the No. 200 sieve	**Silts and Clays:** Liquid limit less than 50	**Inorganic:**	$PI > 7$ and plots on or above "A" line [J]		CL	Lean clay [K, L, M]
			$PI < 4$ or plots below "A" line [J]		ML	Silt [K, L, M]
		Organic:	Liquid limit - oven dried	< 0.75	OL	Organic clay [K, L, M, N]
			Liquid limit - not dried			Organic silt [K, L, M, O]
	Silts and Clays: Liquid limit 50 or more	**Inorganic:**	PI plots on or above "A" line		CH	Fat clay [K, L, M]
			PI plots below "A" line		MH	Elastic Silt [K, L, M]
		Organic:	Liquid limit - oven dried	< 0.75	OH	Organic clay [K, L, M, P]
			Liquid limit - not dried			Organic silt [K, L, M, Q]
Highly organic soils:	Primarily organic matter, dark in color, and organic odor				PT	Peat

[A] Based on the material passing the 3-inch (75-mm) sieve.

[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.

[C] Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.

[D] Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SM poorly graded sand with silt, SP-SC poorly graded sand with clay.

[E] $Cu = D_{60}/D_{10}$ $\quad Cc = \dfrac{(D_{30})^2}{D_{10} \times D_{60}}$

[F] If soil contains $\geq$ 15% sand, add "with sand" to group name.

[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.

[H] If fines are organic, add "with organic fines" to group name.

[I] If soil contains $\geq$ 15% gravel, add "with gravel" to group name.

[J] If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.

[K] If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.

[L] If soil contains $\geq$ 30% plus No. 200 predominantly sand, add "sandy" to group name.

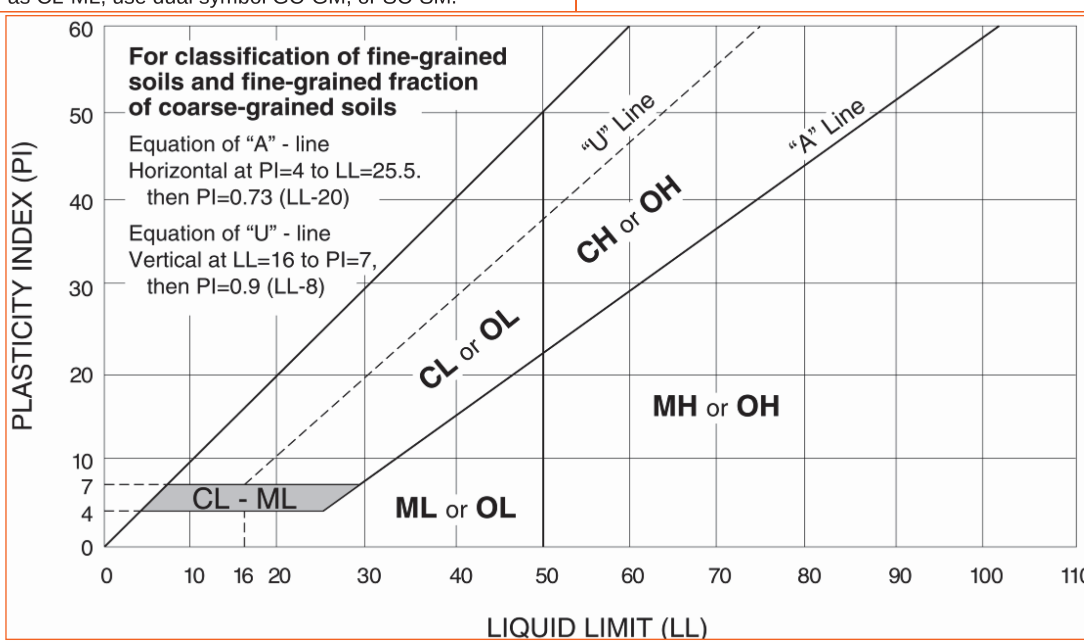
[M] If soil contains $\geq$ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.

[N] $PI \geq 4$ and plots on or above "A" line.

[O] $PI < 4$ or plots below "A" line.

[P] PI plots on or above "A" line.

[Q] PI plots below "A" line.



DESCRIPTION OF ROCK PROPERTIES

Terracon GeoReport

WEATHERING	
Term	**Description**
Unweathered	No visible sign of rock material weathering, perhaps slight discoloration on major discontinuity surfaces.
Slightly weathered	Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discolored by weathering and may be somewhat weaker externally than in its fresh condition.
Moderately weathered	Less than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a continuous framework or as corestones.
Highly weathered	More than half of the rock material is decomposed and/or disintegrated to a soil. Fresh or discolored rock is present either as a discontinuous framework or as corestones.
Completely weathered	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact.
Residual soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported.

STRENGTH OR HARDNESS		
Description	**Field Identification**	**Uniaxial Compressive Strength, psi (MPa)**
Extremely weak	Indented by thumbnail	40-150 (0.3-1)
Very weak	Crumbles under firm blows with point of geological hammer, can be peeled by a pocket knife	150-700 (1-5)
Weak rock	Can be peeled by a pocket knife with difficulty, shallow indentations made by firm blow with point of geological hammer	700-4,000 (5-30)
Medium strong	Cannot be scraped or peeled with a pocket knife, specimen can be fractured with single firm blow of geological hammer	4,000-7,000 (30-50)
Strong rock	Specimen requires more than one blow of geological hammer to fracture it	7,000-15,000 (50-100)
Very strong	Specimen requires many blows of geological hammer to fracture it	15,000-36,000 (100-250)
Extremely strong	Specimen can only be chipped with geological hammer	>36,000 (>250)

