



ATLANTIC TESTING LABORATORIES

WBE certified company

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March 10, 2023

McFarland Johnson
60 Railroad Place, Suite 402
Saratoga Springs, New York 12866

Telephone: 518-580-9380 Ext. 364
Facsimile: 518-580-9383

Attn: Laura Sanda, PE

Email: lsanda@mjinc.com

Re: Subsurface Investigation and Geotechnical Evaluation
Bridge NY Project
Proposed Bridge Replacement (BIN 2227620)
Loft Road over Schenevus Creek
Maryland, Otsego County, New York
ATL No. CD10372E-01-10-22 Rev. 1

Ladies and Gentlemen:

Enclosed is one digital copy of the referenced report. ATL appreciates the opportunity to provide geotechnical services for your project.

Please note that upon completion of the subsurface investigation the boreholes were backfilled with on-site soils. It is important that the backfilled borings be monitored for settlement or subsidence. This will be the responsibility of the McFarland Johnson and/or their Client. ATL assumes no liability for borehole settlement.

The soil samples obtained during this investigation will be retained for a period of six months and subsequently discarded, unless otherwise instructed.

Please contact our office should you have any questions or comments on this information, or if we may be of further service. We look forward to our continued association to obtain a successful completion of this project.

Sincerely,
ATLANTIC TESTING LABORATORIES, Limited

Brian T. Barnes, PE
Senior Engineer

BTB/rs

Enclosures

**SUBSURFACE INVESTIGATION
AND
GEOTECHNICAL EVALUATION**

**BRIDGE NY PROJECT
PROPOSED BRIDGE REPLACEMENT (BIN 2227620)
LOFT ROAD OVER SCHENEVUS CREEK
MARYLAND, OTSEGO COUNTY, NEW YORK**

McFARLAND JOHNSON

**PREPARED FOR: McFarland Johnson
60 Railroad Place, Suite 402
Saratoga Springs, New York 12866**

**PREPARED BY: Atlantic Testing Laboratories, Limited
6431 U.S. Highway 11
P. O. Box 29
Canton, NY 13617**

ATL Report No. CD10372E-01-10-22 Rev. 1

March 10, 2023

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SUBSURFACE INVESTIGATION AND GEOTECHNICAL EVALUATION

BRIDGE NY PROJECT PROPOSED BRIDGE REPLACEMENT (BIN 2227620) LOFT ROAD OVER SCHENEVUS CREEK MARYLAND, OTSEGO COUNTY, NEW YORK

McFARLAND JOHNSON

1.0 INTRODUCTION

At the request of Laura Sanda, PE, representing McFarland Johnson and in accordance with our proposal (ATL File No. CD998-099-01-22, dated January 20, 2022), Atlantic Testing Laboratories, Limited (ATL) performed a subsurface investigation and geotechnical evaluation for the referenced project. The subsurface investigation program, consisting of one (1) soil boring at each abutment location, was conducted on September 22 and 23, 2022.

The purpose of the investigation was to ascertain the general subsurface soil, bedrock, and groundwater conditions at the site, to evaluate the engineering significance of these findings, and to provide recommendations related to the proposed bridge design and construction.

The proposed project is located along Loft Road over Schenevus Creek in the Town of Maryland, Otsego County, New York. The approximate project coordinates are N 42°32'07" latitude and W 74°53'04" longitude. A **Site Location Plan** is included in **Appendix A**. All dimensions and elevations referenced in this report are in units of feet, unless otherwise noted.

2.0 PROJECT DESCRIPTION

The proposed project consists of a complete replacement and horizontal realignment of the Loft Road bridge over Schenevus Creek, located in the Town of Maryland, Otsego County, New York. It is our understanding that the proposed project will consist of the construction of a new bridge structure supported on driven H-piles or drilled micropiles. No other foundation details were provided at the time of report preparation.

3.0 SITE SURFACE CONDITIONS & GEOLOGY

The Loft Road bridge is located approximately 2300 feet north of US-88 and approximately 300 feet south of the intersection of County Route-7 and Iron Kettle Road. The project is generally surrounded by a combination of agricultural fields and pastures, rural residential properties, and interspersed with moderate to heavily wooded areas. The Loft Road Bridge is currently an asphalt paved, single-span crossing. The bridge surface elevation is at approximately 1197 feet with a stream bed elevation around 1180 feet. A Norfolk Southern railroad line is located to the north in close proximity to the bridge. The railroad line is carried over the roadway via a stone arch bridge.

The site is located in the north-western portion of the Catskill Mountain Range Foothills of New York State, along the border of the Allegheny Plateau Region. Based on the Surficial Geologic Map of New York State, Hudson-Mohawk Sheet, 1987, the project area is generally mantled with unconsolidated stream deposited recent alluvium soils comprised primarily of permeable fine sand and gravel, confined to flood plains within valleys. Based on the New York State Geologic Map, Hudson-Mohawk Sheet, 1970, the project area is generally underlain by Middle Devonian age shale, siltstone and sandstone from the Moscow Formation.

4.0 SUBSURFACE INVESTIGATION & SAMPLING METHODOLOGY

Two (2) soil boring locations were selected by representatives McFarland Johnson and staked in the field by ATL. The approximate soil boring locations are depicted on the provided **Boring Location Plan** included in **Appendix B**.

The soil borings were advanced through the existing roadway surface behind the existing bridge abutments utilizing a combination of 3-inch (NW) inside-diameter, flush-joint casing and a 2-15/16 tri-cone roller bit using water as the drilling fluid. Soil sampling and standard penetration testing was performed utilizing a 2-inch outside diameter split spoon sampler and automatic drop hammer in accordance with ASTM D 1586. Soil sampling was generally performed continuously to a depth of 22 feet, and at 5-foot intervals thereafter to weathered bedrock at depths of 30.5 and 57 feet below the existing roadway surface in soil boring B-1 and B-2, respectively. The roller bit was subsequently advanced to more competent bedrock at depths of 34 and 58 feet, respectively, and the bedrock was cored using an NX-size, double-tube core barrel to boring termination at depths of 44 and 68 feet, respectively.

The recovered soil samples were visually classified in the laboratory by an engineering technician in general accordance with the Burmister Soil Classification System. The split spoon sampler does not recover material larger than 1³/₈-inch in nominal dimension; therefore, the soil classifications may not be representative of the entire soil matrix. The visual classifications and the standard penetration test results are presented on the **Subsurface Investigation Logs** included in **Appendix C**.

The boreholes were backfilled with on-site soils upon completion. It is important that the backfilled borings be monitored for settlement or subsidence. This will be the responsibility of McFarland Johnson and/or their Client. ATL assumes no liability for loss or damage resulting from borehole settlement.

5.0 SITE SUBSURFACE CONDITIONS

The following description of subsurface conditions is based on the soil, bedrock, and groundwater conditions encountered during this subsurface investigation. Actual subsurface conditions may vary across the site in both the horizontal and vertical dimensions. Detailed subsurface descriptions are provided on the Subsurface Investigation Logs.

5.1 Soil Borings

5.1.1 North Abutment (B-1)

Soil boring B-1 generally encountered loose (SPT N-values 4 to 10) to medium compact (SPT N-values 10 to 30) silty, sand and gravel that extended to a depth of 6 feet below the surface. The loose to medium compact sand and gravel was generally underlain by medium compact to compact (SPT N-values 30 to 50) sandy gravel with trace silt and clay that extended to weathered bedrock a depth of approximately 30.5 feet. A roller bit was advanced to more competent Siltstone bedrock at a depth of 34 feet and the bedrock was cored utilizing an NX-size, double-tube core barrel to termination at a depth of 44 feet. The rock core description is provided below:

Rock Core Descriptions

Boring No./ Location	Run No.	Depth (ft.)	Recovery (%)	RQD (%)
B-1	1	34.0 – 39.0	77	67
B-1	2	39.0 – 44.0	37	27

Based on the rock core recovery and RQD value, the bedrock is considered fair to poor quality Siltstone.

