REPORT

23-1036

December 12, 2023

Explorations and Geotechnical Engineering Services

Mill Bridge Replacement
Lynch Road over Dyer Creek
Newcastle, Maine

Prepared For:
VHB, Inc.
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157 Capital Street, Suite 2
Augusta, ME 04330

Subject: Explorations and Geotechnical Engineering Services
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Dear Bob:

In accordance with our Proposal, dated June 6, 2023, we have performed subsurface explorations for the subject project. The purpose of our services was to obtain subsurface information at the site in order to develop geotechnical recommendations relative to foundations and earthwork associated with the proposed bridge replacement. 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 Site Construction

The site is Mill Bridge carrying Lynch Road over Dyer Creek in Newcastle, Maine. The site location is shown on the "Site Location Map" attached in Appendix B. Based on available information, we understand the existing crossing was constructed in 2010 and consists of ±14-foot span, concrete box culvert. We understand the existing structure was constructed prior to the dam removal and release of Sherman Lake. Since the removal of the dam, we understand the existing channel has reportedly scoured up to 3 feet and the structure has reportedly settled 3 to 4 inches.

1.2 Proposed Construction

We understand current conceptual design consists of replacing the existing structure with a ±50-foot, single-span bridge. We understand bridge support options include H-piles, micropiles, and spread footings. We understand spread footings would be founded on a mud mat overlying bedrock at the north abutment and glacial till at the south abutment. We understand the replacement structure will generally maintain the existing horizontal alignment and vertical profile.

Proposed and existing site features are shown on the “Exploration Location Plan” attached in Appendix B.

2.0 EXPLORATION AND TESTING

2.1 Explorations

Six test borings (B-101 through B-104, B-102A, B-102B, and B-104A) were made at the site on July 31 and August 1, 2023, by S. W. Cole Explorations, LLC. The exploration locations were selected in consultation with VHB and established in the field by S. W. Cole Engineering, Inc. (S.W.COLE) using measurements from existing site features. The approximate exploration locations are shown on the “Exploration Location Plan” attached in Appendix B. Logs of the explorations and a key to the notes and symbols used on the logs are attached in Appendix C. The elevations shown on the logs were estimated based on topographic information shown on the “Exploration Location Plan”.

2.2 Field Testing

The test borings were drilled using a combination of solid stem auger, cased wash-boring, and NQ2 rock coring drilling techniques. The soils were sampled at ±5-foot intervals using a split-spoon sampler and Standard Penetration Testing (SPT) methods using a calibrated automatic hammer. Pocket Penetrometer Tests (PPT) were performed where stiffer cohesive soils were encountered. Upon encountering refusal, borings B-101 and B-104A were advanced 5 to 10 feet into bedrock using NQ2 rock coring. SPT blow counts, PPT results, rock core intervals, and Rock Quality Designation (RQD) are shown on the logs attached in Appendix C.

2.3 Laboratory Testing

Soil and rock core samples obtained from the explorations were returned to our laboratory for further classification and testing. Laboratory testing included five moisture content, two Atterberg limits, one organic content, and three grain size analyses tests. Moisture content, Atterberg Limits, and organic content tests results are noted on the logs. The results of three gradation tests are attached in Appendix D.

3.0 SUBSURFACE CONDITIONS

3.1 Soil and Bedrock

Test borings were made for the proposed replacement structure and encountered a soils profile generally consisting of a surface layer of pavement overlying fill, overlying glaciomarine deposits, overlying glacial till mantling bedrock. The principal strata encountered in the explorations are summarized below. Not all the strata were encountered at each exploration; refer to the attached logs for more detailed subsurface information.

Fill: Below an approximate 3-to-3.5-inch layer of asphalt pavement, the borings generally encountered granular fill extending to depths of about 5 to 10.5 feet, where penetrated. The granular fill generally consisted of medium dense to very dense, sand with varying amounts of gravel and silt. Below the granular fill, boring B-104A encountered fine-grained fill consisting of stiff to very stiff, silt and sand with varying amounts of clay and gravel to a depth of 10.6 feet. Test borings B-102, B-102A, B-102B, and B-104 were terminated in the fill on probable cobbles or boulders at depths of about 3.5 to 10 feet.

Glaciomarine Deposit: Below the fill, where penetrated, borings B-101, B-103, and B-104A encountered glaciomarine deposits to depths of about 14 to 17 feet. The glaciomarine deposit generally consisted of soft to very stiff, silty clay, trace sand with wood. The wood was interpreted as embedded timbers/logs.

Glacial Till: Below the glaciomarine deposits, the borings encountered glacial till consisting of dense, silty gravelly sand with cobbles. The glacial till varied in thickness from about 1 to 7 feet.

Bedrock: Bedrock was encountered at boring B-103 and sampled at borings B-101 and B-104A. The top of bedrock varied from about 15.5 to 24 feet below ground surface and generally consisted of grey, hard, Granofels of the Bucksport Formation. Joints were generally close, horizontal to moderately dipping, and tight to open.

The following table summarizes the approximate depths to bedrock, corresponding top of bedrock elevations, and RQD, where encountered.

Boring Number	Approximate Depth to Refusal/Bedrock (feet)	Approximate Refusal/Bedrock Elevation (feet)	RQD (Rock Quality)
B-101	23.8	-10	R2: 85% (Good)
B-103	15.6	-1.6	N/A
B-104A	17.2	-3.2	R1: 55% (Fair) R2: 53% (Fair)

RQD values for the bedrock cores ranged from 53 to 85 percent corresponding to a Rock Quality of fair to good. Detailed descriptions of the rock core and RQD values for each core run are shown on the exploration logs in Appendix C. Rock core photographs are shown in Appendix C.

3.2 Groundwater

The soils encountered at the test borings were damp to moist from the ground surface. Water was encountered at depths of about 8 to 10 feet in borings B-101, B-103, and B-104A. Long term groundwater information is not available. It should be anticipated that groundwater levels will fluctuate seasonally, particularly in response to periods of snowmelt and precipitation, as well as changes in site use and the water levels of Dyer Creek.

4.0 EVALUATION AND RECOMMENDATIONS

S.W.COLE conducted geotechnical engineering evaluations in accordance with 2020 AASHTO LRFD Bridge Design Specifications, 9th Edition (LRFD) and the MaineDOT Bridge Design Guide, 2003 Edition with revisions through June 2018 (MaineDOT BDG).

4.1 Foundation Options and Discussion

The site is generally underlain by medium dense to very dense fills, overlying very stiff to soft glaciomarine silty clay with embedded timbers/logs, overlying a thin (1-to-7-foot-thick) layer of dense glacial till with cobbles mantling bedrock at depths of about 15.5 to 24 feet. Bedrock appears to slope downward from north to south with 6 to 8 feet of elevation change between the proposed abutment locations. We understand, based on the site history a single span bridge is the preferred replacement structure.

Spread footing foundations will require braced excavations for temporary excavation support and water cutoff. We anticipate excavations would need to extend 8 to 15 feet below the elevation of Dyer Creek into the underlying glacial till or to bedrock. Regardless of bearing strata, spread footing foundations would need to be protected from and founded below scour.

Timbers and/or logs were encountered within the glaciomarine silty clay during exploration. We anticipate these obstructions will impede the driving of sheet piles for braced excavations and H-piles for abutment foundations and may require pre-augering or predrilling. Additionally, depending on the lateral loading the thin layer of glacial till soils may not provide sufficient embedment for driven piles to develop fixity without rock sockets.

We understand following preliminary design, spread footings on a mud mat founded on bedrock at the north abutment and on properly prepared glacial till at the south abutment are the preferred foundation support option.

4.2 Foundation Support

Design considerations for spread footings founded on bedrock or glacial till as well as micropile and H-pile supported foundation options are provided in the following sections.

4.2.1 Spread Footing Foundations on Concrete Sub-footing (Seal Pour)

We understand the north abutment and wingwalls will be founded on spread footings bearing on concrete sub-footing overlying bedrock. For spread footings founded on spread footings shall be evaluated for all applicable load combinations specified in LRFD Articles 3.4.1 and 11.5.5 and relevant strength, service, and extreme limit states.

4.2.1.1 Strength and Service Limit State Design

The design of abutments and wingwalls founded on spread footings bearing on concrete seals overlying bedrock shall be designed for all relevant strength and service limit state load combinations per LRFD Article 10.6. Design of spread footings at the strength limit state shall consider bearing resistance, eccentricity, lateral sliding, and reinforced-concrete structural failure.

For spread footings or concrete seals founded on bedrock, the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed 0.45 of the footing dimensions in either direction. The eccentricity corresponding to the resultant of reaction forces shall fall within the middle nine-tenths (9/10) of the base width.

