S.W.COLE ENGINEERING, INC.

REPORT

21-0414 S

December 23, 2022

Explorations and Geotechnical Engineering Services

Proposed Bridge Replacement
George Street over Beaver Brook
Keene, New Hampshire

Prepared For:
McFarland Johnson, Inc.
Attention: Samuel White, P.E.
53 Regional Drive
Concord, NH 03301

Prepared By:
S. W. Cole Engineering, Inc.
10 Centre Road
Somersworth, New Hampshire 03878
T: 603-692-0088

TABLE OF CONTENTS

21-0414 S

December 23, 2022

McFarland Johnson, Inc.
Attention: Samuel White, P.E.
53 Regional Drive
Concord, NH 03301

Subject: Explorations and Geotechnical Engineering Services
Proposed Bridge Replacement
George Street over Beaver Brook
Keene, New Hampshire

Dear Sam:

In accordance with our Revised Proposal dated August 2, 2021, we have performed subsurface explorations for the subject project. This report summarizes our findings and geotechnical recommendations, and its contents are subject to the limitations set forth in Appendix A.

1.0 INTRODUCTION

1.1 Scope and Purpose

The purpose of our services was to explore subsurface conditions at the site in order to provide geotechnical recommendations for design of the proposed bridge replacement. Our scope of services included two test borings and one environmental test boring, soils laboratory testing, a geotechnical evaluation of the findings relative to proposed construction, and preparation of this report.

1.2 Site and Proposed Construction

The site is located on George Street at its crossing of Beaver Brook in Keene, New Hampshire. The site consists of an existing 13-foot single-span bridge across Beaver Brook in an east-west direction. Based on the provided plans, site grades are relatively flat along George Street at about Elevation 488 feet. Site grades slope downward to

Beaver Brook at the crossing to about elevation 479 to 480 feet.

Based on correspondence with you, we understand proposed construction will include a new bridge spanning 20 to 30 feet across Beaver Brook. We understand replacement structures under consideration include:

- Concrete box culvert recessed into the brook.
- Three-sided rigid concrete frame supported on spread footings; or
- Shallow stub abutments supported on single row of driven piles.

We understand the new bridge will be constructed on the existing horizontal alignment and have a similar vertical grade as the existing bridge.

2.0 EXPLORATION AND TESTING

2.1 Explorations

Two geotechnical test borings (B-1 and B-2) and one environmental test boring (ENV-1) were made at the site on December 5 and 6, 2022, by S. W. Cole Explorations, LLC. The test borings were advanced utilizing cased rotary-wash drilling techniques. Standard Penetration Testing (SPT) and split-spoon sampling were performed at 2-foot intervals to a depth of 15 feet than at 5-foot intervals, thereafter. Upon encountering a refusal surface, test borings B-1 and B-2 were advanced 5 feet into bedrock using a NQ2 rock core barrel. The exploration locations were selected by S. W. Cole Engineering, Inc. (S.W.COLE) in consultation with Sanborn, Head & Associates, Inc. (SHA) (project environmental consultant) and established in the field by S.W.COLE using taped measurements from existing site features.

Soils in the test borings were screened by SHA using a MiniRAE 2000 photoionization detector (PID). PID results are noted on the boring logs.

The approximate exploration locations are shown on the “Exploration Location Plan,” included in Appendix B. Exploration logs and a key to the notes and symbols used on the logs are included in Appendix C. The elevations shown on the logs were estimated based on topographic information shown on the “Exploration Location Plan”.

2.2 Laboratory Testing

Soil samples obtained from the explorations were returned to our laboratory for further classification and testing. Gradation and moisture content testing was performed on two select soil samples. Gradation test results are included in Appendix D. Moisture content test results are shown on the boring logs.

3.0 SUBSURFACE CONDITIONS

3.1 Soil and Bedrock

3.1.1 West Abutment (B-1)

Test boring B-1 was preformed on the west side of the existing bridge and encountered a surficial asphalt pavement layer overlying granular fills consisting of loose to dense, brown sand with varying portions of silt and gravel to a depth of 9 feet. The fill was underlain by alluvial deposits of loose to medium dense, brown sand and gravel, with some silt to a depth of 15 feet. Below the alluvial deposit, the test boring encountered glacial till soils consisting of medium dense, gray gravelly sand some silt to a depth of 20 feet where bedrock was encountered. Bedrock recovery was minimal and consisted of hard, slightly weathered, moderately to slightly fractured, fine to medium grained, gray to black Granodiorite, with very close to close, low to high angle joints with a Rock Quality Designation (RQD) of 10% indicating very poor rock quality.

3.1.2 East Abutment (B-2)

Test boring B-2 was made on the east side of the existing bridge and encountered a surficial 1.5 feet of grassed topsoil overlying granular fills consisting of medium dense silty sand with varying portions of gravel to a depth of 7 feet. Below the fill, the test boring encountered alluvial deposits of medium dense to very dense, brown gravelly sand with varying portions of silt to a depth of 17 feet. Below the alluvial deposit, the test boring encountered glacial till soils consisting of very dense, gray silty gravelly fine to medium sand to a depth of 21 feet, where bedrock was encountered. Bedrock consisted of hard, slightly weathered, moderately to slightly fractured, fine to medium grained, gray to black Granodiorite, with very close to close, low to high angle joints. The bedrock sampled had an RQD equal to 62% indicating fair rock quality.

3.1.3 Environmental Boring (ENV-1)

Environmental boring ENV-1 was performed in an open grassed area to the northwest of the bridge and encountered a surficial 1.5 feet of grassed topsoil overlying granular fills consisting of loose to medium dense sand with varying portions of silt and gravel to a depth of 9 feet, overlying very loose to loose, sand and sandy silt (alluvial deposits) to a depth of 15 feet, where the exploration was terminated.

3.2 Groundwater

Free water was observed at depths ranging from 8 to 9 feet in the test borings at the time of exploration work. Long term groundwater information is not available. It should be anticipated that groundwater levels will fluctuate, particularly in response to periods of snowmelt and precipitation, changes in site use and the water level of Beaver Brook.