5.1.2 South Abutment (B-2)

Soil boring B-2 generally encountered medium compact silty, sand and gravel that extended to a depth of 8 feet below the surface. The medium compact sand and gravel was underlain by alternating layers of medium compact to very compact (SPT N-values >50) sand, gravel, and non-plastic silt that extended to a depth of approximately 32 feet, underlain by medium compact silty, sand and gravel that extended to weathered bedrock at a depth of 57 feet. A roller bit was advanced to more competent Siltstone bedrock at a depth of 58 feet and the bedrock was cored utilizing an NX-size, double-tube core barrel to termination at a depth of 68 feet. The rock core description is provided below:

Rock Core Descriptions

Boring No./ Location	Run No.	Depth (ft.)	Recovery (%)	RQD (%)
B-2	1	58.0 – 63.0	97	82
B-2	2	63.0 – 68.0	97	98

Based on the rock core recovery and RQD value, the bedrock is considered good quality Siltstone.

Based on the driller's observations, cobbles were encountered throughout the depths of each soil boring.

5.2 Groundwater

Groundwater measurements were performed during the subsurface investigation within the cased and open boreholes. The soil samples were also classified for coloration and relative moisture conditions.

Freestanding water was recorded at depths ranging from 1.2 to 9.2 feet below the surface; however, the groundwater readings were affected by water utilized to advance the boreholes and are not considered a reliable indicator of the true groundwater depths. The borings were backfilled with onsite soils immediately upon completion; therefore, the water levels did not have sufficient time to stabilize in the open boreholes.

Fluctuations in water levels may occur due to seasonal and climatic variations, changes in surface runoff patterns, construction activity, water surface variations in the Schenevus Creek, and subsequent development of the site along with other interrelated factors.

6.0 LABORATORY ANALYSES

Select soil samples were submitted to ATL's geotechnical laboratory for the following physical analyses:

- ◆ Seventeen (17) Water Content Determination of Soil (ASTM D 2216)
- ◆ Four (4) Particle Size Analysis without Hydrometer (ASTM D 422)
- ◆ One (1) One (1) rock core specimen was selected for Unconfined Compressive Strength Determination (ASTM D7012, Method C), summarized in the table below:

Unconfined Compressive Strength of Intact Rock Core Specimens

Boring No.	Run No.	Sample Depth (ft)	RQD (%)	Compressive Strength (psi)	Approximate Rock Unit Weight (pcf)
B-1	1	34-39	67	13,310	170

¹ The compressive strength test was performed on an intact portion recovered from the rock core samples and may not be representative of the in-situ rock mass. The presence of weathered zones or bedding planes will reduce the strength of the in-situ rock mass. The test load was applied along the core axis.

² The calculated unit weight was determined from the laboratory prepared compressive strength test specimen.

The laboratory test results are summarized on the subsurface investigation logs included in Appendix C and contained in **Appendix D, Laboratory Test Reports**.

7.0 GEOTECHNICAL ENGINEERING DISCUSSION

The Geotechnical Engineering Discussion is based on information provided by McFarland Johnson, our understanding of the project, and the subsurface conditions outlined in this report.

The following recommendations are presented as the minimum requirements for the design, planning, and construction of the bridge replacement foundations. The concepts and geotechnical engineering considerations presented should be considered in project design and construction. These concepts may require alterations to meet the specific design and economic considerations for this project.

Based on the compact to very compact sand and gravel soils encountered in the soil borings and the proximity of the adjacent stone arch railroad crossing, low displacement piles such as driven, steel H-piles or drilled micropiles are recommended to support the proposed bridge abutments. It is anticipated that driven steel H-piles will derive their capacity from end bearing on the underlying weathered or more competent siltstone bedrock encountered at a depth of 30.5 feet at boring location B-1 and a depth of 57 feet at boring location B-2. Based on the bedrock depths, it appears the bedrock either slopes or steps downward from the north (B-1) to the south (B-2). The presence of cobbles and very compact soils encountered in the soil borings could affect pile alignment and result in early refusal of driven piles.

It is anticipated drilled micropiles would derive their capacity through grout to bedrock bond resistance in the siltstone bedrock. Since bedrock is deeper at boring location B-2, it is possible the micropiles could be supported in the medium compact silty, sand and gravel, depending on the final abutment depths and foundation loads.

The advantage of micropiles over driven H-piles is micropiles typically induce less ground vibrations during construction and could be advanced through cobbles, boulders, and bedrock to achieve the design tip elevations.

7.1 Proposed Bridge Replacement

7.1.1 Site Work

Site work will require the removal of the existing asphalt pavement, surrounding topsoil and organic material, bridge abutments, wing walls and associated foundations within the footprint of the proposed bridge. Based on current site grades, it is our understanding approximately 8 to 10 feet of Granular Fill will be required to raise site grades at the new south abutment location. It is recommended the Granular Fill be placed and compacted prior to the driven pile or micropile installations to reduce the potential for downdrag loads on the deep foundation system. Since the subsurface soils are granular, it is anticipated that settlement of the new fill will occur as the fill is being placed, with negligible post-construction settlement.

The excavations for foundations may extend below the groundwater and Schenevus Creek water levels. Shoring or other acceptable excavation stabilizing methods and dewatering will be required to provide a dry, stable work area. The on-site granular soils will be susceptible to sidewall sloughing in open excavations, especially below the water levels. The contractor should develop an excavation stabilization and dewatering plan for review by the owner and engineer prior to beginning the excavations.

It is recommended that the bridge foundation subgrade be over excavated a minimum 6 inches and replaced with 6 inches of NYSDOT Number 2 or 3, crushed stone, compacted with a minimum 4 passes with a dual-drum, walk-behind vibratory roller; a Wacker DPU 6055 vibratory plate tamper, or equivalent to provide a stable working surface and dewatering media for foundation construction. Crushed stone layers exceeding 6 inches in compacted thickness should be completely wrapped in a non-woven geotextile fabric (Mirafi 160N or equivalent).

7.1.2 Driven Piles

Steel H-piles should be driven to practical refusal, defined as 5 blows per ¼-inch of pile penetration, on the underlying weathered or more competent siltstone bedrock that was encountered in the soil borings.

Steel piles driven to refusal on bedrock (typically defined as 10 to 20 blows/inch of pile penetration) could be designed for end bearing using an allowable axial stress of 25 % of the yield strength of the pile steel. It is estimated that a 12 X 53 H-pile section using ASTM A572 grade 50 steel could be designed for a service limit state axial pile capacity of 190 kips and a strength limit state axial pile capacity of 278 kips, using a resistance factor of 0.65. The piles should be driven to achieve a nominal axial pile resistance of 428 kips. Other pile sizes could be considered for the project depending on the final foundation loading requirements.

It is our understanding approximately 8 to 10 feet of Granular Fill will be required to raise site grades at the new south abutment location. Downdrag loads due to soil settlement around the piles should be negligible, provided the Granular Fill is placed and compacted prior to the pile installations as recommended in Section 7.1.1.