For the service limit state, a resistance factor, ϕ, of 1.0 shall be used to assess spread footing design for settlement, horizontal movement, and bearing resistance. The overall stability of foundations is typically investigated at the Service I Load Combination and a resistance factor, ϕ, of 0.65. Shear failure along adversely oriented joint surfaces in the rock mass below the foundations is not anticipated; therefore, global stability was not evaluated.

4.2.1.2 Extreme Limit State Design

Extreme limit state design checks for abutments and wingwalls shall include bearing resistance, eccentricity (overturning), failure by sliding, and structural failure with respect to extreme event load conditions relating to seismic forces, hydraulic events, and ice. Resistance factors, ϕ, for the extreme limit state shall be taken as 1.0 except for bearing resistance where a resistance factor of 0.8 shall be used. LRFD Figures C11.5.6-1 and C11.5.6-2 illustrate the typical load factors to produce the extreme factored effect for bearing resistance and sliding and eccentricity.

For scour protection of spread footings or concrete seals, construct the spread footings or concrete seals directly on bedrock surfaces cleaned and free of all weathered, loose, and potentially erodible or scourable rock. With these precautions, strength and extreme limit state designs do not need to consider rock scour for the proposed foundations.

4.2.1.3 Bearing Resistance and Eccentricity

Application of permanent and transient load combinations and applicable load factors are specified in LRFD Article 11.5.6. Based on LRFD Figure 11.6.3.2-2, the stress distribution at the abutments may be assumed to be a triangular or trapezoidal distribution over the effective base.

For abutment and wingwall footings founded on competent, sound bedrock, we recommend the following factored bearing resistances.

Limit State	**Bearing Resistance Factor** φ_b	**Factored Bearing Resistance (ksf)**	**LRFD Reference**
Service	1.0	20.0	Article 10.5.5.1
Strength	0.45	37.7	Table 10.5.5.2.2-1
Extreme	0.8	66.9	Article C11.5.8

LRFD Figures C11.5.6-2 and C11.5.6-4 illustrate the typical load factors to produce the strength and extreme factored conditions for evaluating eccentricity. Based on LRFD Article 11.6.3.3, the location of the resultant force for eccentricity evaluation shall fall within the middle nine-tenths (9/10) of the foundation base for foundations bearing on rock.

4.2.1.4 Sliding Resistance

The following table shows the resistance factors, φ_τ, for sliding analyses of cast-in-place spread footings on bedrock.

Limit State	**Sliding Resistance Factor** φ_τ	**Reference**
Strength	0.8	LRFD Table C10.5.5.2.2-1
Service	1.0	LRFD Article 10.5.5.1
Extreme	1.0	LRFD Article 10.5.5.3.3

Passive earth pressures due to the presence of soils in front of the abutments and wingwalls shall be neglected in the sliding analysis.

For bedrock subgrade prepared in-the-dry and cleaned with high pressure water and air prior to placing footing concrete, sliding computations for resistance of abutment and wingwall footings to lateral loads shall assume a maximum frictional coefficient of 0.7 at the bedrock-concrete seal interface.

Based on guidance from LRFD Article 10.6.1.5, anchorage of the footing to a concrete seal, if used, is required. The dowels should be drilled and grouted into the concrete seal after dewatering and prior to placing the footing concrete. Anchorage of concrete seals to bedrock may also be required to resist sliding forces and improve stability. If bedrock is observed to slope steeper than 4H:1V at the subgrade elevation, the bedrock should be benched to create level steps or excavated to be completely level.

4.2.1.5 Subgrade Preparation and Construction Considerations

Excavation to the bedrock surface in preparation of a placement of a concrete sub-footing (concrete seal pour) will extend about 7 feet below the highwater level of Dyer Creek. Excavation will require an internally braced support of excavation (SOE) likely consisting of driven sheet piles. Given the slightly sloping bedrock surface, a tight connection between the bottom of the sheet piles and the bedrock surface is not possible, creating a “stair step” effect at the base of the sheets where soil can be transported hydraulically from behind the base of the sheets into the excavation. Care must be taken to reduce the potential for migration of soil into the excavation. Further, sheet piles are unlikely to penetrate the bedrock, We recommend the excavation be undertaken in-the-dry by pumping the groundwater from within the SOE. The bedrock subgrade must be cleaned with high pressure water and air prior to placing footing.

4.2.2 Spread Footings on Soil Subgrades

We understand the south abutment and wingwalls will be founded on spread footings bearing on glacial till. Spread footings bearing on properly prepared soil subgrades shall be evaluated for all applicable load combinations specified in LRFD Articles 3.4.1 and 11.5.5 and designed for all relevant strength, service, and extreme limit states.

4.2.2.1 Strength and Service Limit State Design

The design spread footings bearing on properly prepared glacial till subgrades shall be designed for all relevant strength and service limit state load combinations per LRFD Article 10.6. Design of spread footings at the strength limit state shall consider bearing resistance, eccentricity, lateral sliding, and reinforced-concrete structural failure.

For spread footings founded on properly prepared glacial till subgrades, the eccentricity of loading at the strength limit state, based on factored loads, shall not exceed 0.45 of the footing dimensions in either direction. The eccentricity corresponding to the resultant of reaction forces shall fall within the middle two-thirds (2/3) of the base width for foundations on soil per LRFD Article 11.6.3.3.

For the service limit state, a resistance factor, ϕ, of 1.0 shall be used to assess spread footing design for settlement, horizontal movement and bearing resistance.

4.2.2.2 Extreme Limit State Design

Extreme limit state design checks for abutments shall include bearing resistance, eccentricity (overturning), failure by sliding and structural failure with respect to extreme event load conditions relating to seismic forces, hydraulic events, and ice. Resistance factors, ϕ, for the extreme limit state shall be taken as 1.0 except for bearing resistance for which a resistance factor of 0.8 shall be used. LRFD Figures C11.5.6-1 and C11.5.6-2 illustrate the typical load factors to produce the extreme factored effect for bearing resistance and sliding and eccentricity.

For scour protection, the spread footings on properly prepared soil subgrade shall be protected from scour with riprap on the riverbank slope and placed below the depth of scour.

4.2.2.3 Bearing Resistance and Eccentricity

Application of permanent and transient load combinations and applicable load factors are specified in LRFD Article 11.5.6. Based on LRFD Figure 11.6.3.2-2, the stress distribution at the abutments may be assumed to be a triangular or trapezoidal distribution over the effective base.

For abutment footings founded on prepared soil subgrades, we recommend the following factored bearing resistances.

Limit State	Bearing Resistance Factor φ_b	Footing Width (feet)	Factored Bearing Resistance (ksf)
Service	1.0	5	6.0
		6	6.0
		7	6.0
Strength	0.45	5	6.3
		6	7.0
		7	7.7

LRFD Figures C11.5.6-2 and C11.5.6-4 illustrate the typical load factors to produce the strength and extreme factored conditions for evaluating eccentricity. Based on LRFD Article 11.6.3.3, the location of the resultant force for eccentricity evaluation shall fall within the middle two-thirds (2/3) of the foundation base for foundations bearing on soil.

In no instance shall the factored bearing stress exceed the factored compressive resistance of the footing concrete, which may be taken as 0.3f'c. No footing shall be less than 2 feet wide regardless of the applied bearing pressure or bearing material.

4.2.2.4 Sliding Resistance

The following table shows the resistance factors, φ_τ, for sliding analyses of cast-in-place spread footings bearing on 12 inches of compacted NHDOT Section 703 #467 Stone.

Limit State	Sliding Resistance Factor φ_τ	Reference
Strength	0.8	LRFD Article C10.5.5.2.2
Service	1.0	LRFD Article 10.5.5.1
Extreme	1.0	LRFD Article 10.5.5.1

Passive earth pressures due to the presence of soils in front of the abutments, wingwalls, and pier shall be neglected in the sliding analysis.

4.2.2.5 Subgrade Preparation and Construction Considerations

The top of the glacial till was encountered at about elevation -3 feet at B-101, approximately at the proposed footing elevation. In the event glacial till is not present at abutment footing subgrade, the soils should be over-excavated until glacial till is encountered and backfilled with compacted crushed stone or lean concrete. Excavation to the proposed bearing surface will extend about 8 feet below the OHW level. Excavation will require an internally braced SOE, likely consisting of driven sheet piles. The groundwater should be lowered to a level at least 1 foot below proposed foundation subgrade elevation. Excavation to final grade should be accomplished with the use of a smooth edge excavator bucket. The abutment footing should be underlain by at least 12 inches of compacted MaineDOT ¾-inch Crushed Stone.

4.2.3 Micropiles

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 competent bedrock. 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.