4.0 SEISMIC CONSIDERATIONS

4.1 Bedrock Acceleration and Site Response

Seismic site class was evaluated in accordance with AASHTO LRFD 2020 Article 3.10.3.1 using the average SPT N-value method. Based on the information obtained in the explorations, the average N-value was between 15 and 50 blows per foot corresponding to an AASHTO Site Class D as defined in AASHTO Table 3.10.3.1-1.

United States Geological Survey (USGS) Seismic Design Parameters program (Version 2.1) was used to obtain the seismic design parameters for the bridge. Based on the assigned site class (AASHTO Site Class D) and the project site coordinates, the software provides the recommended AASHTO Response Spectrum for a 7 percent probability of exceedance in 75 years. The results for the project site are summarized below:

Site Class D Seismic Design Parameters Site Coordinates: N42.944645°, W72.271597°	
Parameter	**Design Value**
PGA	0.069 g
S_s	0.150 g
S_1	0.043 g
F_{pga}	1.60
F_a	1.60
F_v	2.40
A_s	0.111 g
S_{DS}	0.240 g
S_{D1}	0.103 g
Seismic Zone (based on S_{D1})	Zone 1

Based on the Acceleration Coefficient S_{D1}=0.103 g and AASHTO Article 3.10.6, this site is assigned to Seismic Zone 1. Per AASHTO Article 4.7.4, single span bridges are not required to be analyzed for seismic loads, however the requirements of AASHTO Articles 4.7.4.4 and 3.10.9 shall apply. Per AASHTO Article 3.10.1 the seismic effects for box culverts and buried structures need not be considered except where they cross active faults. There are no known active faults within the site vicinity.

4.2 Liquefaction Assessment

Liquefaction is typically observed in saturated deposits of loose sands and non-plastic silts subjected to ground shaking most commonly from earthquakes. The foundation soils at the proposed abutments are anticipated to consist of native, medium dense to very dense sand or glacial till. Based on the site soils, we anticipate the risk of seismically induced liquefaction below the foundations is low.

5.0 FOUNDATION EVALUATION AND RECOMMENDATIONS

5.1 General

S.W.COLE has conducted geotechnical engineering evaluations in accordance with 2020 AASHTO LRFD Bridge Design Specifications, 9th Edition (AASHTO) and the NHDOT Bridge Design Manual, 2015 Version 2 Edition with November 2020 revisions (NHDOT BDM).

5.2 Frost Considerations

The 100-year freezing index for the Keene, New Hampshire area is approximately 1,300 Fahrenheit degree-days. Considering this, we recommend spread footing foundations or pile caps and grade beams for the abutments and wing walls have at least 4.5 feet of soil cover to provide frost protection. Riprap for scour protection should not be considered as part of the soil cover.

5.3 Foundation Options and Discussion

We understand the proposed bridge replacement alternatives consist of a box culvert, three-sided rigid frame or pile supported bridge integral abutment bridge.

Subsurface conditions at the site consist of fills overlying native sand and gravel, overlying glacial till mantling bedrock at depths of about 20 to 21 feet. Based on the subsurface findings, it is our opinion that driven piles will be too short to develop lateral resistance depending on the bottom of pile cap elevation. Therefore, we anticipate abutment support by cast-in-place spread footings founded below scour or drilled micropiles are feasible foundation options for the bridge replacement.

Uncontrolled fills were encountered at each abutment extending to depths ranging from 7 to 9 feet below ground surface. Uncontrolled fills and soils containing organic material must be completely removed below foundations and from areas of proposed construction and replaced with compacted fill. It is anticipated that these unsuitable soils will be removed during anticipated excavations for the proposed bridge; however, over-excavations, if needed, should extend out one foot horizontally for each foot of over-excavation (1H:1V bearing splay).

It is anticipated that use of spread footings may require a deeper foundation embedment than frost depth for scour protection, potentially increasing efforts associated with excavation and dewatering. It will be important to properly dewater excavations to allow for observation of the bearing surface.

5.4 Foundation Design

5.4.1 Precast Box Culvert

We anticipate a box culvert will be recessed into the brook and founded on medium dense to very dense sand and gravel (alluvial deposits) or medium dense to very dense glacial till soils. We recommend precast box culvert be recessed below scour and supported on at least 12 inches of compacted Crushed Stone overlying properly prepared subgrades. We recommend the precast box culvert include toe walls at the inlet and outlet ends to prevent undermining. Based on the subsurface conditions and our understanding of the proposed construction, we recommend the following foundation design parameters:

GEOTECHNICAL FOUNDATION DESIGN PARAMETERS	
Design Frost Depth	4.5 feet
Strength Limit Sate – Article 10.6.3.1.1	
Nominal Bearing Resistance	12.4 ksf
Strength Limit Bearing Resistance Factor (AASHTO Table 10.5.5.2.2-1)	0.45
Factored Bearing Resistance	5.6 ksf
Estimated Post-Construction Settlement	1.5 inches or less
Service Limit – AASHTO Table C10.6.2.6.1-1	
Nominal Bearing Resistance	4.0 ksf
Service Limit Bearing Resistance Factor	1.0
Factored Bearing Resistance	4.0 ksf
Estimated Post-Construction Settlement	1.5 inches or less

5.4.2 Spread Footing Foundations

We anticipate spread footing foundations will likely be founded on medium dense to very dense sand and gravel (alluvial deposits) or medium dense to very dense glacial till soils. We recommend cast-in-place spread footings be founded below scour and supported on at least 12 inches of compacted Crushed Stone overlying properly prepared subgrades. The purpose of the Crushed Stone layer is intended to create a level and stable working mat and provide a media to sump and pump. Voids left by the removal of cobbles or boulders should be backfilled with additional Crushed Stone. Based on the subsurface conditions and our understanding of the proposed construction, we recommend the following foundation design parameters:

GEOTECHNICAL FOUNDATION DESIGN PARAMETERS	
Design Frost Depth	4.5 feet
Strength Limit Sate – Article 10.6.3.1.1	
Nominal Bearing Resistance	11.7 ksf
Strength Limit Bearing Resistance Factor (AASHTO Table 10.5.5.2.2-1)	0.45
Factored Bearing Resistance	5.3 ksf
Estimated Post-Construction Settlement	1 inch or less
Service Limit – AASHTO Table C10.6.2.6.1-1	
Nominal Bearing Resistance	5.0 ksf
Service Limit Bearing Resistance Factor	1.0
Factored Bearing Resistance	5.0 ksf
Estimated Post-Construction Settlement	1 inch or less

We recommend a minimum footing width of 6 feet. S.W.COLE should be given the opportunity to review the proposed foundation layout during final design to adjust our recommendations based on the actual proposed foundation configuration and dimensions.

