GEOTECHNICAL DESIGN REPORT

18-0136

June 25, 2021

Explorations and Geotechnical Engineering Services

Farmington Falls Bridge #2273 Replacement
Route 41 over Sandy River
Farmington-Chesterville, Maine
WIN 022296.00

PREPARED FOR:
Erdman Anthony
Attention: Christopher Sichak, P.E.
145 Culver Road, Suite 200
Rochester, NY 14620

PREPARED BY:
S. W. Cole Engineering, Inc.
26 Coles Crossing Drive
Sidney, ME 04330
T: (207) 626-0600

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18-0136

June 25, 2021

Erdman Anthony
Attention: Christopher Sichak, P.E.
145 Culver Road, Suite 200
Rochester, NY 14620

Subject: Preliminary Geotechnical Design Report
Explorations and Geotechnical Engineering Services
Farmington Falls Bridge #2273 Replacement
Route 41 over Sandy River
Farmington-Chesterville, Maine
WIN 022296.00

Dear Chris:

In accordance with our Phase I Geotechnical Proposal, dated November 7, 2019 and Phase II Geotechnical Proposal, dated August 10, 2020, we have made subsurface explorations and subcontracted geophysical testing for the subject project. The purpose of our services was to obtain subsurface information in order to provide geotechnical parameters and recommendations for foundations and earthwork associated with the proposed construction. The contents of this report are subject to the limitations in Appendix A.

1.0 INTRODUCTION

1.1 Site Conditions

The site is Farmington Falls Bridge #2273 carrying Route 41 (Vienna Road) over Sandy River separating Farmington and Chesterville, Maine. The site location is shown on the "Site Location Map" attached in Appendix B.

The existing structure consists of a four-span, concrete T-beam superstructure supported on reinforced concrete abutments and piers. We understand the existing structure is about 215 feet long (end-to-end) and about 30 feet wide (out-to-out) with zero skew. Based on the provided information, we understand the existing concrete bridge was constructed in 1931 with pier rehabilitation completed in 2011. Historic Bridge Plans indicate the abutments and pier foundations from the pre-1931 bridge structure were abandoned. We understand concrete deterioration of the bridge deck requires routine maintenance and patching. A Highway Bridge Inspection Report, dated June 16, 2017, indicates the structure is in overall "poor" condition with the deck and superstructure rated as 4 (poor condition) and the substructure rated as 5 (fair condition).

1.2 Proposed Construction

Based on the Preliminary Design Report (PDR), we understand the existing bridge will be replaced with a new 2 span structure with an overall length of about 225 feet. We understand the new bridge will be supported on spread footings founded on bedrock. We understand the alignment will be shifted slightly upstream (west) at the south abutment (Abutment No. 1) and near the existing alignment at the north abutment (Abutment No. 2). We understand 1.75H:1V riprap side slopes will wrap around and in front of the abutments matching into 2H:1V approach side slopes.

Due to the length of an off-site detour, we understand construction is considering use of a temporary bridge located upstream (west) of the existing bridge to maintain traffic.

2.0 EXPLORATIONS AND TESTING

2.1 Explorations

Subsurface conditions were explored by drilling seven test borings. Borings BB-FCSR-101, BB-FCSR-102 and BB-FCSR-103 were made at the site between January 20 and 23, 2020 by S. W. Cole Explorations, LLC using a track-mounted CME 850 drill rig. Borings BB-FCSR-201, BB-FCSR-202, BB-FCSR-202A and BB-FCSR-202B were made at the site between November 20, 2020 and December 9, 2020 by S. W. Cole Explorations, LLC (S.W.COLEX) using a track-mounted Diedrich D-50 drill rig. The exploration locations were selected by S.W.COLE in consultation with Erdman Anthony (EA) and established in the field by S.W.COLE using taped measurements from existing site features. The ground surface elevations of the test borings were provided by EA based on the taped measurements. The approximate exploration locations are shown on the "Boring Location Plan" attached in Appendix B. Logs of the test borings and a Key to Soil and Rock Descriptions and Terms used on the logs are attached as Appendix C.