4.2.3.1 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 30 ksf for the Granofels 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 tension load tests are performed.

Micropile axial resistance is dependent on rock-socket diameter and length. We recommend a minimum 10-foot rock socket considering fair to good rock quality.

A summary of estimated factored strength-limit axial geotechnical resistances for 7.5 and 9.5-inch diameter micropiles with various rock-socket lengths is provided in the following table. Additional pile diameters can be evaluated depending on foundation loading.

Embedment in Competent Bedrock (ft)	Strength Limit Factored Axial Resistance (kips)[1]	
	7.5-inch Uncased Diameter[2]	9.5-inch Uncased Diameter[3]
10	415	525
15	620	790
20	830	1,050

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 rock-socket diameters and 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 verification load test results and selected means and methods.

4.2.3.2 Downdrag

We understand the approach grades will remain within ½ foot of existing grades therefore, settlement will be negligible. Therefore, downdrag is not considered to be an issue.

4.2.3.3 Lateral Resistance

We anticipate micropiles will be subjected to lateral loading; the micropiles should 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.

4.2.4 H-Piles

Based on the subsurface findings and relatively shallow depth to bedrock, we anticipate piles will need to be drilled and socketed into bedrock to prevent translation of the pile tip. In accordance with LRFD Article 10.7.1.2, center-to-center pile spacing should not be less than 30 inches or 2.5 pile diameters, whichever is greater.

4.2.4.1 Strength Limit State Design

Design of pile foundations bearing within bedrock at the strength limit state shall consider.

- Compressive axial geotechnical resistance of individual piles.
- Structural resistance of individual piles in axial compression.
- Structural resistance of individual piles in combined axial loading and flexure.

Pile groups should be designed to resist all lateral earth loads, vehicular loads, dead and live loads, and lateral forces transferred through the abutments. The pile group resistance after scour due to the design flood shall provide adequate foundation resistance using the resistance factors given in this section.

We anticipate H-piles will be subjected to lateral loading; therefore, the piles should be evaluated for resistance against combined axial compression and flexure in accordance with LRFD Articles 6.9.2.2 and 6.15.2.

Per LRFD Article 6.5.4.2, at the strength limit state, the axial resistance factor ϕ_c = 0.70 and the flexural resistance factor ϕ_f = 1.0 shall be applied to the combined axial and flexural resistance of the pile in the interaction equation (LRFD Eq. 6.9.2.2-1 or -2).

Structural Resistance. The nominal axial compressive structural resistance (P_n) for piles loaded in compression shall be as specified in LRFD Article 6.9.4.1. The nominal axial structural compressive resistance (P_n) subject to the combined axial compression and flexure shall be evaluated based on unbraced lengths (l) and effective length factors (K) as determined from LPile once structural loads are available. The nominal axial structural resistance should be evaluated based on combined axial compression and flexure.

Preliminary estimates of the structural axial resistance for selected H-pile sections were calculated using a resistance factor, ϕ_c = 0.60, for good driving conditions. The unbraced pile lengths (l) and effective length factors (K) in these evaluations have been assumed. It is the responsibility of the structural engineer to calculate the nominal axial structural compressive resistance (P_n) based on unbraced lengths (l) and effective length factors (K) determined from LPile as needed.

Geotechnical Resistance. The nominal axial geotechnical resistance in the strength limit state were calculated using the guidance in LRFD Article 10.7.3.2.3 which states the nominal bearing resistance of piles driven to point bearing on hard rock shall not exceed the structural pile resistances obtained from LRFD Article 6.9.4.1 with a resistance factor ϕ_c = 0.60 for good driving conditions.

Drivability Analyses. We anticipate piles, if selected, for foundation support will be placed in predrilled holes and grouted into bedrock therefore drivability analyses were not evaluated.

A summary of the calculated factored axial compressive structural, geotechnical, and drivability resistances of selected H-piles for the strength limit states are provided in the following table.

Factored Axial Pile Resistances at Strength Limit States				
Pile Section	**Factored Axial Pile Resistance (kips)**			
	Structural Resistance $\phi_c = 0.5$	**Geotechnical Resistance $\phi = 0.5$**	**Drivability Resistance**	**Controlling Axial Pile Resistance**
HP 12x74	545	545	N/A	545
HP 14x89	650	650	N/A	650

Notes: 1. End bearing on bedrock
2. The drivability resistance in not applicable (N/A) since piles will be pre-drilled and socketed into bedrock.

Additional pile sections may be considered depending on the factored design axial loads. S.W.COLE can provide additional input on pile size once pile loading has been developed for the proposed structure.

4.2.4.2 Service and Extreme Limit State Design

The design of H-piles 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 $\varphi = 1.0$ 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 $\varphi = 1.0$ 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.

The nominal axial geotechnical pile resistance at the service and extreme limit state was calculated using the guidance in LRFD Article 10.7.3.2.3. A summary of the calculated factored axial structural, geotechnical, and drivability resistances of selected H-piles for the extreme and service limit states are provided in the following table.

Factored Axial Pile Resistances at Service and Extreme Limit States				
Pile Section	**Factored Axial Pile Resistance (kips)**			
	Structural Resistance $\phi_c = 1.0$	**Geotechnical Resistance $\phi = 1.0$**	**Drivability Resistance**	**Controlling Axial Pile Resistance**
HP 12x74	1,090	1,090	N/A	1,090
HP 14x89	1,305	1,305	N/A	1,305

Notes: 1. End bearing on bedrock
2. The drivability resistance in not applicable (N/A) since piles will be pre-drilled and socketed into bedrock.

4.2.4.3 Downdrag

We understand the approach grades will remain within ½ foot of existing grades therefore, settlement will be negligible. Therefore, downdrag is not considered to be an issue.

4.2.4.4 Lateral Pile Resistance

In accordance with LRFD Article 6.15.1, the structural analysis of pile groups subjected to lateral loads shall include consideration of soil-structure interaction effects as specified in LRFD Article 10.7.3.9. Assumptions regarding a fixed or pinned condition at the pile tip should be also confirmed with soil-structure interaction analyses.

S.W.COLE is available to perform lateral pile analyses using LPile® 2016 (LPile) software with axial and lateral loads to be supplied by the structural engineer.

4.3 Abutment Design Considerations

Abutments should be designed for all relevant strength, service, and extreme limit states and load combinations specified in LRFD Articles 3.4.1 and 11.5.5. Stub abutments shall be designed to resist all lateral earth loads, vehicular loads, dead and live loads, and lateral forces transferred through the superstructure. Strength limit state design shall also consider changes in foundation conditions and foundation resistance after scour due to the design (Q_{100}) flood.

A resistance factor (φ) of 1.0 shall be used to assess abutment design at the service limit state, including: settlement, excessive horizontal movement, and movement resulting after scour due to the design (Q_{100}) flood. The overall stability of the foundation should be investigated at the Service I Load Combination and a resistance factor, φ, of 0.65.

Extreme limit state design of integral abutment supported on H-piles or micropiles shall include pile structural resistance, pile geotechnical resistance, pile resistance in combined axial and flexure, and overall stability. Resistance factors for extreme limit state shall be taken as 1.0. Extreme limit state design shall also check that the nominal foundation resistance remaining after scour due to the check (Q_{500}) flood can support the extreme limit state loads with a resistance factor of 1.0.

The designer may assume the following abutment backfill material soil properties in accordance with MaineDOT Bridge Design Guide (BDG) Section 3.6.1.

- Angle of internal friction (ϕ) of 32 degrees,
- Total unit weight (γ) of 125 pcf, and
- Soil-concrete interface friction angle (δ) of 20 degrees.

Abutments shall be designed to withstand a lateral earth load equal to the passive pressure state. AASHTO LRFD Article C3.11.5.4 suggests full passive pressure is mobilized when the ratio of lateral abutment movement to abutment height (y/H) is 0.05H in loose cohesionless soils and less than 0.05H in dense cohesionless soils. Additionally, Federal Highway Authority (FHWA) NHI-06-089 Figure 10-4, indicates mobilization of full passive pressure in dense cohesionless soils occurs at a y/H ratio of 0.02H.

Considering the above information, we recommend the structural designer estimate the abutment rotation. S.W.COLE can then assist in selection of a passive pressure coefficient.

The abutment design shall include a drainage system behind the abutment to mitigate excessive hydrostatic pressures. Drainage behind the structure shall be in accordance with MaineDOT BDG Section 5.4.2.13.

Backfill within 10 feet of the abutments and side slope fill shall conform to MaineDOT Specification 703.19 “Granular Borrow for Underwater Backfill.”

Slopes in front of the pile supported abutments should be protected with riprap and erosion control geotextile. The riprap covered slopes should not exceed 1.75:1(H:V) and be “toe-in” at least 2 feet.