It is anticipated that use of spread footings may require a deeper foundation embedment than frost depth for scour protection, potentially increasing efforts associated with excavation and dewatering. It will be important to properly dewater excavations to allow for observation of the bearing surface.

5.4.3 Micropiles

We anticipate micropiles will consist of a cased section from the bottom of pile cap down to the top of competent bedrock with an uncased section below. The micropiles will be reinforced with a single, continuously threaded central bar running the full length of the micropile and filled with 5,000 psi Portland cement grout. In accordance with LRFD Article 10.9.1.2, center-to-center micropile spacing should not be less than 30 inches or 3 pile diameters, whichever is greater.

5.4.3.1 Axial Resistance

Rock-socketed micropiles will generally develop axial resistance through side friction in the rock socket. For design, per LRFD Table C10.9.3.5.2-1, we recommend a presumptive nominal grout-to-ground bond resistance of 20 ksf for the granodiorite bedrock. Per LRFD Table 10.5.5.2.5-1, at the strength limit state, axially loaded micropiles shall be designed using a geotechnical resistance factor ϕ_{stat} of 0.70 provided pile load testing is performed. If no load test is performed, a geotechnical resistance factor ϕ_{stat} of 0.55 shall be used.

Micropile axial resistance is dependent on pile diameter and grout-to-ground bond length. A summary of estimated factored strength-limit axial geotechnical resistances for 7.5 and 9.5-inch diameter micropile with various bond lengths are provided in the following table:

Embedment in Competent Bedrock (ft)	Strength Limit Factored Axial Geotechnical Resistance (kips)[1]	
	7.5-inch Uncased Diameter[2]	9.5-inch Uncased Diameter[3]
10	274	347
15	411	520
20	548	694

Notes: 1. Resistance factor of 0.7 used for the strength limit state
2. 8.625-inch OD, 7.625-inch ID casing to top of rock, assumed 7.5-inch diameter rock-socket
3. 10.75-inch OD, 9.75-inch ID casing to top of rock, assumed 9.5-inch diameter rock-socket

Additional micropile grout-to-ground bond lengths may be considered depending on the actual factored design axial loads. S.W.COLE can provide additional input on micropile size once abutment loading has been developed for the proposed structure. Final axial design of micropiles shall be performed by the micropile specialty contractor during construction-phase, based on pile load test results and selected means and methods.

5.4.3.2 Downdrag

We anticipate settlement in the native soils generated from the load applied by embankment fills will be elastic and occur during construction with negligible long-term settlement. Therefore, downdrag is not considered to be an issue.

5.4.3.3 Lateral Resistance

The micropiles will be subjected to lateral loading; therefore, the micropiles shall be evaluated for resistance against combined axial compression and flexure in accordance with LRFD Table Article 10.7.3.9. Lateral resistance can also be derived from the use of battered piles.

5.4.3.4 Service and Extreme Limit State Design

The design of micropiles at the service limit state shall consider tolerable transverse and longitudinal movement of piles and pile group movement considering changes in soil conditions due to scour based on the design flood (Q_{100}). For the service limit state, resistance factors of φ = 1.0 should be used in accordance with LRFD Article 10.5.5.1. The exception is the overall global stability of the foundation which should be investigated at the Service I load combination and a resistance factor, φ, of 0.65.

Extreme limit state design shall include pile axial compressive resistance, overall global stability of the pile group, pile failure by uplift in tension, and structural failure. The extreme event load combinations are those related to seismic forces, ice loads, debris loads, and hydraulic events. Extreme limit state design shall also check that the nominal pile foundation resistance remaining after scour due to the check flood (Q_{500}) can support the extreme limit state loads. Resistance factors for extreme limit states, per LRFD Article 10.5.5.3, shall be taken as φ = 1.0 except for uplift of piles, for which the resistance factor, φ_{up}, shall be 0.80 or less per LRFD Article 10.5.5.3.2.

5.5 Abutments and Wingwalls

The material properties will be controlled by the backfill which is anticipated to consist of NHDOT Granular Backfill (Bridge) Item 209. We assume the bridge will be backfilled with free-draining, granular wall backfill resulting in a drained condition (i.e. no hydrostatic pressure) within the wall backfill. Based on the use of a free-draining, granular wall backfill, we recommend design consider the following parameters:

GEOTECHNICAL PARAMETERS FOR ABUTMENT AND WING WALLS	
Total Unit Weight of Backfill (γ_t)	125 pcf
Internal Friction Angle of Backfill (NHDOT Granular Backfill Item 209)	32°
Active Lateral Earth Pressure Coefficient (K_a)	0.3
At-Rest Lateral Earth Pressure Coefficient (K_o)	0.5
Passive Lateral Earth Pressure (K_p)	3.3

AASHTO recommends that live load surcharge be applied as a uniform lateral surcharge pressure using an equivalent fill height. Recommendations for equivalent lateral surcharge height are provided in AASHTO Article 3.11.6.4 based on wall height and distance from the wall back face to the edge of traffic.

5.6 Excavation and Dewatering Considerations

The excavations to foundation subgrade will generally encounter granular fills overlying native sands with varying portions of silt and gravel, overlying glacial till and bedrock. The foundation should be underlain with a minimum 6 inches of Crushed Stone (NHDOT #57

Crushed Stone). This will help provide a stable surface from which to construct forms and provide a media from which to collect, sump, and pump groundwater.