2.2 Testing

The test borings were drilled using a combination of solid-stem auger, cased wash boring and rock core drilling techniques. The soils were sampled at approximate 5 foot intervals using a split-spoon sampler and Standard Penetration Testing (SPT) methods using calibrated automatic hammers. Upon encountering refusal, borings BB-FCSR-101, BB-FCSR-102, BB-FCSR-103, BB-FCSR-201 and BB-FCSR-202B were advanced up to 10 feet into bedrock using either BX or NQ2 rock coring. Borings BB-FCSR-202 and BB-FCSR-202A was terminated prior to encountering refusal on bedrock.

Both the S.W.COLEX drill rigs were equipped with automatic hammers to drive the split-spoon sampler. Both hammers were calibrated per ASTM D4633-10 "Standard Test Method for Energy Measurement for Dynamic Penetrometers." Corrected N-values discussed in this report were computed by applying the corresponding average energy transfer factors of 0.801 (CME 850 hammer) and 0.995 (Diedrich D-50 hammer) to the raw field N values. The hammer efficiency factors (0.801 and 0.995), uncorrected SPT blow counts and the uncorrected and corrected

SPT N-values, rock core intervals and Rock Quality Designation (RQD) are shown on the boring logs provided in Appendix C.

Soil and rock core samples recovered from the test borings were visually classified in the field and returned to our laboratory for further classification. Laboratory testing was performed on disturbed SPT and rock core samples obtained during the explorations. Laboratory testing was performed by GeoTesting Express of Acton, Massachusetts, in accordance with applicable American Association of State Highway and Transportation Officials (AASHTO) testing procedures. Laboratory testing included 4 natural water content tests, 4 grain size analyses (2 with hydrometer and 2 without hydrometer), 2 rock core unit weight and 2 unconfined rock core compressive strength tests. A summary of the laboratory testing results are provided in Appendix D.

2.3 Geophysical Testing

NDT Corporation (NDT) of Sterling, Massachusetts, working under subcontract to S.W.COLE, conducted a geophysical investigation using seismic refraction along the alignment (within channel and abutments) on November 11, 2020. The purpose of the seismic refraction surveys was to profile the top of bedrock along the proposed bridge alignment.

NDT completed seismic refraction surveys along four (4) 100 foot lines within the Sandy River channel, two (2) 100 foot lines along the South approach and one (1) 100 foot line along the North approach. NDT's geophysical investigation report is attached as Appendix E. Results of the seismic refraction surveys have been incorporated on the Interpretive Subsurface Profile (ISP) provided in Appendix B.

3.0 SUBSURFACE CONDITIONS

3.1 Surficial and Bedrock Geology

According to the Maine Geological Survey's (MGS's) mapping of the Farmington Quadrangle, Maine[1], surficial geologic units within the site vicinity consists of stream alluvium (sand, gravel and silt) and glacial till. The subsurface conditions encountered in the test borings generally consisted of fill soils from previous site development overlying alluvium overlying glacial till. Alluvium and glacial till were not encountered at the north abutment (Abutment No. 2).

According to MGS[2], bedrock in the site vicinity is mapped as Sangerville Formation, Anasagunticook Member metasiltstone and metapelite. The bedrock, where sampled, was generally composed of dark grey, phyllitic schist with some garnets.

[1] Caldwell W. D., 1986, Surficial Geology of the Farmington Quadrangle, Maine, Maine Geological Survey, Open-File Map 86-29, Map, Scale 1:62,500.

[2] Pankiwskyj, K. A., 1978, Bedrock Geology of the Farmington Quadrangle, Maine Geological Survey, Open File Map 78-16.

3.2 Subsurface Conditions

The test borings encountered a soils profile generally consisting of fill overlying alluvium and glacial till mantling bedrock. The principal strata encountered in the explorations are summarized below. An "Interpretive Subsurface Profile" is attached in Appendix B. Refer to the attached logs for more detailed information of the subsurface findings at the exploration locations.