The contractor should complete wave equation analyses, using the selected hammer and pile section, to confirm the hammer can achieve the required ultimate pile capacity, without resulting in excessive driving stresses, and to establish preliminary driving criteria. The pile stresses during driving should not exceed 90 percent of the pile yield strength, as recommended by the American Association of State Highway and Transportation Officials (AASHTO).

At least 2% of the production piles, but no less than two piles should be subject to dynamic load testing in accordance with ASTM D4945 "Standard Test Method for High Strain Dynamic Testing of Piles" during pile driving to verify the nominal pile capacity and pile integrity. Results of the load tests should determine the required pile lengths and establish driving criteria for construction.

The piles should be embedded a sufficient depth to provide adequate lateral and uplift resistance. Pre-drilling may be required if practical refusal is encountered prior to achieving the design depths. The pile tips should be fitted with a drive shoe to limit toe damage during driving.

The estimated pile capacities are based on the subsurface conditions encountered during our subsurface investigation and theoretical static pile analysis. The actual subsurface conditions and pile capacities may vary during driving. The actual pile penetration depths and nominal pile capacities should be based on pile load tests performed during pile installation.

Individual pile spacing (center to center) should be greater than 3 times the pile's largest cross sectional dimension and not less than 36 inches. The pile spacing should be measured center to center at the cut off elevation.

7.1.3 Drilled Micropiles

Micro-piles will derive their capacity from the bond between the grout and underlying siltstone bedrock, or in the medium compact silty, sand and gravel stratum at the south abutment location, depending on the final abutment depths and foundation loads. The effective bond zone should extend into the bearing stratum a depth sufficient to provide the required axial, lateral, and uplift resistance.

For preliminary design purposes, a nominal grout-to-rock bond value of 20 kips/ft² (ksf) is recommended for gravity-grouted (Type A) micro-piles installed within competent siltstone bedrock at both abutment locations. At the south abutment location, a nominal grout-to-soil bond value of 4.5 ksf is recommended for pressure-grouted (Type B) micro-piles installed within the medium compact silty, sand and gravel. Grout should have a minimum 28-day compressive strength of 4000 psi.

Individual micro-pile spacing (center to center) should be greater than 3 times the piles largest cross sectional dimension and not less than 30 inches. The pile spacing should be measured center to center at the cut off elevation.

The final micro-pile design should be provided by a qualified specialty contractor. The means and methods of the micro-pile installations should be the responsibility of the contractor. The contractor should prepare a "Micro-pile Installation Plan" and submit it to the engineer and owner. The installation of the micro-piles should be continuously monitored by a Geotechnical Engineer familiar with the subsurface conditions present at the site and the micro-pile procedures utilized by the contractor. It is recommended that at least one static pile load test be performed for each micro-pile type utilized at the project site.

7.2 Frost Protection

Shallow foundations requiring frost protection should be founded a minimum of 4.5 feet below final exterior grade.

7.3 Seismic

Based on the field standard penetration test results, the Mapped Spectral Accelerations (S_S & S_1 , with 2% probability of exceedance in 50 years), Site Coefficients (F_a & F_v), and Design Spectral Response Acceleration (S_{DS} & S_{D1}) values, as determined from ASCE 7-16, are summarized in the following table:

Seismic Design Parameters

Design Parameter Description	Parameter
Site Longitude	42.53456°N
Site Latitude	74.88467°W
0.2 second Mapped Spectral Response Acceleration (S_S)	0.166g
1.0 second Mapped Spectral Response Acceleration (S_1)	0.054g
Site Class	D
Site Coefficient, F_a	1.6
Site Coefficient, F_v	2.4
Mapped Spectral Response Acceleration at Short Periods (S_{MS})	0.266g
Mapped Spectral Response Acceleration at 1-Second Period (S_{M1})	0.13g
Design Spectral Response Acceleration at Short Periods (S_{DS})	0.177g
Design Spectral Response Acceleration at 1-Second Period (S_{D1})	0.087g

7.4 General

Foundation excavations may encounter groundwater, cobbles, and sloughing granular soils. The in-situ granular soils will be susceptible to caving in open excavations, particularly below groundwater levels.

Granular Fill should be used to raise site grades beneath the new abutments and roadway. Select on-site, sand and gravel soils may be utilized as abutment backfill, provided all organics and oversize material (particles larger than 4 inches in diameter) are removed and the material can be properly moisture conditioned. The abutment excavations should be backfilled with Common Fill below the water level in the creek. Rip-rap and/or geotextiles should be placed on the backfill slopes to provide erosion protection.

All controlled fill and backfill should be placed and compacted in lifts not exceeding eight inches in loose thickness, at a moisture content of $\pm 2\%$ of the Optimum Moisture Content, and to densities in excess of 95%, as determined by ASTM D1557, or as directed by the Geotechnical Engineer.

Compaction should be performed with vibratory rollers unless there is concern for damage to adjacent structures or underground utilities.

Common Fill should be relatively impervious exhibiting a coefficient of permeability in the range of 1×10^{-3} to 1×10^{-5} cm/sec. The Common Fill should conform to the following gradation.

Sieve Size	*Percent Passing
3"	100
1/4"	35 - 70
#200	15 - 30

*Other material gradations may be acceptable for common fill.

Granular Fill should consist of a clean, screened, crushed, or bank-run gravel conforming to the following gradation:

Sieve Size	Percent Passing
4"	100
1/4"	35-65
#200	0-10

The empirical soil parameters presented in the following table may be used for the following backfill materials.

Table of Soil Properties

Soil Property	Common Fill	On-site Sand and Gravel	Granular Fill
Angle of Internal Friction (°)	28	30	32
Active Earth Coefficient (K_a)*	0.36	0.33	0.31
At Rest Earth Coefficient (K_o)*	0.53	0.50	0.47
Passive Earth Coefficient (K_p)*	2.77	3.00	3.25
Moist Unit Weight (pcf)	120-130	125-135	130-140

*The Rankine earth pressure coefficients (ultimate values) are for level backfill placed in a fully drained condition.

7.5 Dewatering

The contractor should anticipate that groundwater may be encountered in shallow foundation and utility excavations, especially during wetter periods of the year.

It will be the contractor's responsibility to maintain adequate water control at all times. Project specifications should clearly indicate that standing water, and/or saturated, unstable soil conditions will not be tolerated in areas to receive foundations or utilities. The project specifications should state that the contractor will not be reimbursed for extras related to the control of water.

All dewatering activities should comply with New York State Department of Environmental Conservation (NYSDEC) storm water discharge requirements and/or local regulations for construction.

7.6 Testing and Inspection

The final site grading and foundation plans and project specifications should be reviewed by ATL, as the Geotechnical Engineer of Record, to verify that there has not been a misinterpretation of this report and/or ATL's understanding of the project.

We recommend that ATL, as the Geotechnical Engineer of Record, be retained to perform applicable monitoring and inspection activities during site earthwork and foundation installations. An ATL geotechnical representative familiar with the findings and recommendations of this report will be able to assess the subsurface conditions encountered during construction, provide necessary remedial recommendations, and verify that adequate bearing capacities and proper foundation installation requirements are achieved.

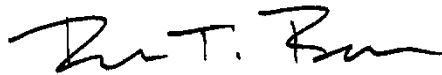
All foundation construction and backfilling should be monitored and tested by an Independent Testing Agency, conforming to ASTM E-329, "Standard Specification for Agencies Engaged in Construction Inspection, Testing, or Special Inspection." ATL conforms to ASTM E-329 and can be retained to perform required construction phase monitoring and testing services, including applicable Special Inspections and Structural Tests in accordance with the Building Code of New York State.