Excavations below the water level of Beaver Brook will be difficult and will likely need sheetpiles for groundwater cutoff and river flow diversion to help control groundwater. Controlling the water levels to at least one foot below planned excavation depths will help stabilize subgrades during construction. Surface water should be diverted from entering the foundation excavation.

Excavations must be properly shored and/or sloped to prevent sloughing and caving of the sidewalls during construction. All excavations should be performed in accordance with OSHA requirements. The contractor is responsible for developing an appropriate dewatering and excavation plans to install the foundations and maintain stable subgrades.

5.7 Backfill and Compaction

Embankment fill for approaches and backfill placed adjacent to the new abutments and wing walls should be clean, non-frost susceptible sand and gravel meeting the gradation requirements for NHDOT Granular Backfill (Bridge) Item 209.

Fill should be placed in horizontal lifts and be compacted. Lift thickness should be limited to that which can be thoroughly compacted using small, hand operated compaction equipment to avoid over compaction of material within 3 feet of abutment and wing walls. We recommend fill against the proposed structure and wing walls be compacted to between 95 to 98 percent of its maximum dry density as determined by AASHTO T-99.

If foundation construction takes place during cold weather conditions, subgrades and foundations must be protected from freezing conditions.

5.8 Design Review and Construction Testing

We recommend S.W.COLE be provided the opportunity to review recommendations in this report and make modifications as necessary once the final design for the replacement structure has been determined. S.W.COLE should be retained to review the final design and specifications to determine that our earthwork and foundation recommendations have been properly interpreted and implemented.

Further, we recommend S.W.COLE be retained to provide soils engineering and testing services during the excavation and foundation phases of the work. This is to observe compliance with the design concepts, specifications, and design recommendations and to allow design changes in the event that subsurface conditions are found to differ from those anticipated prior to the start of construction. S.W.COLE is available to provide testing of soil, concrete, and asphalt construction materials.

6.0 CLOSURE

It has been a pleasure to be of assistance to you with this phase of your project. We look forward to working with you during the construction phase of the project.

Sincerely,

S. W. Cole Engineering, Inc.

Tyler Demers

Digitally signed by Tyler Demers
DN: dc=lan, dc=SWCole, ou=SWCole Users, ou=Somersworth, cn=Tyler Demers
Date: 2022.12.23 12:08:18 -05'00'
Adobe Acrobat version: 2022.003.20282

STATE OF NEW HAMPSHIRE
TYLER SCOTT DEMERS
No.16284
LICENSED
PROFESSIONAL ENGINEER

Tyler S. Demers, P.E
Project Geotechnical Engineer

TSD:mas

APPENDIX A

Limitations

This report has been prepared for the exclusive use of McFarland Johnson, Inc. for specific application to the Proposed Bridge Replacement on George Street Over Beaver Brook in Keene, New Hampshire. S. W. Cole Engineering, Inc. (S.W.COLE) has endeavored to conduct our services in accordance with generally accepted soil and foundation engineering practices. No warranty, expressed or implied, is made.

The soil profiles described in the report are intended to convey general trends in subsurface conditions. The boundaries between strata are approximate and are based upon interpretation of exploration data and samples.

The analyses performed during this investigation and recommendations presented in this report are based in part upon the data obtained from subsurface explorations made at the site. Variations in subsurface conditions may occur between explorations and may not become evident until construction. If variations in subsurface conditions become evident after submission of this report, it will be necessary to evaluate their nature and to review the recommendations of this report.

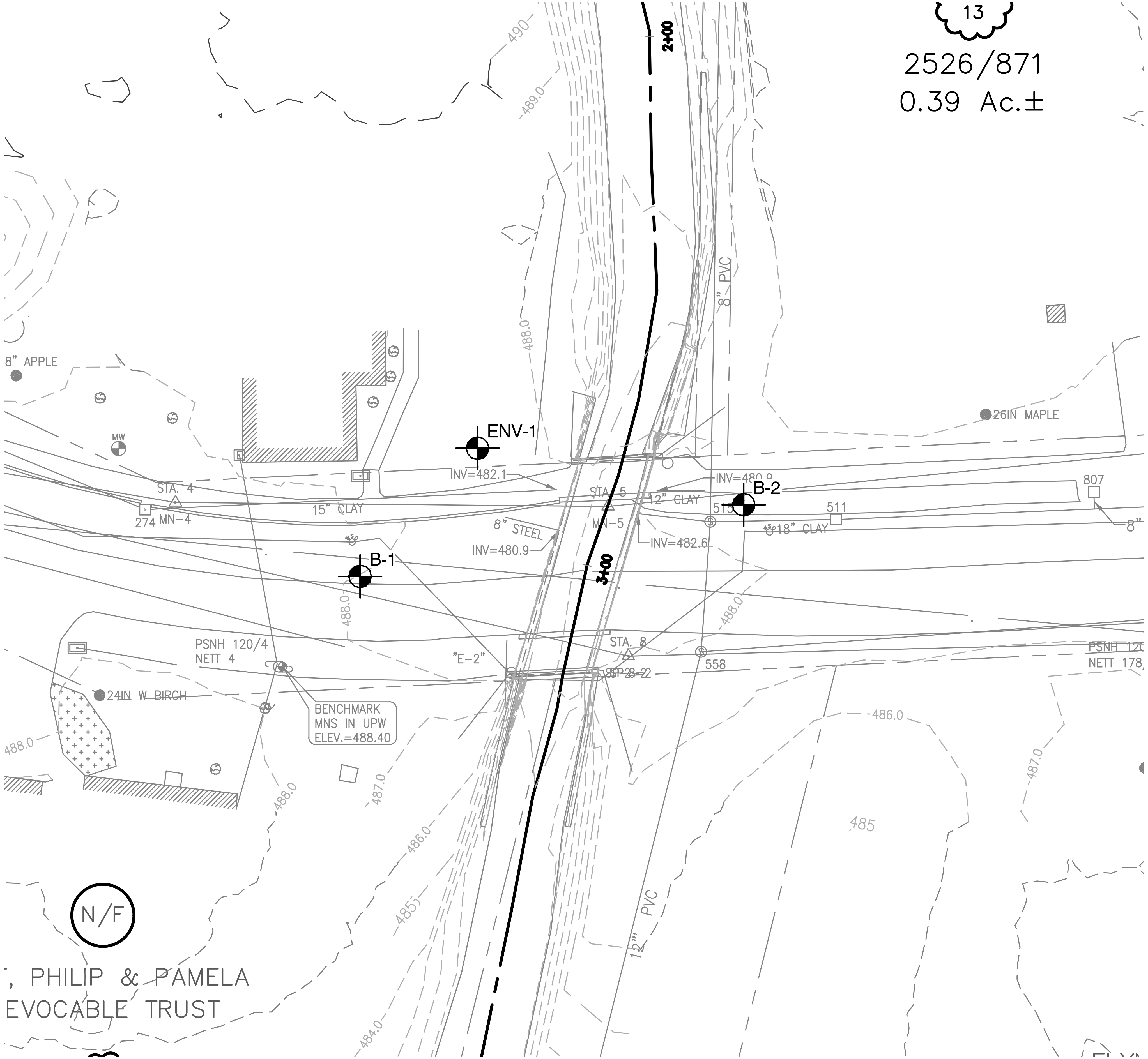
Observations have been made during exploration work to assess site groundwater levels. Fluctuations in water levels will occur due to variations in rainfall, temperature, and other factors.