Pavement: Boring BB-FCSR-101 was made behind the existing south abutment and encountered a 7 inch surface layer of pavement. Borings BB-FCSR-102, BB-FCSR-201, BB-FCSR-202, BB-FCSR-202A and BB-FCSR-202B were made through the 11 to 12.75-inch thick bridge deck.

Fill: Below the pavement in BB-FCSR-101 and from the ground surface in BB-FCSR-103, these borings encountered fill extending to depths of about 10 and 14.5 feet below ground surface (bgs), corresponding to Elevation (El.) 334.1 and 328.8 feet. The fill generally consisted of:

- Brown, SAND, some gravel, little silt;
- Brown, SAND, little gravel, little silt;
- Brown, SAND, some silt, little to trace gravel; and
- Brown, SAND, some silt, little gravel, trace clay.

The fill was generally loose to dense with SPT N_{60} values ranging from 7 to 39 blows per foot (bpf).

Alluvium: Below the fill in BB-FCSR-101 and from the river bottom in borings BB-FCSR-102, BB-FCSR-201, BB-FCSR-202, BB-FCSR-202A and BB-FCSR-202B, these borings encountered stream alluvium to depths of about 0.5 to 19 feet bgs, corresponding to Elevation (El.) 310.0 to 321.1 feet. Where sampled, the alluvium generally consisted of:

- Brown, SILT, some to trace sand, little clay, trace gravel;
- Grey, SILT, little clay, little to trace sand;
- Brown, SAND, little silt, trace clay;
- Brown, SAND, some gravel, little silt;
- Grey and brown, GRAVEL, some sand, trace silt; and
- Brown, Sandy GRAVEL, trace silt.

The alluvial silt was generally soft to hard with SPT N_{60} values ranging from 4 to 51 bpf. The alluvial sand was generally very loose to very dense with SPT N_{60} values ranging from of 3 to 73 bpf.

Glacial Till: Below the alluvium, borings BB-FCSR-101, BB-FCSR-102 and BB-FCSR-202, encountered glacial till with cobbles and boulders to depths of about 9.0 to 29.5 feet bgs, corresponding to Elevation (El.) 305.2 to 314.6 feet. Where sampled, the glacial till consisted of:

- Grey, Sandy SILT, some gravel.

The glacial till was generally hard with an SPT N_{60} value of 147 bpf.

Bedrock: Bedrock was encountered and cored in borings BB-FCSR-101, BB-FCSR-102, BB-FCSR-103, BB-FCSR-201 and BB-FCSR-202B. The top of bedrock varied from about 9.0 to 29.5 feet bgs (±El. 306.6 to 328.8 feet). The bedrock consisted of dark grey, moderately hard, fresh, phyllitic SCHIST.

The following table summarizes the approximate depths to bedrock, corresponding top of bedrock elevations and Rock Quality Designation (RQD) where encountered.

Boring Number (Substructure)	**Approximate Depth to Bedrock (feet)**	**Approximate Bedrock Elevation (feet)**	**RQD (RMQ) R1 R2 R3**
BB-FCSR-101 (South Abutment)	29.5	314.6	N/A (boulders) 23% (Very Poor) 17% (Very Poor)
BB-FCSR-102 (River Channel)	9.0	306.6	34% (Poor)
BB-FCSR-103	14.5	328.8	8% (Very Poor) 35% (Poor)
BB-FCSR-201 (North Abutment)	1.9	321.1	27% (Poor) 72% (Fair) 29% (Poor)
BB-FCSR-202B (River Channel)	9.9	306.0	10% (Very Poor) 40% (Poor) 50% (Poor)

Rock quality designation (RQD) values for the bedrock generally ranged from 17 to 72 percent corresponding to a Rock Mass Quality (RMQ) of very poor to fair. Detailed descriptions of the rock core and RQD values for each core run are shown on the exploration logs in Appendix C.

3.3 Groundwater Conditions

The soils encountered at the test borings were damp to wet from the ground surface. The measured water levels during drilling ranged from 9.5 feet bgs at BB-FCSR-101, 9 feet bgs at BB-FCSR-103, at the ground surface at BB-FCSR-201 and above the river bottom at BB-FCSR-102, BB-FCSR-202, BB-FCSR-202A and BB-FCSR-202B made through the bridge deck. 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 level of the Sandy River.