8.0 LIMITATIONS

The subsurface investigation logs and this report in its entirety should be provided to the contractors for information and interpretation. The subsurface investigation logs may not be representative of the entire site subsurface condition, but only what was encountered at the individual test location at the time of the investigation. The subsurface soil, bedrock, and groundwater conditions may be different from those described on the subsurface investigation logs and summarized in this report.

This report was prepared to present the findings of our subsurface investigation and engineering evaluation, and to outline concepts to be utilized in foundation design and construction. These concepts may require alterations to meet the specific design and economic considerations for this project.

Prepared by:

A handwritten signature in black ink, appearing to read "Brian T. Barnes". The signature is fluid and cursive, with the first name "Brian" and last name "Barnes" clearly distinguishable.

Brian T. Barnes, PE
Senior Engineer

RS/BTB/rs

APPENDIX A
SITE LOCATION PLAN



Site Location Map

Drawn by:
BTB

Scale:
Not to scale

Project No.:
CD10372

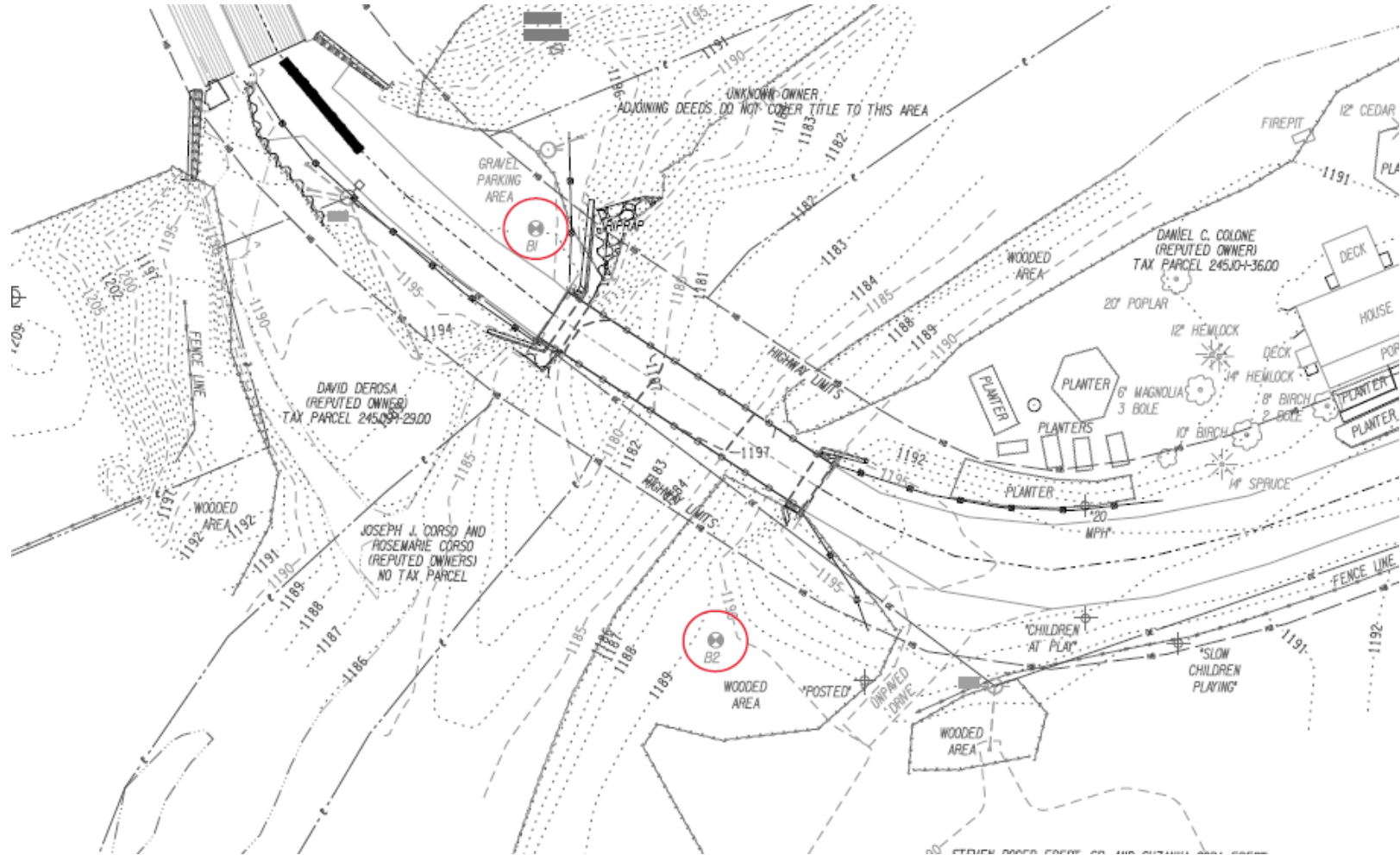
Date:
December 9, 2022

**Proposed Loft Road Bridge Replacement
Maryland, Otsego County, New York**

ATLANTIC TESTING LABORATORIES, Limited

Albany, NY	Binghamton, NY	Canton, NY	Elmira, NY	Plattsburgh, NY
Poughkeepsie, NY	Syracuse, NY	Rochester, NY	Utica, NY	Watertown, NY

APPENDIX B
BORING LOCATION PLAN



North

Boring Location Plan

Drawn by: BTB

Scale:
Not to scale

Project No.:
CD10372

Date:
December 9, 2022

**Proposed Loft Road Bridge
Replacement
Maryland, Otsego County, New York**



ATLANTIC TESTING LABORATORIES, Limited

- | | | | | |
|------------------|----------------|---------------|------------|-----------------|
| Albany, NY | Binghamton, NY | Canton, NY | Elmira, NY | Plattsburgh, NY |
| Poughkeepsie, NY | Syracuse, NY | Rochester, NY | Utica, NY | Watertown, NY |

APPENDIX C
SUBSURFACE INVESTIGATION LOGS

ATLANTIC TESTING LABORATORIES, Limited

Subsurface Investigation

Client: McFarland Johnson
 Project: Subsurface Investigation
BridgeNY Project - Loft Road over Schenevus Creek
Town of Maryland, New York

Report No.: CD10372E-01-10-22
 Boring Location: See Boring Location Plan

Boring No.: B-1 Sheet 1 of 2

Start Date: 9/22/2022 Finish Date: 9/23/2022

Coordinates _____ Sampler Hammer _____
 Latitude _____ Weight: 140 lbs.
 Longitude _____ Fall: 30 in.
 Hammer Type: Automatic

Groundwater Observations			
Date	Time	Depth	Casing
<u>9/22/2022</u>	<u>PM</u>	<u>*8.2'</u>	<u>30.0'</u>
<u>9/23/2022</u>	<u>AM</u>	<u>9.2'</u>	<u>30.0'</u>
<u>9/23/2022</u>	<u>AM</u>	<u>7.3'</u>	<u>CAVED</u>

Ground Elev.: _____ Boring Advance By: Borehole caved at 8.5 feet. *May be affected by
NW (3") Casing/2 15/16" Wet Rotary/NX Core water utilized to advance the borehole.