S.W.COLE's scope of services has not included the investigation, detection, or prevention of any Biological Pollutants at the project site or in any existing or proposed structure at the site. The term "Biological Pollutants" includes, but is not limited to, molds, fungi, spores, bacteria, and viruses, and the byproducts of any such biological organisms.

Recommendations contained in this report are based substantially upon information provided by others regarding the proposed project. In the event that any changes are made in the design, nature, or location of the proposed project, S.W.COLE should review such changes as they relate to analyses associated with this report. Recommendations contained in this report shall not be considered valid unless the changes are reviewed by S.W.COLE.

APPENDIX B

Figures



LEGEND:

APPROXIMATE BORING LOCATION

NOTES:

1. EXPLORATION LOCATION PLAN WAS PREPARED FROM A SCALE PLAN OF THE SITE ENTITLED "EXISTING CONDITIONS SURVEY," PREPARED BY GM2 ASSOCIATES, INC., DATED 10/3/2022.
2. THE BORINGS WERE LOCATED IN THE FIELD BY TAPED MEASUREMENTS FROM EXISTING SITE FEATURES.
3. THIS PLAN SHOULD BE USED IN CONJUNCTION WITH THE ASSOCIATED S. W. COLE ENGINEERING, INC. GEOTECHNICAL REPORT.
4. THE PURPOSE OF THIS PLAN IS ONLY TO DEPICT THE LOCATION OF THE EXPLORATIONS IN RELATION TO THE EXISTING CONDITIONS AND PROPOSED CONSTRUCTION AND IS NOT TO BE USED FOR CONSTRUCTION.

0 20 40 Feet

S.W.COLE
ENGINEERING, INC.

MCFARLAND JOHNSON, INC.

EXPLORATION LOCATION PLAN

PROPOSED BRIDGE REPLACEMENT
GEORGE STREET OVER BEAVER BROOK
KEENE, NEW HAMPSHIRE

Job No.:	21-0414	Scale:	1" = 20'
Date :	12/22/2022	Sheet:	1

R:\2021\21-0414\CAD\Drawings\21-0414 ELP.dwg, 12/22/2022 2:16:19 PM, 1:1, CEM, S. W. Cole Engineering, Inc.

APPENDIX C

Exploration Logs and Key

BORING LOG

S.W.COLE

CLIENT: McFarland Johnson, Inc.
PROJECT: Proposed Bridge Replacement
LOCATION: George Street, Keene, New Hampshire

BORING NO.: B- 1
SHEET: 1 of 1
PROJECT NO. 21-0414
DATE START: 12/6/2022
DATE FINISH: 12/6/2022

Drilling Information

LOCATION: See Exploration Location Plan | **ELEVATION (FT):** 488' +/- | **TOTAL DEPTH (FT):** 25.0 | **LOGGED BY:** Sean Hlywa
DRILLING CO.: S. W. Cole Explorations, LLC | **DRILLER:** Matt Bussey | **DRILLING METHOD:**
RIG TYPE: Track Mounted Mobile Drill B-48 | **AUGER ID/OD:** N/A / N/A | **SAMPLER:** Standard Split-Spoon
HAMMER TYPE: Automatic | **HAMMER WEIGHT (lbs):** 140 | **CASING ID/OD:** N/A /N/A | **CORE BARREL:** NQ2
HAMMER EFFICIENCY FACTOR: 0.91 | **HAMMER DROP (inch):** 30
WATER LEVEL DEPTHS (ft): ∇ 9 ft Water observed at 9.0 feet on 12/6/2022
GENERAL NOTES:

KEY TO NOTES AND SYMBOLS:
Water Level
∇ At time of Drilling
▼ At Completion of Drilling
▼ After Drilling

D = Split Spoon Sample
U = Thin Walled Tube Sample
R = Rock Core Sample
V = Field Vane Shear

Pen. = Penetration Length
Rec. = Recovery Length
bpf = Blows per Foot
mpf = Minute per Foot

WOR = Weight of Rods
WOH = Weight of Hammer
RQD = Rock Quality Designation
PID = Photoionization Detector

S_v = Field Vane Shear Strength, kips/sq.ft.
q_U = Unconfined Compressive Strength, kips/sq.ft.
Ø = Friction Angle (Estimated)
N/A = Not Applicable