4.0 PRELIMINARY GEOTECHNICAL EVALUATION

S.W.COLE conducted geotechnical engineering evaluations in accordance with 2017 AASHTO LRFD Bridge Design Specifications, 8th Edition (LRFD) and the MaineDOT Bridge Design

Guide, 2003 Edition with revisions through June 2018 (MaineDOT BDG) and offers the following:

4.1 Foundation Options

The PDR identifies a 2 span, steel and concrete structure supported on spread footings bearing on bedrock. We understand the alignment will be shifted slightly upstream (west) at the south abutment (Abutment No. 1) and near the existing alignment at the north abutment (Abutment No. 2). We understand 1.75H:1V riprap side slopes will wrap around and in front of the abutments matching into 2H:1V approach embankment side slopes.

The site is underlain by loose to very dense fills and soft to hard alluvium overlying hard glacial till overlying bedrock. The river channel is lined with cobbles, boulders and granite blocks from previous site development and alluvial deposition. Based on the findings at the exploration locations and seismic refraction surveys, we anticipate bedrock will be at about El. 320 feet at the north abutment, about El. 310 feet at the south abutment and about El. 306 feet at the interior pier.

Based on the depth to bedrock, we anticipate abutment excavations will require braced excavations to retain the existing approaches and the interior pier excavation will require construction of a cofferdam dam and dewatering.

4.2 Abutment, Wingwall and Pier Design

The abutments, wingwalls and pier 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. In addition, the interior pier shall be designed to transmit the loads on the superstructure and the loads acting on the pier itself into the foundation.

4.2.1 Strength Limit State Design

The design of abutments, wingwalls and concrete pier founded on spread footings bearing on bedrock or 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.

Additionally, a modified strength limit state analysis should be performed for the pier foundation that includes the ice pressures specified in MaineDOT BDG Section 3.9 Ice Loads.

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.

4.2.2 Service Limit State Design

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 are 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.

4.2.3 Extreme Limit State Design

Extreme limit state design checks for abutments, wingwalls and pier 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 with the exception of 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.

The ice pressures for Extreme Event II shall be applied at the Q1.1 and Q50 elevations as defined in MaineDOT BDG Section 3.9 with the design ice thickness increased by 1 foot and a load factor of 1.0.

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.

4.3 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, wingwall and pier 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	9.2	Table 10.5.5.2.2-1
Extreme	0.8	16.4	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.

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.

4.4 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 Article C10.5.5.2.2
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, wingwalls and pier 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 MaineDOT BDG Section 5.2.2, 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.

4.5 Earth Pressure and Surcharge

4.5.1 Earth Pressure

The abutments and wingwalls should be designed for active earth pressure over the wall height unless restrained from movement. Walls restrained from movement should be designed for at-rest active earth pressure over the wall height. For design of gravity and semi-gravity walls backfilled with granular soil and drained (e.g. no hydrostatic pressures), we recommend the following earth pressure coefficients:

- Active Earth Pressure Coefficient, k_a = 0.28
- At-rest Earth Pressure Coefficient, k_o = 0.47

The resultant earth pressure is orientated at an angle δ of 21.33 degrees from a perpendicular line to the wall back-face, where δ is the angle of friction between the abutment backfill soil and the wall back-face.

Based on MaineDOT BDG Section 3.6.1, the designer may assume Soil Type 4 for the backfill material with the following soil properties:

- Internal Friction Angle, ϕ = 32 degrees
- Total Unit Weight, γ = 125 pcf

4.5.2 Surcharge Pressure

Lateral earth pressure due to construction surcharge or live load surcharge is required per MaineDOT BDG Section 3.6.8 for the abutments and wingwalls if an approach slab is not specified. When a structural approach slab is specified, reduction, not elimination of the surcharge loads is permitted per LRFD Article 3.11.6.5.