4.4 Seismic Considerations

Seismic site class was evaluated in accordance with LRFD Article 3.10.3.1 Method B (average N-value method). AASHTO allows for an N-value of 100 to be used for bedrock in the upper 100 feet of the soil profile. Based on the subsurface information and an N-value of 100 for the bedrock, the average N-value was between 15 and 50 bpf corresponding to an AASHTO Site Class D as defined in LRFD Table 3.10.3.1-1.

The United States Geological Survey (USGS) Seismic Design Parameters program (Version 2.1) was used to obtain the seismic design parameters for the site. Based on the assigned site class (AASHTO Site Class D) and site coordinates, the software provides the recommended AASHTO Response Spectrum for a 7 percent probability of exceedance in 75 years (1,000-year return period).

RECOMMENDED SEISMIC DESIGN PARAMETERS[1]	
Site Class	D
PGA	0.069 g
S_s	0.146 g
S_1	0.042 g
F_{pga}	1.6
F_a	1.6
F_v	2.4
A_s	0.11 g
S_{DS}	0.23 g
S_{D1}	0.10 g
Seismic Zone (based on S_{D1})	Zone 1

NOTE: 1. Site Coordinates: N44.003652, W69.588012
2. Seismic Zone from AASHTO LRFD Table 3.10.6-1

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.

4.5 Frost Considerations

Pile supported integral abutments should be embedded a minimum of 4.0 feet for frost protection. Foundations bearing on soil should be designed with an appropriate embedment for frost protection. The design freezing index for the Newcastle, Maine area is approximately 1,300 freezing degree-days. Based on the MaineDOT BDG, Section 5.2.1 and Table 5-1 and subsurface soils encountered, the maximum seasonal frost penetration is estimated to be on the order of about 4.5 feet; consequently, we recommend foundations should have at least 4.5 feet of soil cover to provide frost protection.

Riprap is not to be considered as contributing to the overall thickness of soils required for frost protection.

4.6 Construction Considerations

Construction of the abutments will require installation of drilled micropiles or H-piles placed in drilled rock sockets. The new abutments will be constructed behind the existing crossing. We understand current planning is considering full road closure. Construction shall avoid disturbance of the sensitive soils outside the excavation limits and avoiding placement of fills in the river.

The contractor should monitor the stability of slopes, excavation, soils at the roadway grade and the temporary earth retaining systems during construction. The contractor should control groundwater, surface water infiltration and soil erosion. Water should be controlled by pumping from sumps.

4.7 Design Review and Construction Testing

S.W.COLE should be retained to review the construction documents prior to bidding to determine that our earthwork and foundation recommendations have been properly interpreted and implemented.

A construction material testing and quality assurance program should be implemented during construction to observe compliance with the design concepts, plans, and specifications. S.W.COLE is available to observe earthwork activities, installation of piles as well as to provide testing for soils, concrete, and asphalt construction materials.

5.0 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.

2023.12.12
18:30:33 -05'00'

STATE OF MAINE
MICHAEL A. ST. PIERRE
No. 12519
LICENSED
PROFESSIONAL ENGINEER

Michael A. St. Pierre, P.E.
Senior Geotechnical Engineer

MAS:tsd-rec

APPENDIX A

Limitations

This report has been prepared for the exclusive use of VHB, Inc. for specific application to the proposed Mill Bridge Replacement carrying Lynch Road over Dyer Creek in Newcastle, Maine. 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.

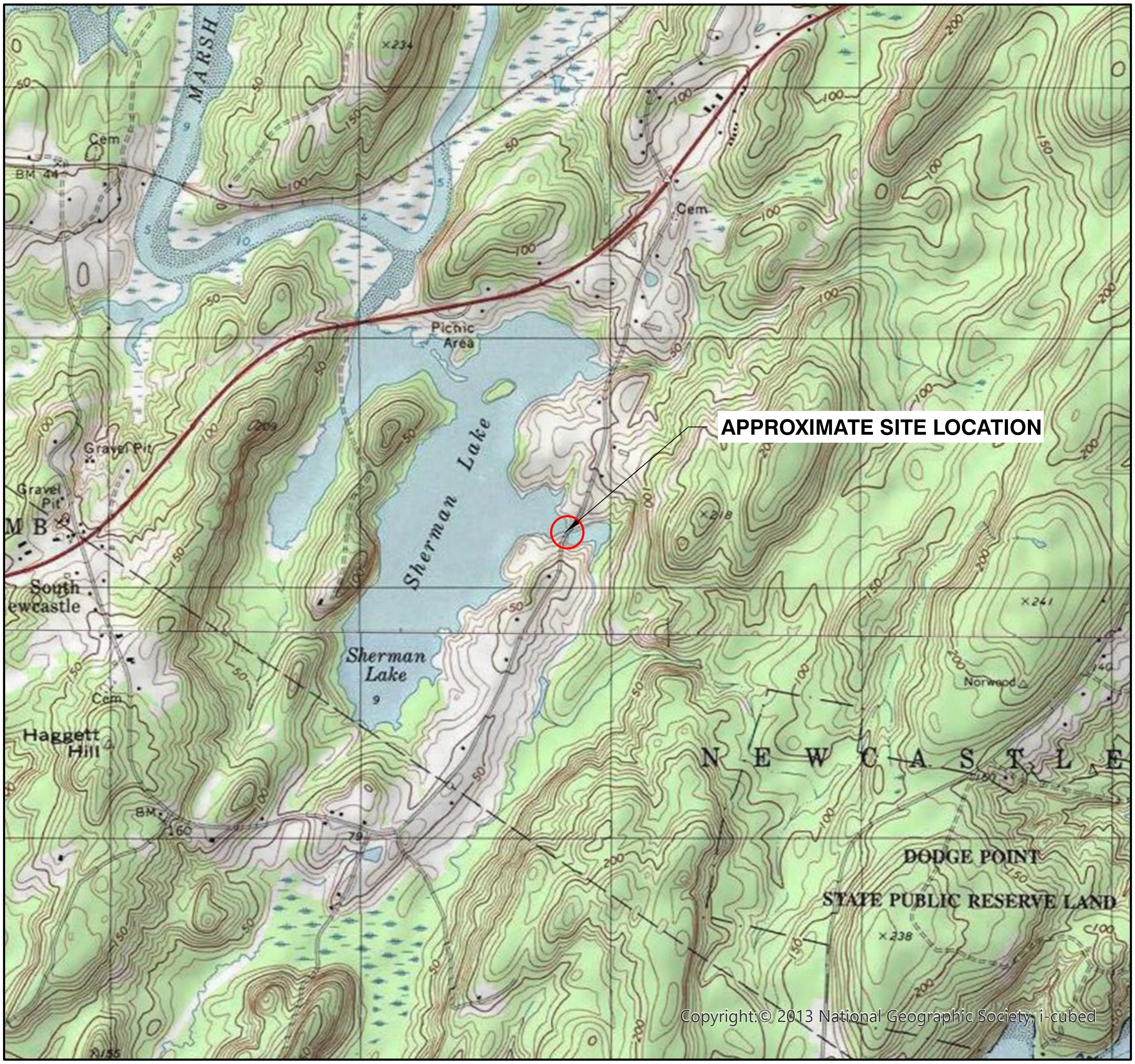
Observations have been made during exploration work to assess site groundwater levels. Fluctuations in water levels will occur due to variations in rainfall, temperature, and other factors.

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



2,000 0 2,000 4,000

Scale in Feet

S.W.COLE ENGINEERING, INC.

VHB, INC.