ATL-LOG1 LL CD10372 MCFARLAND JOHNSON - MARYLAND.GPJ ATL4-08.GDT 10/28/22

DEPTH	METHOD OF ADVANCE	SAMPLE NO.	DEPTH OF SAMPLE		SAMPLE TYPE	BLOWS ON SAMPLER PER 6" 2" O.D. SAMPLER	DEPTH OF CHANGE	CLASSIFICATION OF MATERIAL	Recovery (Inches)
			From	To					
1	C S N G	1	0.0	2.0	SS	12 8 8 7	6.0	Brown cmf SAND; and mf GRAVEL; little SILT (moist, non-plastic)	5
2		2	2.0	4.0	SS	6 4 4 2		Encountered COBBLE fragments from surface to 30.5 feet.	2
3		3	4.0	6.0	SS	2 3 4 7		Brown mf GRAVEL; some cmf SAND; trace SILT (moist, non-plastic) w = 7.3%	6
4	4	6.0	8.0	SS	16 12 14 16	Brown f GRAVEL; some cmf SAND; little SILT; trace CLAY (wet, very slightly plastic) w = 18.9%		14	
5	5	8.0	10.0	SS	8 13 12 12	Brown mf GRAVEL; some cmf SAND; little SILT; trace CLAY (wet, very slightly plastic) w = 12.1%		10	
6	6	10.0	12.0	SS	12 14 24 26	Similar Soil (wet, very slightly plastic) w = 14.3%		9	
7	7	12.0	14.0	SS	18 16 21 26	Brown cmf GRAVEL; some cmf SAND; trace SILT (wet, non-plastic) w = 12.1%		11	
8	8	14.0	16.0	SS	18 63 35 52	Similar Soil (wet, very slightly plastic) w = 9.5%		15	
9	9	16.0	18.0	SS	24 16 17 30	Similar Soil (wet, very slightly plastic) w = 12.0%		20	
10	10	18.0	18.8	SS	22 50/4"	Similar Soil (wet, very slightly plastic) COBBLE Fragments		6	
11	11	20.0	22.0	SS	18 20 35 34	Brown mf GRAVEL; some cmf SAND; trace SILT; trace CLAY (wet, very slightly plastic)		16	
12	12	24.0	26.0	SS	17 21 17 17	Similar Soil (wet, very slightly plastic)		12	

SS Split Spoon Sample
 NX Rock Core
 SH Undisturbed Sample (Shelby Tube)
 Estimated Groundwater

Drillers: Mark Childs; Greydenn Griffin
 Inspector: _____

ATLANTIC TESTING LABORATORIES, Limited

Subsurface Investigation

Boring No.: B-1

Report No.: CD10372E-01-10-22

Sheet 2 of 2

DEPTH	METHOD OF ADVANCE	SAMPLE NO.	DEPTH OF SAMPLE		SAMPLE TYPE	BLOWS ON SAMPLER PER 6" 2" O.D. SAMPLER	DEPTH OF CHANGE	CLASSIFICATION OF MATERIAL	RECOVERY (inches)
			From	To					
26									
27									
28									
29		13	29.0	30.9	SS	12 14 15 50/3"		Brown cmf SAND; some mf GRAVEL; little SILT; trace CLAY (wet, very slightly plastic) Possible WEATHERED ROCK Fragments	11
30	WET ROTTARY CORE						30.5		
31								Encountered possible WEATHERED ROCK at 30.5 feet.	
32								Advanced casing to 30.0 feet. Advanced 2 15/16" tri-cone roller bit wet rotary open hole to 34.0 feet and began coring.	
33								34.0	
34			34.0	39.0	NX	RUN 1		Grey SILTSTONE 46" or 77% Recovery 7 Pieces (46") - 0% Chips and Fragments 4 Pieces longer than 4" (40") - RQD = 67%	46
35									
36									
37									
38	(WET)								
39			39.0	44.0	NX	RUN 2	39.0	Grey SILTSTONE 22" or 37% Recovery 4 Pieces (21") - 5% Chips and Fragments 2 Pieces longer than 4" (16") - RQD = 27%	22
40									
41									
42									
43									
44							44.0		
45								Boring terminated at 44.0 feet.	
46								Notes:	
47								1. Borehole backfilled with on-site soils.	
48									
49									
50									
51									
52									
53									
54									
55									
56									
57									
58									
59									
60									
61									
62									

ATL-LOG1 LL CD10372 MCFARLAND JOHNSON - MARYLAND.GPJ ATL4-08.GDT 10/28/22

ATLANTIC TESTING LABORATORIES, Limited

Subsurface Investigation

Client: McFarland Johnson
 Project: Subsurface Investigation
BridgeNY Project - Loft Road over Schenevus Creek
Town of Maryland, New York

Report No.: CD10372E-01-10-22
 Boring Location: See Boring Location Plan

Start Date: 9/23/2022 Finish Date: 9/23/2022

Boring No.: B-2 Sheet 1 of 3

Groundwater Observations			
Date	Time	Depth	Casing
<u>9/23/2022</u>	<u>PM</u>	<u>*1.2'</u>	<u>CAVED</u>

Coordinates
 Latitude _____
 Longitude _____

Sampler Hammer
 Weight: 140 lbs.
 Fall: 30 in.
 Hammer Type: Automatic

Ground Elev.: _____ Boring Advance By: Borehole caved at 5.5 feet. *May be affected by
NW (3") Casing/2 15/16" Wet Rotary/NX Core water utilized to advance the borehole.

DEPTH	METHOD OF ADVANCE	SAMPLE NO.	DEPTH OF SAMPLE		SAMPLE TYPE	BLOWS ON SAMPLER PER 6" 2" O.D. SAMPLER				DEPTH OF CHANGE	CLASSIFICATION OF MATERIAL	Recovery (Inches)
			From	To		3	5	4	3			
1	C S N G	1	0.0	2.0	SS	3	5	4	3	2.0	Brown cmf SAND; and mf GRAVEL; trace SILT; trace ORGANIC MATERIAL (grass, roots) (moist, non-plastic)	6
2		2	2.0	4.0	SS	4	5	8	12			
3		3	4.0	6.0	SS	14	8	10	10			
4										8.0	Encountered COBBLE fragments from 2.0 feet to 56.0 feet. Brown cmf- GRAVEL; little cmf SAND; trace SILT (wet, non-plastic) w = 8.8% Brown mf GRAVEL; trace cmf SAND; trace SILT (wet, non-plastic) w = 12.5%	4
5	4	6.0	8.0	SS	6	5	6	8				
6												
7												
8		5	8.0	10.0	SS	10	11	7	6	8.0	Brown mf GRAVEL; trace cmf SAND; trace SILT (wet, non-plastic) w = 10.2 %	2
9												
10		6	10.0	12.0	SS	6	9	12	6			
11												
12		7	12.0	14.0	SS	5	11	22	17		Brown cmf SAND; and mf GRAVEL; trace SILT (wet, non-plastic) w = 9.2%	22
13												
14		8	14.0	16.0	SS	14	17	24	21		Similar Soil (wet, non-plastic) w = 10.2%	9
15												
16		9	16.0	18.0	SS	14	15	18	28		Brown cmf SAND; some mf GRAVEL; trace SILT (wet, non-plastic) w = 9.5%	18
17												
18		10	18.0	20.0	SS	11	15	19	17		Brown SILT; some mf GRAVEL; some cmf- SAND (wet, non-plastic) w = 16.3%	10
19												
20		11	20.0	22.0	SS	17	18	17	21		Similar Soil (wet, non-plastic)	19
21												
22												
23												
24		12	24.0	26.0	SS	14	18	18	21		Brown cmf GRAVEL; some cmf SAND; trace SILT (wet,	8
25												

ATL-LOG1 LL CD10372 MCFARLAND JOHNSON - MARYLAND.GPJ ATL4-08.GDT 10/28/22

SS Split Spoon Sample
 NX Rock Core
 SH Undisturbed Sample (Shelby Tube)
 Estimated Groundwater