Elev. (ft)	Depth (ft)	Casing Pen. (bpf)	Sample No.	Type	Sample Depth (ft)	Pen./ Rec. (in)	Blow Count or RQD	Field / Lab Test Data / PID Readings	Strata Depth	Sample Description & Classification	H_20 Depth	Remarks
										Asphalt Pavement		
			1D	D	1-3	24/18	29-21-19-13	PID=0 ppm	1.0	Dense, brown, Gravelly Silty fine to medium SAND (FILL)		
485			2D	D	3-5	24/15	5-6-4-6	PID=0 ppm				
									3.7	Loose, brown, SILT and fine SAND (Fill)		
	5		3D	D	5-7	24/8	13-17-11-7	PID=.1 ppm	5.0	Medium dense, brown, Gravelly Silty SAND (FILL)		
			4D	D	7-9	24/3	7-9-7-11	PID=0 ppm				
480									9.0	Loose to medium dense, brown, SAND and GRAVEL some silt	∇	
	10		5D	D	10-12	24/2	5-4-6-4	PID=0 ppm				
			6D	D	12-14	24/7	5-7-7-10	PID=0 ppm ID 22365s w =12.3 %				
475												
	15		7D	D	15-17	24/1	15-12-10-5	PID=0 ppm	15.0	Medium dense, gray, Gravelly SAND some silt (Glacial Till)		
470												
	20		1R	R	20-25	60/15	10		20.0	Bedrock: Hard, slightly weathered, moderately to slightly fractured, fine to medium grained, gray to black, Granodiorite. Joints are very close to close, low to high angle. RQD = 10%		
465												
	25											

Bottom of Exploration at 25.0 feet
Probable Boulder or Bedrock

Stratification lines represent approximate boundary between soil types, transitions may be gradual. Water level readings have been made at times and under conditions stated. Fluctuations of groundwater may occur due to other factors than those present at the time measurements were made.

BORING / WELL 10-12-2022 21-0414.GPJ SWCE TEMPLATE.GDT 12/23/22

BORING LOG

S.W.COLE

CLIENT: McFarland Johnson, Inc.
PROJECT: Proposed Bridge Replacement
LOCATION: George Street, Keene, New Hampshire

BORING NO.: B- 2
SHEET: 1 of 1
PROJECT NO. 21-0414
DATE START: 12/5/2022
DATE FINISH: 12/5/2022

Drilling Information

LOCATION: See Exploration Location Plan
ELEVATION (FT): 488' +/-
TOTAL DEPTH (FT): 26.1
LOGGED BY: Sean Hlywa
DRILLING CO.: S. W. Cole Explorations, LLC
DRILLER: Matt Bussey
DRILLING METHOD: Cased Boring
RIG TYPE: Track Mounted Mobile Drill B-48
AUGER ID/OD: N/A / N/A
SAMPLER: Standard Split-Spoon
HAMMER TYPE: Automatic
HAMMER WEIGHT (lbs): 140
CASING ID/OD: 4 in / 4 1/2 in
CORE BARREL: NQ2
HAMMER EFFICIENCY FACTOR: 0.91
HAMMER DROP (inch): 30
WATER LEVEL DEPTHS (ft): ⊻ 8 ft Water observed at 8.0 feet on 12/5/2022
GENERAL NOTES:

KEY TO NOTES AND SYMBOLS:
Water Level
⊻ At time of Drilling
▼ At Completion of Drilling
▼ After Drilling

D = Split Spoon Sample
U = Thin Walled Tube Sample
R = Rock Core Sample
V = Field Vane Shear

Pen. = Penetration Length
Rec. = Recovery Length
bpf = Blows per Foot
mpf = Minute per Foot

WOR = Weight of Rods
WOH = Weight of Hammer
RQD = Rock Quality Designation
PID = Photoionization Detector

S_v = Field Vane Shear Strength, kips/sq.ft.
q_U = Unconfined Compressive Strength, kips/sq.ft.
Ø = Friction Angle (Estimated)
N/A = Not Applicable

Elev. (ft)	Depth (ft)	Casing Pen. (bpf)	Sample No.	Type	Depth (ft)	Pen./ Rec. (in)	Blow Count or RQD	Field / Lab Test Data / PID Readings	Graphic Log	Sample Description & Classification	H_2O Depth	Remarks
			1D	D	0-2	24/8	6-6-7-7	PID=0 ppm		Grassed Topsoil		
485			2D	D	2-4	24/2	10-8-6-7	PID=0 ppm	1.5	Medium dense, brown, Gravelly Silty SAND (Fill)		
	5		3D	D	5-7	24/13	20-12-13-17	PID=.7 ppm	5.0	Medium dense, brown Silty fine to medium SAND some gravel (Fill)		
480			4D	D	7-9	24/13	27-11-15-13	PID=.3 ppm	7.0	Medium dense, brown, Silty Gravelly SAND	⊻	
	10		5D	D	10-12	24/13	17-13-19-20	PID=.8 ppm	10.0	Dense, brown, Gravelly SAND some silt		
475			6D	D	12-14	24/16	14-33-21-16	PID=.1 ppm ID 22366s w =18.1 %	12.0	Very dense, brown, SAND trace gravel trace silt		
	15		7D	D	15-17	24/1	20-15-9-9		15.0	Cobbles		
470									17.0	Very dense, gray, Silty Gravelly fine to medium SAND (Glacial Till)		
	20		8D	D	20-21	12/2	11-7-50/0"					
465	25		1R	R	21.1-26.1	60/58	62		21.0	Bedrock: Hard, slightly weathered, moderately to slightly fractured, fine to medium grained, gray to black, Granodiorite. Joints are very close to close, low to high angle. RQD = 62%		

Bottom of Exploration at 26.1 feet
Bedrock

Stratification lines represent approximate boundary between soil types, transitions may be gradual. Water level readings have been made at times and under conditions stated. Fluctuations of groundwater may occur due to other factors than those present at the time measurements were made.