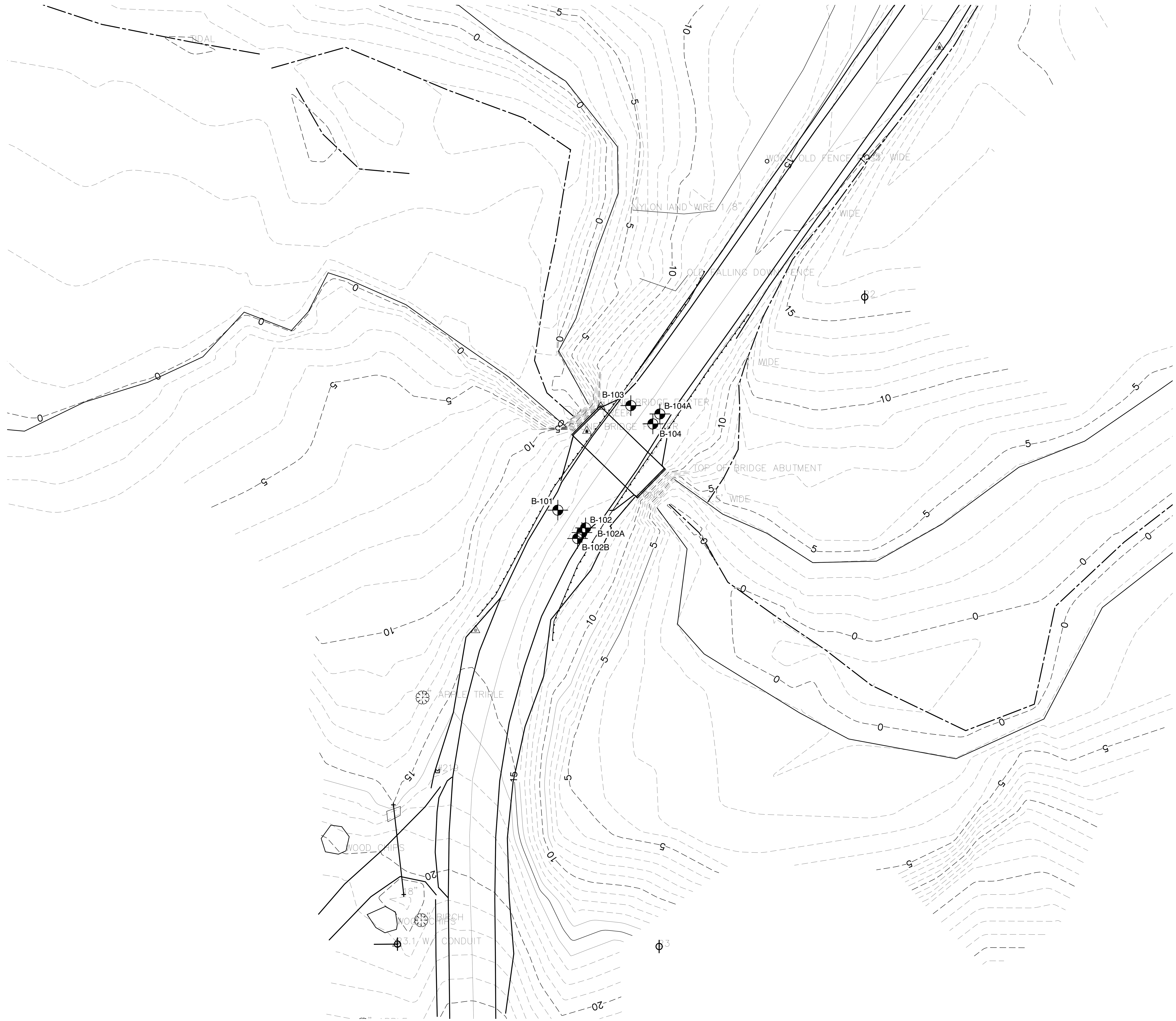
SITE LOCATION MAP

MILL BRIDGE REPLACEMENT
LYNCH ROAD OVER DYER CREEK
NEWCASTLE, MAINE

Job No.	23-1036	Scale	1" = 2000'
Date:	08/14/2023	Sheet	1

NOTE:
SITE LOCATION MAP PREPARED FROM ESRI ArcGIS ONLINE AND DATA PARTNERS INCLUDING USGS AND © 2007 NATIONAL GEOGRAPHIC SOCIETY.

R:\2023\23-1036\ArcProject\23-1036.aprx, 8/14/2023 9:16 AM, 1:24,000,



LEGEND:

APPROXIMATE BORING LOCATION

NOTES:

1. EXPLORATION LOCATION PLAN WAS PREPARED FROM SCALE PLANS OF THE SITE PROVIDED BY VHB, INC., RECEIVED VIA E-MAIL 8/9/2023.
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.

VHB, INC.

EXPLORATION LOCATION PLAN

MILL BRIDGE REPLACEMENT
LYNCH ROAD OVER DYER CREEK
NEWCASTLE, MAINE

Job No.:	23-1036	Scale:	1" = 20'
Date :	09/19/2023	Sheet:	2

APPENDIX C

Exploration Logs and Key

KEY TO NOTES & SYMBOLS
Test Boring and Test Pit Explorations

Stratification lines represent the approximate boundary between soil types and the transition may be gradual.

Key to Symbols Used:

w	-	water content, percent (dry weight basis)
q_u	-	unconfined compressive strength, kips/sq. ft. - laboratory test
S_v	-	field vane shear strength, kips/sq. ft.
L_v	-	lab vane shear strength, kips/sq. ft.
q_p	-	unconfined compressive strength, kips/sq. ft. – pocket penetrometer test
O	-	organic content, percent (dry weight basis)
W_L	-	liquid limit - Atterberg test
W_P	-	plastic limit - Atterberg test
WOH	-	advance by weight of hammer
WOM	-	advance by weight of man
WOR	-	advance by weight of rods
HYD	-	advance by force of hydraulic piston on drill
RQD	-	Rock Quality Designator - an index of the quality of a rock mass.
γ_T	-	total soil weight
γ_B	-	buoyant soil weight

Description of Proportions:

Trace:	0 to 5%
Some:	5 to 12%
"Y"	12 to 35%
And	35+%
With	Undifferentiated

Description of Stratified Soils

Parting:	0 to 1/16" thickness
Seam:	1/16" to 1/2" thickness
Layer:	½" to 12" thickness
Varved:	Alternating seams or layers
Occasional:	one or less per foot of thickness
Frequent:	more than one per foot of thickness

REFUSAL: **Test Boring Explorations** - Refusal depth indicates that depth at which, in the drill foreman's opinion, sufficient resistance to the advance of the casing, auger, probe rod or sampler was encountered to render further advance impossible or impracticable by the procedures and equipment being used.

REFUSAL: **Test Pit Explorations** - Refusal depth indicates that depth at which sufficient resistance to the advance of the backhoe bucket was encountered to render further advance impossible or impracticable by the procedures and equipment being used.

Although refusal may indicate the encountering of the bedrock surface, it may indicate the striking of large cobbles, boulders, very dense or cemented soil, or other buried natural or man-made objects or it may indicate the encountering of a harder zone after penetrating a considerable depth through a weathered or disintegrated zone of the bedrock.

BORING LOG

S.W.COLE

CLIENT: VHB, Inc.
PROJECT: Mill Bridge Replacement
LOCATION: Lynch Road over Dyer Creek, Newcastle, Maine

BORING NO.: B-101
SHEET: 1 of 2
PROJECT NO. 23-1036
DATE START: 8/1/2023
DATE FINISH: 8/1/2023

Drilling Information

LOCATION: See Exploration Location Plan
ELEVATION (FT): 13.8' +/-
TOTAL DEPTH (FT): 28.8
LOGGED BY: Michael St. Pierre
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 / 2
HAMMER CORRECTION FACTOR: 1.51
HAMMER DROP (inch): 30
WATER LEVEL DEPTHS (ft): ⊻ 7.8 ft after drilling
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	Graphic Log	Sample Description & Classification	H_20 Depth	Remarks
									0.3	3" of Pavement		
			1D	D	0.9-2.9	24/15	18-22-17-29			Very dense, gray, damp, gravelly SAND, some silt, with cement (Fill - Possible Cement Treated Base/Subbase Gravel)		
									1.4	Medium dense, brown, damp, gravelly SAND, some silt, with asphalt fragments (Fill)		
	5		2D	D	5-7	24/22	4-3-3-3	q_P=5.0-5.5 ksf ID 15029A w =20.1 % W_L=24 W_P=18	5.0	Very stiff, gray, moist, silty CLAY, some sand, trace organics (reeds)	⊻	
	10		3D	D	10-12	24/16	5-9-15-17		10.7	Medium stiff to soft, gray, wet, silty CLAY, trace sand, with organics (wood, wood fiber)		
	15		4D	D	15-17	24/0	5-1-1-1					
			5D	D	17-17.2	2/2	50/2"		17.0	Dense, brown, wet, silty gravelly SAND, with cobbles (Glacial Till)		
			1R	R	17.9-22.9	60/29						1R core through cobbles
-10			2R	R	23.8-28.8	60/60	85		23.8	Gray, fine-grained, GRANOFELS, hard, fresh, joints are close to moderately close,		

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.