Drillers: Mark Childs; Greydenn Griffin
 Inspector: _____

ATLANTIC TESTING LABORATORIES, Limited

Subsurface Investigation

Boring No.: B-2

Report No.: CD10372E-01-10-22

Sheet 2 of 3

DEPTH	METHOD OF ADVANCE	SAMPLE NO.	DEPTH OF SAMPLE		SAMPLE TYPE	BLOWS ON SAMPLER PER 6" 2" O.D. SAMPLER	DEPTH OF CHANGE	CLASSIFICATION OF MATERIAL	RECOVERY (inches)	
			From	To						
26							32.0	non-plastic		
27										
28										
29		13	29.0	31.0	SS	17 16 28 24			Brown cmf SAND; some mf GRAVEL; trace SILT (wet, non-plastic)	7
30										
31										
32										
33										
34		14	34.0	36.0	SS	19 17 18 16			Grey mf GRAVEL; and cmf SAND; trace SILT (wet, non-plastic)	10
35										
36										
37										
38										
39		15	39.0	41.0	SS	12 9 12 17		Grey mf GRAVEL; trace cmf SAND; trace SILT (wet, non-plastic)	7	
40										
41										
42										
43										
44		16	44.0	46.0	SS	4 7 21 22		Grey cmf SAND; some mf GRAVEL; trace SILT (wet, non-plastic)	12	
45										
46										
47										
48										
49		17	49.0	51.0	SS	15 10 12 15		Grey cmf SAND; little f GRAVEL; trace SILT (wet, non-plastic)	12	
50										
51										
52										
53										
54		18	54.0	56.0	SS	16 8 20 18		Grey mf GRAVEL; trace cmf SAND; trace SILT (wet, non-plastic)	9	
55										
56										
57							57.0			
58							58.0	Encountered possible WEATHERED ROCK at 57.0 feet.		
59	NX E R O C (WET)		58.0	63.0	NX	RUN 1		Advanced casing to 57.0 feet. Advanced 2 15/16" tri-cone roller bit wet rotary open hole to 58.0 feet and began coring.	58	
60								Grey SILTSTONE		
61								58" or 97% Recovery		
62								7 Pieces (58") - 0% Chips and Fragments 4 Pieces longer than 4" (49") - RQD = 82%		

ATL-LOG1.LL CD10372 MCFARLAND JOHNSON - MARYLAND.GPJ ATL4-08.GDT 10/28/22

ATLANTIC TESTING LABORATORIES, Limited

Subsurface Investigation

Boring No.: B-2

Report No.: CD10372E-01-10-22

Sheet 3 of 3

DEPTH	METHOD OF ADVANCE	SAMPLE NO.	DEPTH OF SAMPLE		SAMPLE TYPE	BLOWS ON SAMPLER PER 6" 2" O.D. SAMPLER	DEPTH OF CHANGE	CLASSIFICATION OF MATERIAL	RECOVERY (inches)	
			From	To						
63			63.0	68.0	NX	RUN 2	63.0	Grey SILTSTONE 59" or 97% Recovery 7 Pieces (59") - 0% Chips and Fragments 7 Pieces longer than 4" (59") - RQD = 98%	59	
64										
65										
66										
67										
68							68.0			
69										
70										
71										
72										
73										
74										
75										
76										
77										
78										
79										
80										
81										
82										
83										
84										
85										
86										
87										
88										
89										
90										
91										
92										
93										
94										
95										
96										
97										
98										
99										
100										

ATL-LOG1 LL CD10372 MCFARLAND JOHNSON - MARYLAND.GPJ ATL4-08.GDT 10/28/22

f - fine
 m - medium
 c - coarse

and - 35-50%
 some - 20-35%
 little - 10-20%
 trace - 0-10%

Boring terminated at 68.0 feet.

Notes:
1. Borehole backfilled with on-site soils.

APPENDIX D
LABORATORY TEST REPORTS



ATLANTIC TESTING LABORATORIES

WBE certified company

LABORATORY DETERMINATION OF MOISTURE CONTENT OF SOILS ASTM D 2216

PROJECT INFORMATION

Client: McFarland Johnson
Project: Loft Rd over Schenevus Creek
Town of Maryland, New York

ATL Report No.: CD10372E-01-10-22
Report Date: October 27, 2022
Date Received: October 17, 2022

TEST DATA

Boring No.	Sample No.	Depth (ft)	Moisture Content (%)
B-1	S-2 ¹	2-4	7.3
	S-3 ¹	4-6	18.9
	S-4 ¹	6-8	9.8
	S-5 ¹	8-10	12.1
	S-6 ¹	10-12	14.3
	S-7 ¹	12-14	12.1
	S-8 ¹	14-16	9.5
	S-9 ¹	16-18	12.0
	B-2	S-2 ¹	2-4
S-3 ¹		4-6	8.8
S-4 ¹		6-8	12.5
S-5 ¹		8-10	10.2
S-6 ¹		10-12	20.3
S-7 ¹		12-14	9.2
S-8 ¹		14-16	10.2
S-9 ¹		16-18	9.5
S-10 ¹		18-20	16.3

REMARKS

1. Sample mass was less than the minimum mass outlined in the referenced test method.

Reviewed By:

Judith Ames

Date:

10/27/22



ATLANTIC TESTING LABORATORIES

Particle Size Distribution Report

Project: Loft Rd over Schenevus Creek, Town of Maryland, NY **Report No.:** CD10372E-01-10-22

Client: Mcfarland Johnson

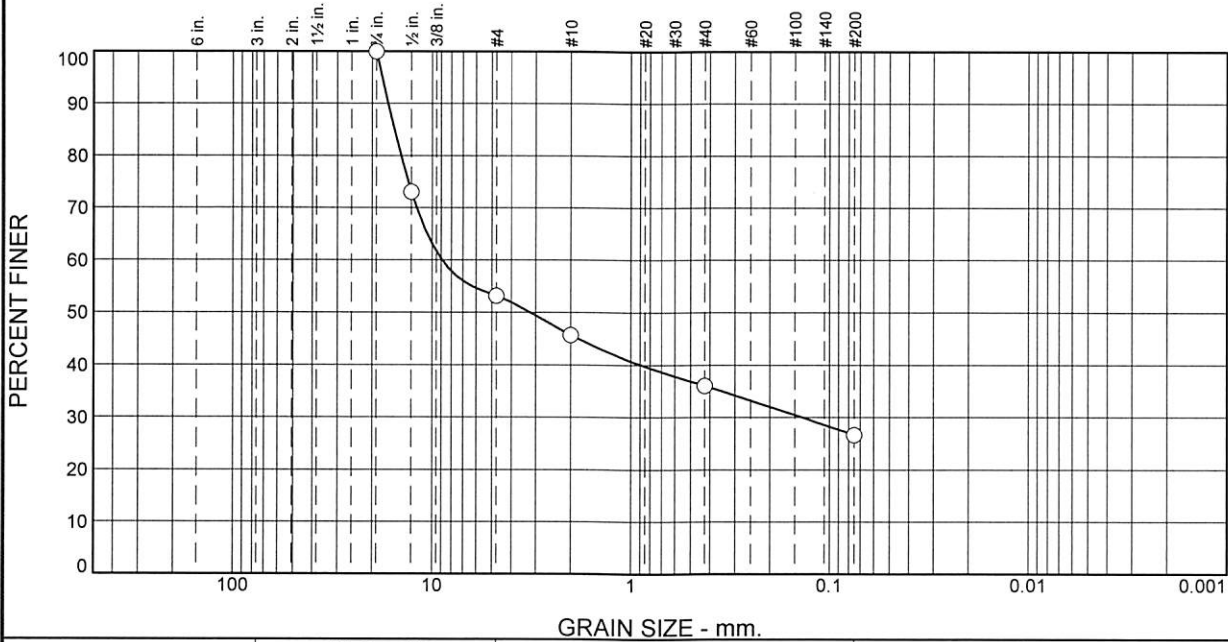
Date: 10/27/22

Sample No: B-1, S-3

Source of Sample: Boring Sample

Location: In-place

Elev./Depth: 4-6'