BORING / WELL 10-12-2022 21-0414.GPJ SWCE TEMPLATE.GDT 12/23/22

BORING LOG

CLIENT: McFarland Johnson, Inc.
PROJECT: Proposed Bridge Replacement
LOCATION: George Street, Keene, New Hampshire

BORING NO.: ENV-1
SHEET: 1 of 1
PROJECT NO. 21-0414
DATE START: 12/5/2022
DATE FINISH: 12/5/2022

Drilling Information

LOCATION:	See Exploration Location Plan	**ELEVATION (FT):**	488' +/-	**TOTAL DEPTH (FT):**	15.0	**LOGGED BY:**	Sean Hlywa
DRILLING CO.:	S. W. Cole Explorations, LLC	**DRILLER:**	Matt Bussey	**DRILLING METHOD:**	Hollow Stem Auger		
RIG TYPE:	Track Mounted Mobile Drill B-48	**AUGER ID/OD:**	2 1/4 in / 5 5/8 in	**SAMPLER:**	Standard Split-Spoon		
HAMMER TYPE:	Automatic	**HAMMER WEIGHT (lbs):**	140	**CASING ID/OD:**	N/A /N/A	**CORE BARREL:**	N/A
HAMMER EFFICIENCY FACTOR:	0.91	**HAMMER DROP (inch):**	30				

WATER LEVEL DEPTHS (ft): ⊻ 9 ft Water observed at 9.0 feet on 12/5/2022

GENERAL NOTES:

KEY TO NOTES AND SYMBOLS:

Water Level
⊻ At time of Drilling
▼ At Completion of Drilling
▼ After Drilling

D = Split Spoon Sample
U = Thin Walled Tube Sample
R = Rock Core Sample
V = Field Vane Shear

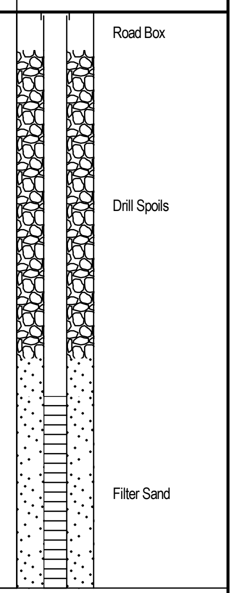
Pen. = Penetration Length
Rec. = Recovery Length
bpf = Blows per Foot
mpf = Minute per Foot

WOR = Weight of Rods
WOH = Weight of Hammer
RQD = Rock Quality Designation
PID = Photoionization Detector

S_v = Field Vane Shear Strength, kips/sq.ft.
q_U = Unconfined Compressive Strength, kips/sq.ft.
Ø = Friction Angle (Estimated)
N/A = Not Applicable

Elev. (ft)	Depth (ft)	Casing Pen. (bpf)	Sample No.	Type	Depth (ft)	Pen./ Rec. (in)	Blow Count or RQD	Field / Lab Test Data / PID Readings	Graphic Log	Sample Description & Classification	H_2O Depth
			1D		0-2	24/8	2-3-2-2	PID=0 ppm		Grassed Topsoil	
			2D		2-4	24/6	2-5-3-3	PID=0 ppm	1.5	Loose, brown, Silty fine SAND (Fill)	
485											
	5		3D		5-7	24/12	9-12-19-19	PID=.5 ppm	5.0	Medium dense to dense, brown, Gravelly fine to medium SAND some silt (Fill)	
			4D		7-9	24/6	12-10-14-8	PID=.2 ppm			
480									9.0	Loose, brown, SAND some gravel some silt	⊻
	10		5D		10-12	24/24	3-4-4-2	PID=.1 ppm			
			6D		12-14	24/17	3-3-2/12"	PID=0 ppm	11.7	Very loose, gray, fine Sandy SILT	
									12.0	Loose, brown, Silty SAND	
475									13.0	Very loose, gray, fine Sandy SILT	
	15										

Well Diagram



Bottom of Exploration at 15.0 feet

Stratification lines represent approximate boundary between soil types, transitions may be gradual. Water level readings have been made at times and under conditions stated. Fluctuations of groundwater may occur due to other factors than those present at the time measurements were made.