(Continued Next Page)

BORING NO.: B-101

BORING LOG

CLIENT: VHB, Inc.
PROJECT: Mill Bridge Replacement
LOCATION: Lynch Road over Dyer Creek, Newcastle, Maine

BORING NO.: B-101
SHEET: 2 of 2
PROJECT NO. 23-1036
DATE START: 8/1/2023
DATE FINISH: 8/1/2023

Elev. (ft)	Depth (ft)	Casing Pen. (bpf)	SAMPLE INFORMATION Sample No.	Type	Depth (ft)	Pen./ Rec. (in)	Blow Count or RQD	Field / Lab Test Data	Graphic Log	Sample Description & Classification	H_20 Depth	Remarks
-15										horizontal to moderately dipping, and tight (Bucksport Formation)		

Bottom of Exploration at 28.8 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.: B-101

BORING / WELL 10-12-2022 23-1036.GPJ SWCE TEMPLATE.GDT 12/12/23

BORING LOG

CLIENT:	VHB, Inc.
PROJECT:	Mill Bridge Replacement
LOCATION:	Lynch Road over Dyer Creek, Newcastle, Maine

BORING NO.:	**B-102**
SHEET:	1 of 1
PROJECT NO.	23-1036
DATE START:	8/1/2023
DATE FINISH:	8/1/2023

Drilling Information

LOCATION: See Exploration Location Plan **ELEVATION (FT):** 13.7' +/- **TOTAL DEPTH (FT):** 5.4 **LOGGED BY:** Michael St. Pierre
DRILLING CO.: S. W. Cole Explorations, LLC **DRILLER:** Matt Bussey **DRILLING METHOD:** Solid Stem Auger
RIG TYPE: Track Mounted Mobile Drill B-48 **AUGER ID/OD:** N/A / 4 1/2 in **SAMPLER:** Standard Split-Spoon
HAMMER TYPE: Automatic **HAMMER WEIGHT (lbs):** 140 **CASING ID/OD:** N/A /N/A **CORE BARREL:**
HAMMER CORRECTION FACTOR: 1.51 **HAMMER DROP (inch):** 30
WATER LEVEL DEPTHS (ft): No free water observed
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 INFORMATION: Sample No.	Type	Depth (ft)	Pen./ Rec. (in)	Blow Count or RQD	Field / Lab Test Data	Graphic Log	Sample Description & Classification	H_2O Depth	Remarks
									0.3	3 1/2" of Pavement		
										Dense, gray, damp, gravelly SAND, some silt, with cement (Fill - Possible Cement Treated Base/Subbase Gravel)		
			1D		1.8-3.8	24/18	16-15-15-15	ID 15031A w =5.9 %	1.8	Dense, brown, damp, silty gravelly SAND, with asphalt fragments (Fill)		
10												
	5		2D		5-5.3	3/3	50/3"			Medium dense, gray, moist, SAND and SILT, some fine gravel (Fill)		

Auger Refusal at 5.4 feet
Probable Boulder

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 LOG

S.W.COLE

CLIENT: VHB, Inc.
PROJECT: Mill Bridge Replacement
LOCATION: Lynch Road over Dyer Creek, Newcastle, Maine

BORING NO.: B-102A
SHEET: 1 of 1
PROJECT NO. 23-1036
DATE START: 8/1/2023
DATE FINISH: 8/1/2023

Drilling Information

LOCATION: See Exploration Location Plan **ELEVATION (FT):** 13.7' +/- **TOTAL DEPTH (FT):** 6.2 **LOGGED BY:** Michael St. Pierre
DRILLING CO.: S. W. Cole Explorations, LLC **DRILLER:** Matt Bussey **DRILLING METHOD:** Solid Stem Auger
RIG TYPE: Track Mounted Mobile Drill B-48 **AUGER ID/OD:** N/A / 4 1/2 in **SAMPLER:** Standard Split-Spoon
HAMMER TYPE: Automatic **HAMMER WEIGHT (lbs):** 140 **CASING ID/OD:** N/A /N/A **CORE BARREL:**
HAMMER CORRECTION FACTOR: 1.51 **HAMMER DROP (inch):** 30
WATER LEVEL DEPTHS (ft): No free water observed
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 INFORMATION: Sample No.	Type	Depth (ft)	Pen./ Rec. (in)	Blow Count or RQD	Field / Lab Test Data	Graphic Log	Sample Description & Classification	H_20 Depth	Remarks
										See boring B-102 for summary of subsurface strata from 0 to 5.4 ft.		
10												
	5											
	5.4									Probable Fill		

Auger Refusal at 6.2 feet
Probable Boulder

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.: B-102A

BORING LOG

S.W.COLE

BORING NO.:	B-102B
SHEET:	1 of 1
PROJECT NO.	23-1036
DATE START:	8/1/2023
DATE FINISH:	8/1/2023

CLIENT: VHB, Inc.
PROJECT: Mill Bridge Replacement
LOCATION: Lynch Road over Dyer Creek, Newcastle, Maine

Drilling Information

LOCATION: See Exploration Location Plan **ELEVATION (FT):** 13.8' +/- **TOTAL DEPTH (FT):** 9.9 **LOGGED BY:** Michael St. Pierre
DRILLING CO.: S. W. Cole Explorations, LLC **DRILLER:** Matt Bussey **DRILLING METHOD:** Solid Stem Auger
RIG TYPE: Track Mounted Mobile Drill B-48 **AUGER ID/OD:** N/A / 4 1/2 in **SAMPLER:** Standard Split-Spoon
HAMMER TYPE: Automatic **HAMMER WEIGHT (lbs):** 140 **CASING ID/OD:** N/A /N/A **CORE BARREL:**
HAMMER CORRECTION FACTOR: 1.51 **HAMMER DROP (inch):** 30
WATER LEVEL DEPTHS (ft): No free water observed
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 INFORMATION: Sample No.	Type	Depth (ft)	Pen./ Rec. (in)	Blow Count or RQD	Field / Lab Test Data	Graphic Log	Sample Description & Classification	H_2O Depth	Remarks
										See borings B-102 and B-102A for summary of subsurface strata from 0 to 6.2 ft.		
10	5											
	6.2									Probable Fill		
5												

Auger Refusal at 9.9 feet
Probable Boulder

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 LOG

S.W.COLE

CLIENT: VHB, Inc.
PROJECT: Mill Bridge Replacement
LOCATION: Lynch Road over Dyer Creek, Newcastle, Maine

BORING NO.: B-103
SHEET: 1 of 1
PROJECT NO. 23-1036
DATE START: 7/31/2023
DATE FINISH: 7/31/2023

Drilling Information

LOCATION: See Exploration Location Plan **ELEVATION (FT):** 14' +/- **TOTAL DEPTH (FT):** 15.6 **LOGGED BY:** Michael St. Pierre
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:**
HAMMER CORRECTION FACTOR: 1.51 **HAMMER DROP (inch):** 30
WATER LEVEL DEPTHS (ft): ∇ 10.3 ft during drilling
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	Graphic Log	Strata Depth (ft)	Sample Description & Classification	H_2O Depth	Remarks
										0.3	3" of Pavement		
			1D	D	1-3	24/14	18-23-20-21			1.4	Dense, gray, damp, gravelly SAND, some silt, with cement (Fill - Possible Cement Treated Base/Subbase Gravel)		
											Medium dense, brown, damp, gravelly SAND, some silt (Fill)		
10													
	5		2D	D	5-7	24/16	5-5-4-5	ID 15032A w =5.7 %			Medium dense, brown, moist, silty gravelly SAND, with asphalt fragments (Fill)		
5													
	10		3D	D	10-12	24/11	5-4-8-11				Medium dense, brown, wet, gravelly silty SAND (Fill)	∇	
										10.6	Medium stiff, gray, wet, silty CLAY, trace sand, with organics (wood, wood fiber)		
0										14.5			
	15		4D	D	15-15.3	3/3	50/3"				Dense, brown, wet, silty gravelly SAND (Glacial Till)		

Auger Refusal at 15.6 feet
Probable 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 LOG

S.W.COLE

CLIENT: VHB, Inc.
PROJECT: Mill Bridge Replacement
LOCATION: Lynch Road over Dyer Creek, Newcastle, Maine

BORING NO.: B-104
SHEET: 1 of 1
PROJECT NO. 23-1036
DATE START: 7/31/2023
DATE FINISH: 7/31/2023

Drilling Information

LOCATION: See Exploration Location Plan
ELEVATION (FT): 14' +/-
TOTAL DEPTH (FT): 3.6
LOGGED BY: Michael St. Pierre
DRILLING CO.: S. W. Cole Explorations, LLC
DRILLER: Matt Bussey
DRILLING METHOD: Solid Stem Auger
RIG TYPE: Track Mounted Mobile Drill B-48
AUGER ID/OD: N/A / 4 1/2 in
SAMPLER: Standard Split-Spoon
HAMMER TYPE: Automatic
HAMMER WEIGHT (lbs): 140
CASING ID/OD: N/A /N/A
CORE BARREL:
HAMMER CORRECTION FACTOR: 1.51
HAMMER DROP (inch): 30
WATER LEVEL DEPTHS (ft): No free water observed
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 INFORMATION: Sample No.	Type	Depth (ft)	Pen./ Rec. (in)	Blow Count or RQD	Field / Lab Test Data	Graphic Log	Sample Description & Classification	H_2O Depth	Remarks
										0.3 3 1/2" of Pavement		
			1D		1.4-3.2	21/19	24-20-27-50/3"			Dense, gray, damp, SAND and GRAVEL, some silt, with cement (Fill - Possible Cement Treated Base/Subbase Gravel)		
										1.9 Dense, brown, damp, gravelly SAND, some silt (Fill)		