DISCONTINUITY DESCRIPTION			
Fracture Spacing (Joints, Faults, Other Fractures)		**Bedding Spacing (May Include Foliation or Banding)**	
Description	**Spacing**	**Description**	**Spacing**
Extremely close	< ¾ in (<19 mm)	**Laminated**	< ½ in (<12 mm)
Very close	¾ in – 2-1/2 in (19 - 60 mm)	**Very thin**	½ in – 2 in (12 – 50 mm)
Close	2-1/2 in – 8 in (60 – 200 mm)	**Thin**	2 in – 1 ft. (50 – 300 mm)
Moderate	8 in – 2 ft. (200 – 600 mm)	**Medium**	1 ft. – 3 ft. (300 – 900 mm)
Wide	2 ft. – 6 ft. (600 mm – 2.0 m)	**Thick**	3 ft. – 10 ft. (900 mm – 3 m)
Very Wide	6 ft. – 20 ft. (2.0 – 6 m)	**Massive**	> 10 ft. (3 m)

Discontinuity Orientation (Angle): Measure the angle of discontinuity relative to a plane perpendicular to the longitudinal axis of the core. (For most cases, the core axis is vertical; therefore, the plane perpendicular to the core axis is horizontal.) For example, a horizontal bedding plane would have a 0-degree angle.

ROCK QUALITY DESIGNATION (RQD) [1]	
Description	**RQD Value (%)**
Very Poor	0 - 25
Poor	25 – 50
Fair	50 – 75
Good	75 – 90
Excellent	90 - 100

1. The combined length of all sound and intact core segments equal to or greater than 4 inches in length, expressed as a percentage of the total core run length.

Reference: U.S. Department of Transportation, Federal Highway Administration, Publication No FHWA-NHI-10-034, December 2009
Technical Manual for Design and Construction of Road Tunnels – Civil Elements

DESCRIPTION OF ROCK PROPERTIES

WEATHERING	
Fresh	Rock fresh, crystals bright, few joints may show slight staining. Rock rings under hammer if crystalline.
Very slight	Rock generally fresh, joints stained, some joints may show thin clay coatings, crystals in broken face show bright. Rock rings under hammer if crystalline.
Slight	Rock generally fresh, joints stained, and discoloration extends into rock up to 1 in. Joints may contain clay. In granitoid rocks some occasional feldspar crystals are dull and discolored. Crystalline rocks ring under hammer.
Moderate	Significant portions of rock show discoloration and weathering effects. In granitoid rocks, most feldspars are dull and discolored; some show clayey. Rock has dull sound under hammer and shows significant loss of strength as compared with fresh rock.
Moderately severe	All rock except quartz discolored or stained. In granitoid rocks, all feldspars dull and discolored and majority show kaolinization. Rock shows severe loss of strength and can be excavated with geologist's pick.
Severe	All rock except quartz discolored or stained. Rock "fabric" clear and evident, but reduced in strength to strong soil. In granitoid rocks, all feldspars kaolinized to some extent. Some fragments of strong rock usually left.
Very severe	All rock except quartz discolored or stained. Rock "fabric" discernible, but mass effectively reduced to "soil" with only fragments of strong rock remaining.
Complete	Rock reduced to "soil". Rock "fabric" no discernible or discernible only in small, scattered locations. Quartz may be present as dikes or stringers.

HARDNESS (for engineering description of rock – not to be confused with Moh's scale for minerals)	
Very hard	Cannot be scratched with knife or sharp pick. Breaking of hand specimens requires several hard blows of geologist's pick.
Hard	Can be scratched with knife or pick only with difficulty. Hard blow of hammer required to detach hand specimen.
Moderately hard	Can be scratched with knife or pick. Gouges or grooves to ¼ in. deep can be excavated by hard blow of point of a geologist's pick. Hand specimens can be detached by moderate blow.
Medium	Can be grooved or gouged 1/16 in. deep by firm pressure on knife or pick point. Can be excavated in small chips to pieces about 1-in. maximum size by hard blows of the point of a geologist's pick.
Soft	Can be gouged or grooved readily with knife or pick point. Can be excavated in chips to pieces several inches in size by moderate blows of a pick point. Small thin pieces can be broken by finger pressure.
Very soft	Can be carved with knife. Can be excavated readily with point of pick. Pieces 1-in. or more in thickness can be broken with finger pressure. Can be scratched readily by fingernail.

Joint, Bedding, and Foliation Spacing in Rock [1]

Spacing	Joints	Bedding/Foliation
Less than 2 in.	Very close	Very thin
2 in. – 1 ft.	Close	Thin
1 ft. – 3 ft.	Moderately close	Medium
3 ft. – 10 ft.	Wide	Thick
More than 10 ft.	Very wide	Very thick

1. Spacing refers to the distance normal to the planes, of the described feature, which are parallel to each other or nearly so.

Rock Quality Designator (RQD) [1]

RQD, as a percentage	Diagnostic description
Exceeding 90	Excellent
90 – 75	Good
75 – 50	Fair
50 – 25	Poor
Less than 25	Very poor

1. RQD (given as a percentage) = length of core in pieces 4 inches and longer / length of run

Joint Openness Descriptors

Openness	Descriptor
No Visible Separation	Tight
Less than 1/32 in.	Slightly Open
1/32 to 1/8 in.	Moderately Open
1/8 to 3/8 in.	Open
3/8 in. to 0.1 ft.	Moderately Wide
Greater than 0.1 ft.	Wide

References: American Society of Civil Engineers. Manuals and Reports on Engineering Practice - No. 56. Subsurface Investigation for Design and Construction of Foundations of Buildings. New York: American Society of Civil Engineers, 1976. U.S. Department of the Interior, Bureau of Reclamation, Engineering Geology Field Manual.