% Cobbles	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	0	47	7	10	9	27	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	OUT OF SPEC. (X)
3/4"	100		
1/2"	73		
#4	53		
#10	46		
#40	36		
#200	27		

* (no specification provided)

Soil Description
Brown f GRAVEL; some cmf SAND; little SILT; trace CLAY

Atterberg Limits
PL= -- LL= -- PI= --

Coefficients
D₈₅= 15.4532 D₆₀= 8.8996 D₅₀= 3.2108
D₃₀= 0.1374 D₁₅= D₁₀=
C_u= C_c=

Classification
USCS= AASHTO=

Remarks
Moisture Content = 18.9%

Figure

ATLANTIC TESTING LABORATORIES, LIMITED

Reviewed by: Jedez Ames

Date: 10/27/22



ATLANTIC TESTING LABORATORIES

Particle Size Distribution Report

Project: Loft Rd over Schenevus Creek, Town of Maryland, NY **Report No.:** CD10372E-01-10-22

Client: Mcfarland Johnson

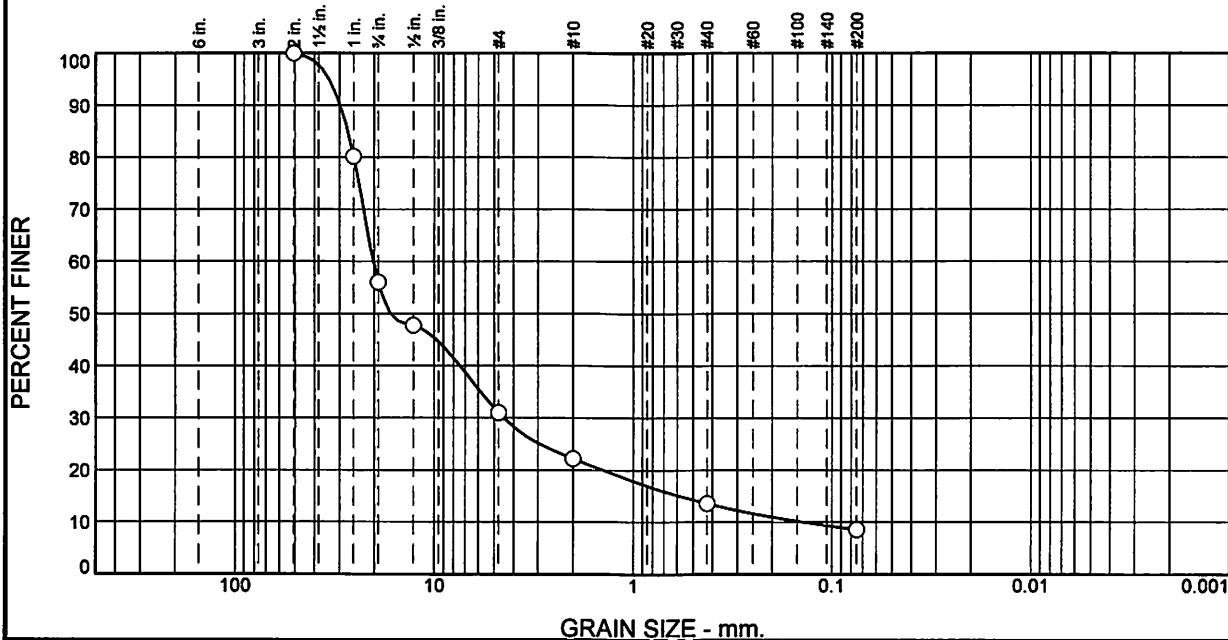
Date: 10/27/22

Sample No: B-1, S-7

Source of Sample: Boring Sample

Location: In-place

Elev./Depth: 12-14'



% Cobbles	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	44	25	9	8	5	9	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	OUT OF SPEC. (X)
2"	100		
1"	80		
3/4"	56		
1/2"	48		
#4	31		
#10	22		
#40	14		
#200	8.6		

Soil Description
Brown cmf GRAVEL; some cmf SAND; trace SILT

Atterberg Limits
PL= -- LL= -- PI= --

Coefficients
D₈₅= 27.1767 D₆₀= 20.1616 D₅₀= 16.4768
D₃₀= 4.4776 D₁₅= 0.5836 D₁₀= 0.1406
C_u= 143.36 C_c= 7.07

Classification
USCS= AASHTO=

Remarks
Moisture Content = 12.1%

* (no specification provided)

Figure

ATLANTIC TESTING LABORATORIES, LIMITED

Reviewed by: Judith Amas

Date: 10/27/22



ATLANTIC TESTING LABORATORIES

Particle Size Distribution Report

Project: Loft Rd over Schenevus Creek, Town of Maryland, NY **Report No.:** CD10372E-01-10-22

Client: Mcfarland Johnson

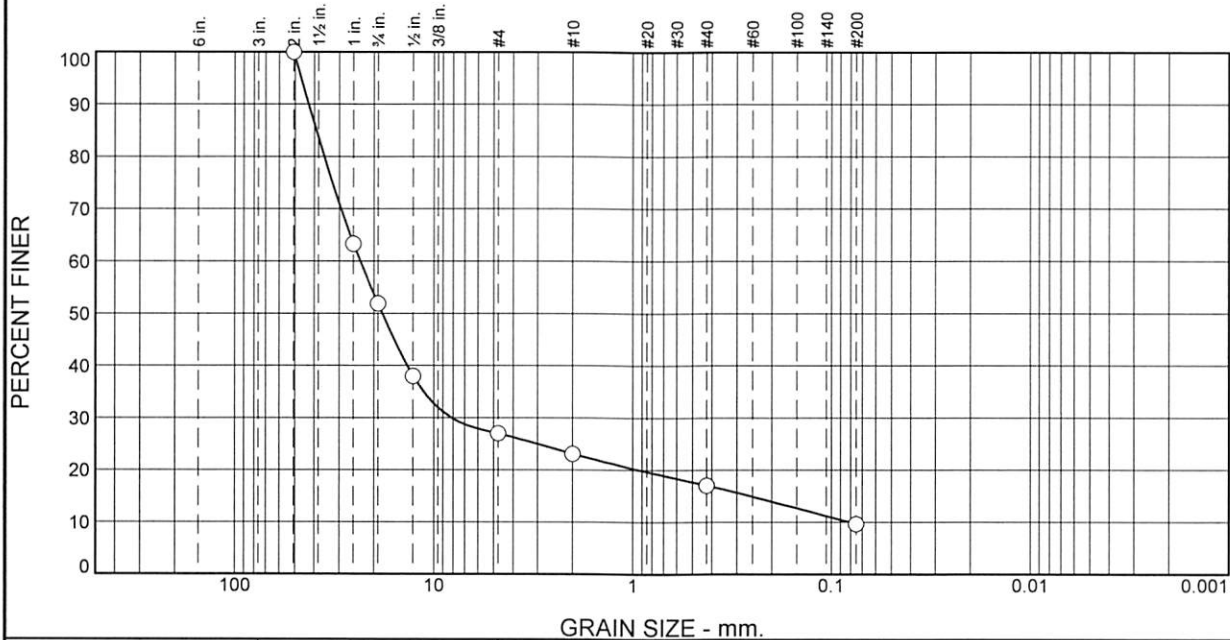
Date: 10/27/22

Sample No: B-2, S-3

Source of Sample: Boring Sample

Location: In-place

Elev./Depth: 4-6'