Auger Refusal at 3.6 feet
Probable Boulder

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 LOG

S.W.COLE

CLIENT: VHB, Inc.
PROJECT: Mill Bridge Replacement
LOCATION: Lynch Road over Dyer Creek, Newcastle, Maine

BORING NO.: B-104A
SHEET: 1 of 2
PROJECT NO. 23-1036
DATE START: 7/31/2023
DATE FINISH: 7/31/2023

Drilling Information

LOCATION: See Exploration Location Plan
ELEVATION (FT): 14' +/-
TOTAL DEPTH (FT): 27.2
LOGGED BY: Michael St. Pierre
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 / 2
HAMMER CORRECTION FACTOR: 1.51
HAMMER DROP (inch): 30
WATER LEVEL DEPTHS (ft): ⊻ 8.2 ft during drilling
GENERAL NOTES:

KEY TO NOTES AND SYMBOLS:

Water Level	D = Split Spoon Sample	Pen. = Penetration Length	WOR = Weight of Rods	S_v = Field Vane Shear Strength, kips/sq.ft.
⊻ At time of Drilling	U = Thin Walled Tube Sample	Rec. = Recovery Length	WOH = Weight of Hammer	q_U = Unconfined Compressive Strength, kips/sq.ft.
▼ At Completion of Drilling	R = Rock Core Sample	bpf = Blows per Foot	RQD = Rock Quality Designation	Ø = Friction Angle (Estimated)
▼ After Drilling	V = Field Vane Shear	mpf = Minute per Foot	PID = Photoionization Detector	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	Strata Depth (ft)	Sample Description & Classification	H_20 Depth	Remarks
										See boring B-104 for summary of strata from 0 to 3.6 ft.		
10									3.6	Medium dense, brown, damp, gravelly SAND, some silt (Fill)		
	5		2D		5-7	24/16	39-16-9-7	q_P=5.0-7.5 ksf				
									5.8	Very stiff, gray, moist, clayey sandy SILT, trace gravel (Fill)	⊻	
5	10		3D		10-12	24/12	19-11-2-1	ID 15034A w =39.8 % W_L=32 W_P=18 O =9.4 %		Stiff, gray, wet, SILT and SAND, some clay, some gravel (Fill)		
									10.6	Medium stiff, gray, wet, silty CLAY, trace sand, with organics (wood, wood fiber)		
0									14.0	Dense, brown, wet, silty gravelly SAND (Glacial Till)		
	15		4D		15-17	24/14	9-12-13-23	ID 15035A w =11.5 %				
			1R		17.2-22.2	60/56	55	ID 15265A q_U=13050 psi Unit Wt = 162 pcf	17.2	Gray, fine-grained, quartz-biotite GRANOFELS, hard, fresh, joints are very close to moderately close, horizontal to steep, and tight to open (Bucksport Formation)		
-5	20											
			2R		22.2-27.2	60/60	53					
-10												

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.

(Continued Next Page)

BORING LOG

S.W.COLE

CLIENT: VHB, Inc.
PROJECT: Mill Bridge Replacement
LOCATION: Lynch Road over Dyer Creek, Newcastle, Maine

BORING NO.: B-104A
SHEET: 2 of 2
PROJECT NO. 23-1036
DATE START: 7/31/2023
DATE FINISH: 7/31/2023

Elev. (ft)	Depth (ft)	Casing Pen. (bpf)	SAMPLE INFORMATION						Graphic Log	Sample Description & Classification	H_20 Depth	Remarks
			Sample No.	Type	Depth (ft)	Pen./ Rec. (in)	Blow Count or RQD	Field / Lab Test Data				

Bottom of Exploration at 27.2 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.: B-104A

Mill Bridge Replacement
Lynch Road over Dyer Creek
Newcastle, Maine

Boring No.	Run	Depth (ft)	Recovery (%)	RQD (%)	Rock Type	Box Row
B-104A	R1	17.2-22.2	93%	55%	GRANOFELS	1
B-104A	R2	22.2-27.2	100%	53%	GRANOFELS	2
B-101	R1	23.8-28.8	100%	85%	GRANOFELS	3/4

APPENDIX D

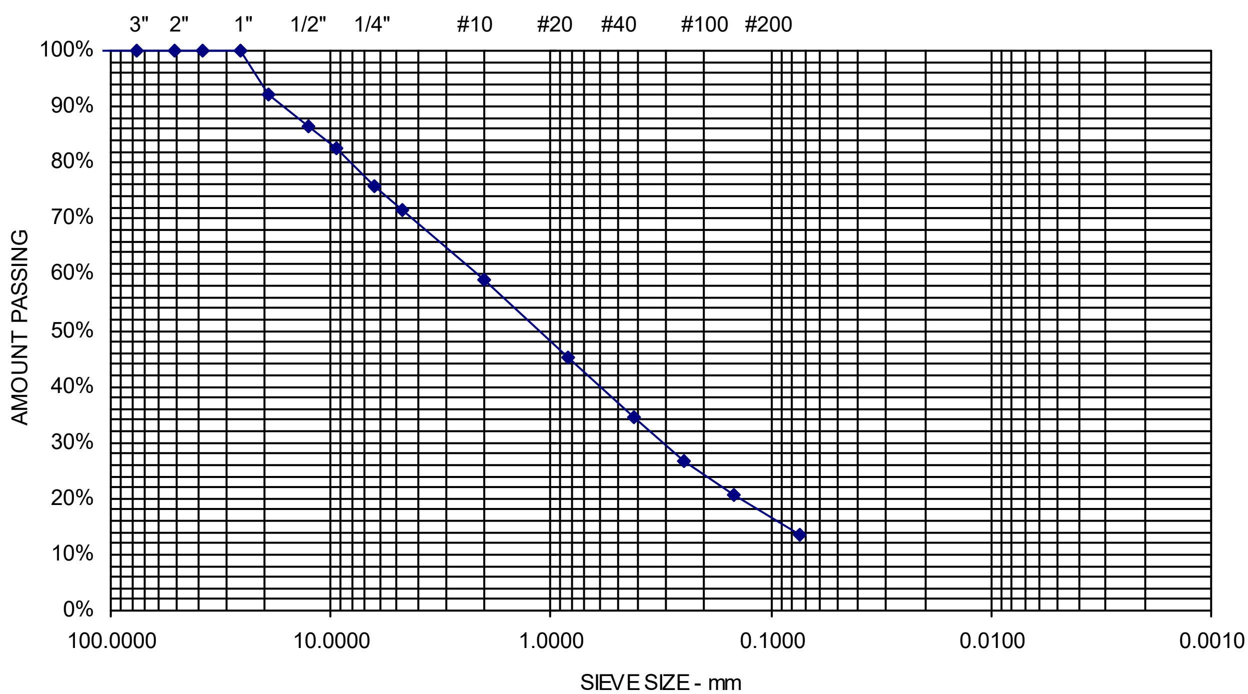
Laboratory Test Results

Report of Gradation

ASTM C-117 & C-136

Project Name	NEWCASTLE ME - MILL BRIDGE #0618 REPLACEMENT - EXPLORATIONS AND GEOTECHNICAL ENGINEERING SERVICES	Project Number	23-1036
Client	VANASSE HANGEN BRUSTLIN (VHB)	Lab ID	15031A
Exploration	**B-102**	Date Received	8/7/2023
Material Source	**1D, 1.8 FT**	Date Completed	8/14/2023
		Tested By	TRAVIS SMITH

STANDARD DESIGNATION (mm/µm)	SIEVE SIZE	AMOUNT PASSING (%)	
150 mm	**6"**	**100**	
100 mm	**4"**	**100**	
75 mm	**3"**	**100**	
50 mm	**2"**	**100**	
38.1 mm	**1-1/2"**	**100**	
25.0 mm	**1"**	**100**	
19.0 mm	**3/4"**	**92**	
12.5 mm	**1/2"**	**86**	
9.5 mm	**3/8"**	**83**	
6.3 mm	**1/4"**	**76**	
4.75 mm	**No. 4**	**71**	28.6% Gravel
2.00 mm	**No. 10**	**59**	
850 um	**No. 20**	**45**	
425 um	**No. 40**	**34**	57.9% Sand
250 um	**No. 60**	**27**	
150 um	**No. 100**	**21**	
75 um	**No. 200**	**13.5**	13.5% Fines