BORING NO.: ENV-1

BORING / WELL 10-12-2022 21-0414.GPJ SWCE TEMPLATE.GDT 12/23/22

APPENDIX D

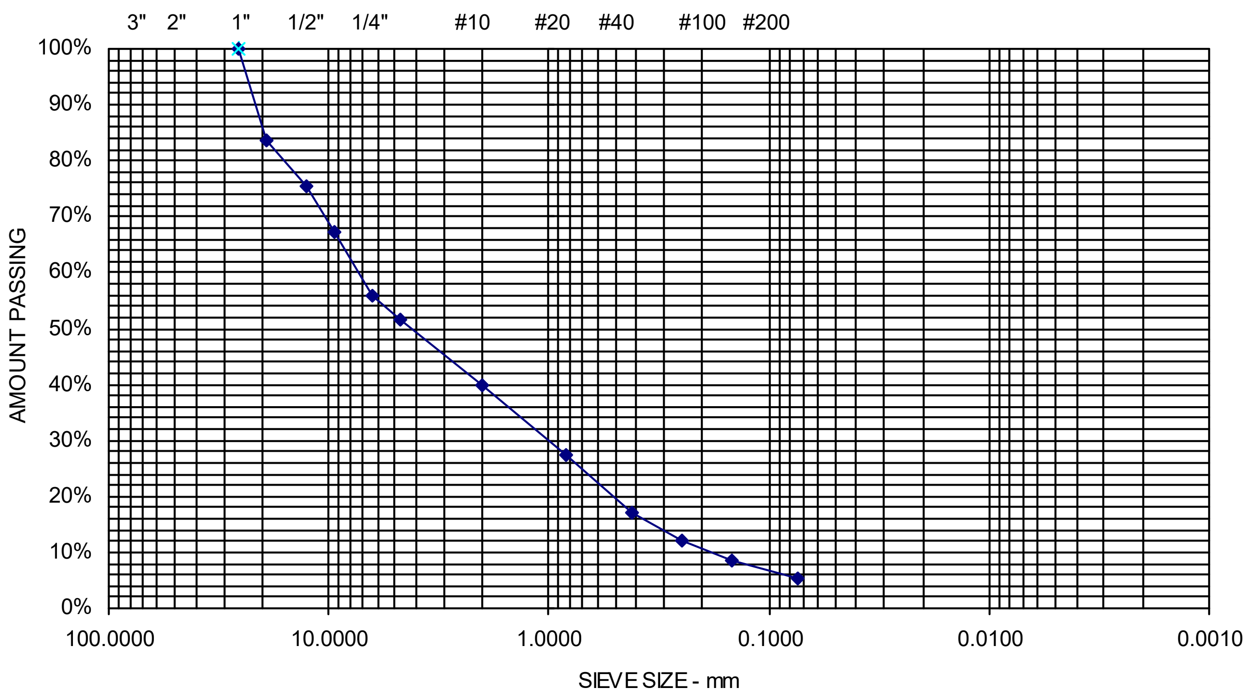
Laboratory Test Results

Report of Gradation

ASTM C-117 & C-136

Project Name	KEENE NH - GEORGE STREET OVER BEAVER BROOK BRIDGE REPLACEMENT - GEOTECHNICAL ENGINEERING SERVICES	Project Number	21-0414
Client	MCFARLAND JOHNSON INC.	Lab ID	22365S
		Date Received	12/13/2022
		Date Completed	12/19/2022
Material Source	**B-1, 6D, 12.0'-14.0'**	Tested By	BRADLEY GERSCHWILER

STANDARD DESIGNATION (mm/µm)	SIEVE SIZE	AMOUNT PASSING (%)	
25.0 mm	**1"**	**100**	
19.0 mm	**3/4"**	**84**	
12.5 mm	**1/2"**	**75**	
9.5 mm	**3/8"**	**67**	
6.3 mm	**1/4"**	**56**	
4.75 mm	**No. 4**	**52**	48.4% Gravel
2.00 mm	**No. 10**	**40**	
850 um	**No. 20**	**27**	
425 um	**No. 40**	**17**	46.4% Sand
250 um	**No. 60**	**12**	
150 um	**No. 100**	**8**	
75 um	**No. 200**	**5.2**	5.2% Fines



Comments: Moisture Content = 12.3%

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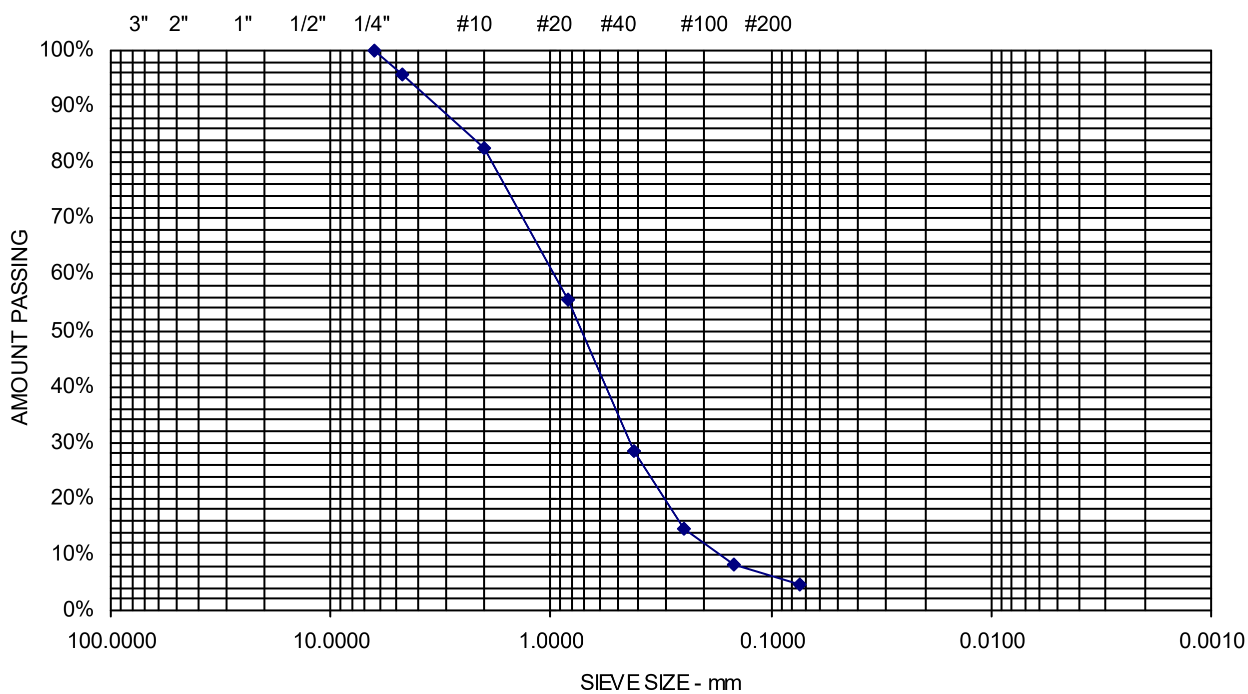
Report of Gradation

ASTM C-117 & C-136

Project Name	KEENE NH - GEORGE STREET OVER BEAVER BROOK BRIDGE REPLACEMENT - GEOTECHNICAL ENGINEERING SERVICES	Project Number	21-0414
Client	MCFARLAND JOHNSON INC.	Lab ID	22366S
		Date Received	12/13/2022
		Date Completed	12/19/2022
Material Source	**B-2, 6D, 12.0'-14.0'**	Tested By	BRADLEY GERSCHWILER

STANDARD DESIGNATION (mm/µm)	SIEVE SIZE	AMOUNT PASSING (%)	
6.3 mm	**1/4"**	**100**	
4.75 mm	**No. 4**	**96**	4.4% Gravel
2.00 mm	**No. 10**	**83**	
850 um	**No. 20**	**55**	
425 um	**No. 40**	**29**	91% Sand
250 um	**No. 60**	**14**	
150 um	**No. 100**	**8**	
75 um	**No. 200**	**4.6**	4.6% Fines



Comments: Moisture Content = 18.1%

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