% Cobbles	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	48	25	4	6	7	10	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	OUT OF SPEC. (X)
2"	100		
1"	63		
3/4"	52		
1/2"	38		
#4	27		
#10	23		
#40	17		
#200	9.6		

Soil Description
Brown cmf- GRAVEL; little cmf SAND; trace SILT

Atterberg Limits
PL= -- LL= -- PI= --

Coefficients
D₈₅= 39.0391 D₆₀= 23.4752 D₅₀= 18.1049
D₃₀= 8.1164 D₁₅= 0.2568 D₁₀= 0.0814
C_u= 288.23 C_c= 34.45

Classification
USCS= AASHTO=

Remarks
Moisture Content = 8.8%

* (no specification provided)

Figure

ATLANTIC TESTING LABORATORIES, LIMITED

Reviewed by: Judith Ames

Date: 10/27/22



Particle Size Distribution Report

Project: Loft Rd over Schenevus Creek, Town of Maryland, NY **Report No.:** CD10372E-01-10-22

Client: Mcfarland Johnson

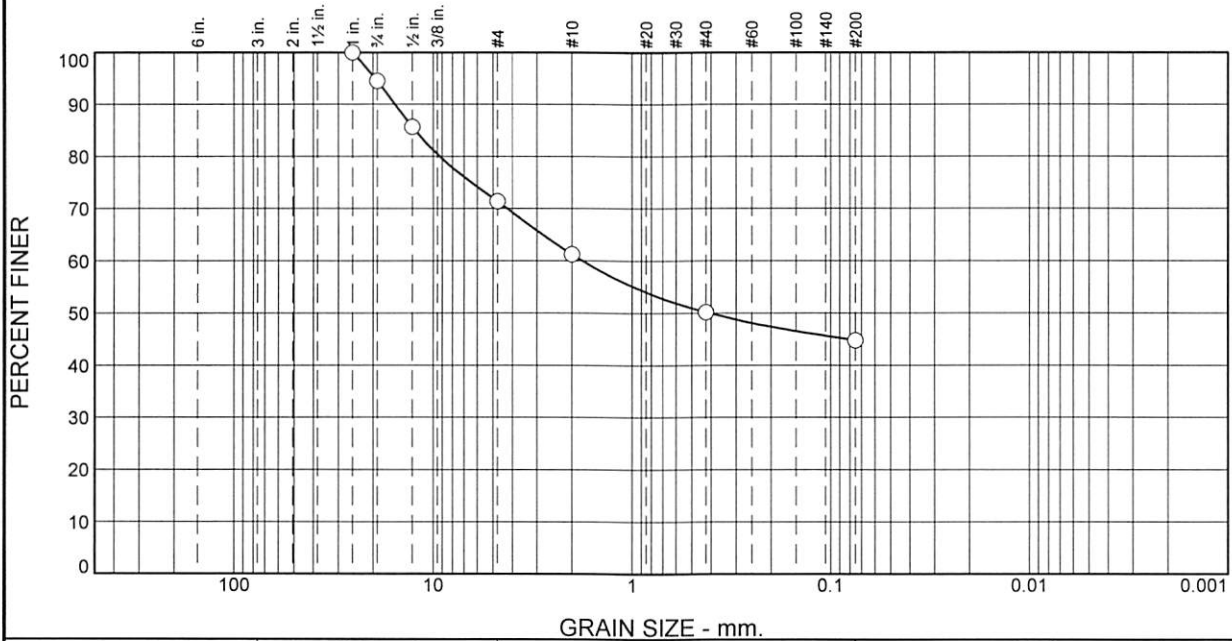
Date: 10/27/22

Sample No: B-2, S-10

Source of Sample: Boring Sample

Location: In-place

Elev./Depth: 18-20'



% Cobbles	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay
0	5	24	10	11	5	45	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	OUT OF SPEC. (X)
1"	100		
3/4"	95		
1/2"	86		
#4	71		
#10	61		
#40	50		
#200	45		

Soil Description
Brown SILT; some mf GRAVEL; some cmf- SAND

Atterberg Limits
PL= -- LL= -- PI= --

Coefficients
D₈₅= 12.3167 D₆₀= 1.7722 D₅₀= 0.4094
D₃₀= D₁₅= D₁₀=
C_u= C_c=

Classification
USCS= AASHTO=

Remarks
Moisture Content = 16.3%

* (no specification provided)

Figure

ATLANTIC TESTING LABORATORIES, LIMITED

Reviewed by: Judge Ames

Date: 10/27/22



ATLANTIC TESTING LABORATORIES

WBE certified company

PROJECT INFORMATION

Client: McFarland Johnson
Project: Loft Road over Schenevus Creek
Town of Maryland, New York

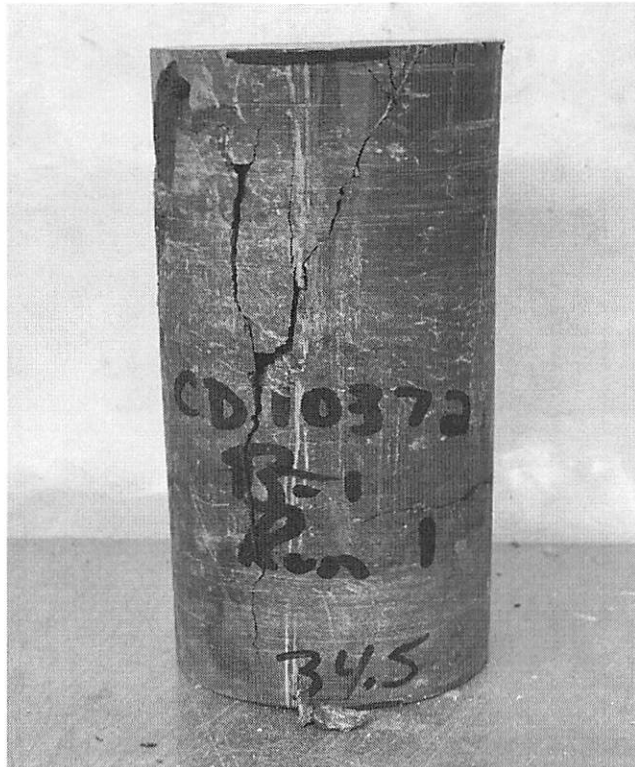
ATL Report No.: CD10372E-01-10-22
Report Date: October 27, 2022
Date Received: October 17, 2022

UNCONFINED COMPRESSIVE STRENGTH OF INTACT ROCK CORE SPECIMENS ASTM D 7012, Method C

Boring No.	Run No.	Depth (ft)	Diameter (in)	Length (in)	Load Rate (lbs/sec)	Total Load (lbs)	Area (in ²)	Compressive Strength (psi)	Calculated Density (pcf)
1	1	34-39	1.98	4.108	270	41,010	3.08	13,310	170

Failure Sketch

B-1, Run-1
Depth: 34-39'



Reviewed By:

Judith Ames

Date:

10/27/22