Comments: w = 5.9%

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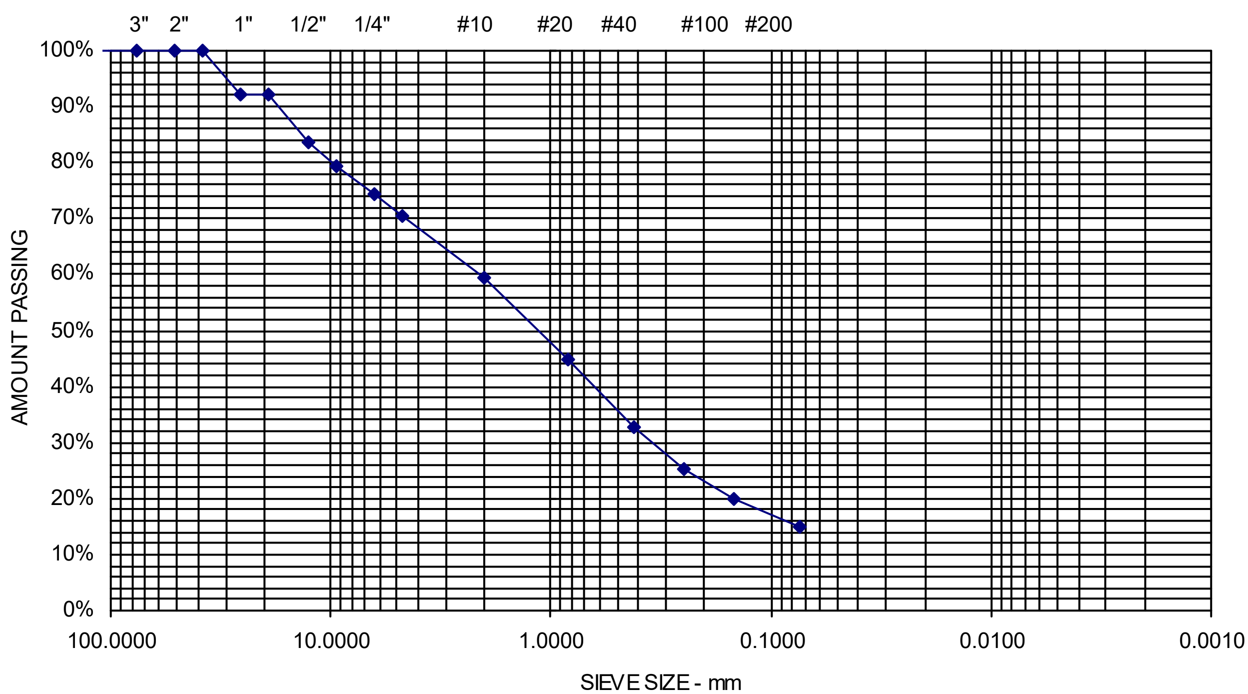
S.W.COLE ENGINEERING, INC.

Report of Gradation

ASTM C-117 & C-136

Project Name	NEWCASTLE ME - MILL BRIDGE #0618 REPLACEMENT - EXPLORATIONS AND GEOTECHNICAL ENGINEERING SERVICES	Project Number	23-1036
Client	VANASSE HANGEN BRUSTLIN (VHB)	Lab ID	15032A
Exploration	**B-103**	Date Received	8/7/2023
Material Source	**2D, 5 FT**	Date Completed	8/14/2023
		Tested By	RICHARD SEYMOUR III

STANDARD DESIGNATION (mm/µm)	SIEVE SIZE	AMOUNT PASSING (%)	
150 mm	**6"**	**100**	
100 mm	**4"**	**100**	
75 mm	**3"**	**100**	
50 mm	**2"**	**100**	
38.1 mm	**1-1/2"**	**100**	
25.0 mm	**1"**	**92**	
19.0 mm	**3/4"**	**92**	
12.5 mm	**1/2"**	**84**	
9.5 mm	**3/8"**	**79**	
6.3 mm	**1/4"**	**74**	
4.75 mm	**No. 4**	**71**	29.4% Gravel
2.00 mm	**No. 10**	**59**	
850 um	**No. 20**	**45**	
425 um	**No. 40**	**33**	55.5% Sand
250 um	**No. 60**	**25**	
150 um	**No. 100**	**20**	
75 um	**No. 200**	**15.1**	15.1% Fines



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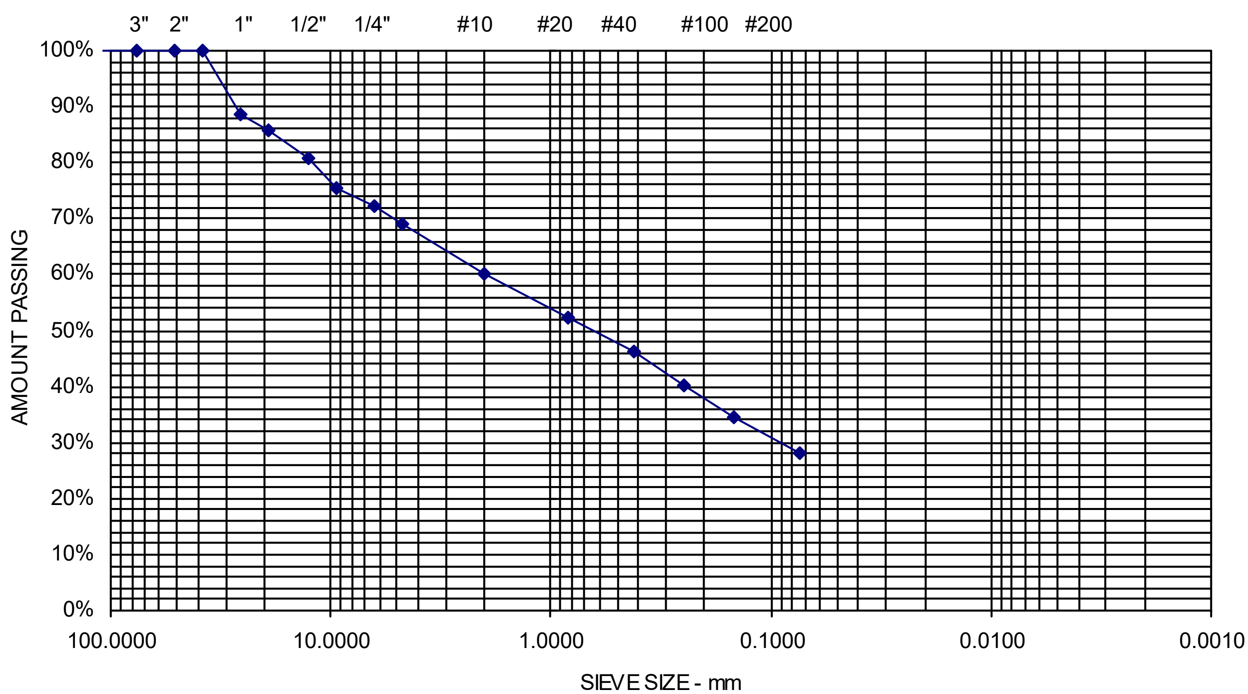
S.W.COLE ENGINEERING, INC.

Report of Gradation

ASTM C-117 & C-136

Project Name	NEWCASTLE ME - MILL BRIDGE #0618 REPLACEMENT - EXPLORATIONS AND GEOTECHNICAL ENGINEERING SERVICES	Project Number	23-1036
Client	VANASSE HANGEN BRUSTLIN (VHB)	Lab ID	15035A
Exploration	**B-104A**	Date Received	8/7/2023
Material Source	**4D, 15 FT**	Date Completed	8/14/2023
		Tested By	EMMA ROBERTS

STANDARD DESIGNATION (mm/µm)	SIEVE SIZE	AMOUNT PASSING (%)	
150 mm	**6"**	**100**	
100 mm	**4"**	**100**	
75 mm	**3"**	**100**	
50 mm	**2"**	**100**	
38.1 mm	**1-1/2"**	**100**	
25.0 mm	**1"**	**89**	
19.0 mm	**3/4"**	**86**	
12.5 mm	**1/2"**	**81**	
9.5 mm	**3/8"**	**76**	
6.3 mm	**1/4"**	**72**	
4.75 mm	**No. 4**	**69**	31.1% Gravel
2.00 mm	**No. 10**	**60**	
850 um	**No. 20**	**52**	
425 um	**No. 40**	**46**	40.8% Sand
250 um	**No. 60**	**40**	
150 um	**No. 100**	**35**	
75 um	**No. 200**	**28.1**	28.1% Fines



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