The live load surcharge on wing walls may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) of 2.0 feet, per LRFD Table 3.11.6.4-2. The live load surcharge on abutments may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) based on the following:

Abutment Height (feet)	Equivalent Height of Soil, h_{eq} (feet)
5	4.0
10	3.0
≥20	2.0

Abutment and wingwall modifications and design shall include a drainage system to ensure that drainage of water behind the structure is maintained. Drainage behind the structures shall be in accordance with MaineDOT BDG Section 5.4.1.4 Drainage.

4.6 Frost Considerations

According to MaineDOT BDG Section 5.2.1 and BDG Figure 5-2, pile supported foundations shall 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. Based on the Maine Design Freezing Index Map[3], the design freezing index for the Farmington-Chesterville, Maine area is approximately 1,800 freezing degree-days. Based on Section 5.2.1 of the MaineDOT BDG and subsurface soils encountered, the maximum seasonal frost penetration is estimated to be on the order of about 7.5 feet; consequently, we recommend foundations should have at least 7.5 feet of soil cover to provide frost protection.

[3] Maine Department of Transportation, Bridge Design Guide (BDG), August 2003, with Revisions through June 2018, Figure 5-1.

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

4.7 Seismic Design Considerations

Seismic site class was evaluated in accordance with LRFD Article 3.10.3.1 Method B (average N-value method). Based on the subsurface information, the average N-value for the soil profile 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). The results for the project site are summarized below and program output are provided in Appendix F.

RECOMMENDED SEISMIC DESIGN PARAMETERS	
Site Class	D
PGA	0.081 g
S_s	0.169 g
S_1	0.048 g
F_{pga}	1.6
F_a	1.6
F_v	2.4
A_s	0.130 g
S_{DS}	0.271 g
S_{D1}	0.115 g
Seismic Zone (based on S_{D1})	Zone 1

NOTE: Site Coordinates: N44.62008, W70.07484

4.8 Recommendations for Scour Evaluation

Laboratory grain size analyses were performed on soil samples taken near the approximate streambed elevation to generate parameters to be used in scour analyses. Results of the grain size analyses tests are included in Appendix D and summarized in the following table:

Boring No.	Sample No.	Depth (ft)	Elevation (ft)	Estimated D_{95} (mm)	Estimated D_{50} (mm)
BB-FCSR-101	3D	10	334.1	0.62	0.027
BB-FCSR-103	3D	10	333.3	21	0.347

Design at the strength limit state should consider loss of lateral support due to scour. Design at the extreme limit state should check that the nominal foundation resistance due to the check flood (Q_{500}) event is no less than the extreme limit state loads. At the service limit state, the design shall limit movements and ensure overall stability considering scour at the design load.

For scour protection of footings on bedrock, place the bottom of concrete seals or footings directly on bedrock surfaces cleaned of all weathered, loose and potentially erodible or scourable rock. Bridge and channel soil slopes above the soil-bedrock interface shall be armored with 3 feet of riprap.

Riprap shall conform to MaineDOT Standard Specification 703.26 “Plain and Hand Laid Riprap” and shall be placed at a maximum slope of 1.75H:1V. The riprap section shall be underlain by a 1 foot layer of MaineDOT Standard Specification 703.19 “Granular Borrow Material for Underwater Backfill” and a Class 1 nonwoven erosion control geotextile per MaineDOT Standard Specification 722.03.

5.0 CLOSURE

We trust this information meets your present needs. Please contact us if you have any questions or need further assistance.

Sincerely,

S. W. Cole Engineering, Inc.

Digitally signed by Michael St. Pierre, P.E.
Date: 2021.06.28 13:49:37 -04'00'

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

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

Timothy J. Boyce, P.E.
Senior Geotechnical Engineer

MAS/tjb

APPENDIX A
LIMITATIONS

This report has been prepared for the exclusive use of Erdman Anthony and the Maine Department of Transportation for specific application to the Farmington Falls Bridge #2273 Replacement carrying Route 41 over Sandy River (MaineDOT WIN 022296.00) in Farmington-Chesterville, 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.

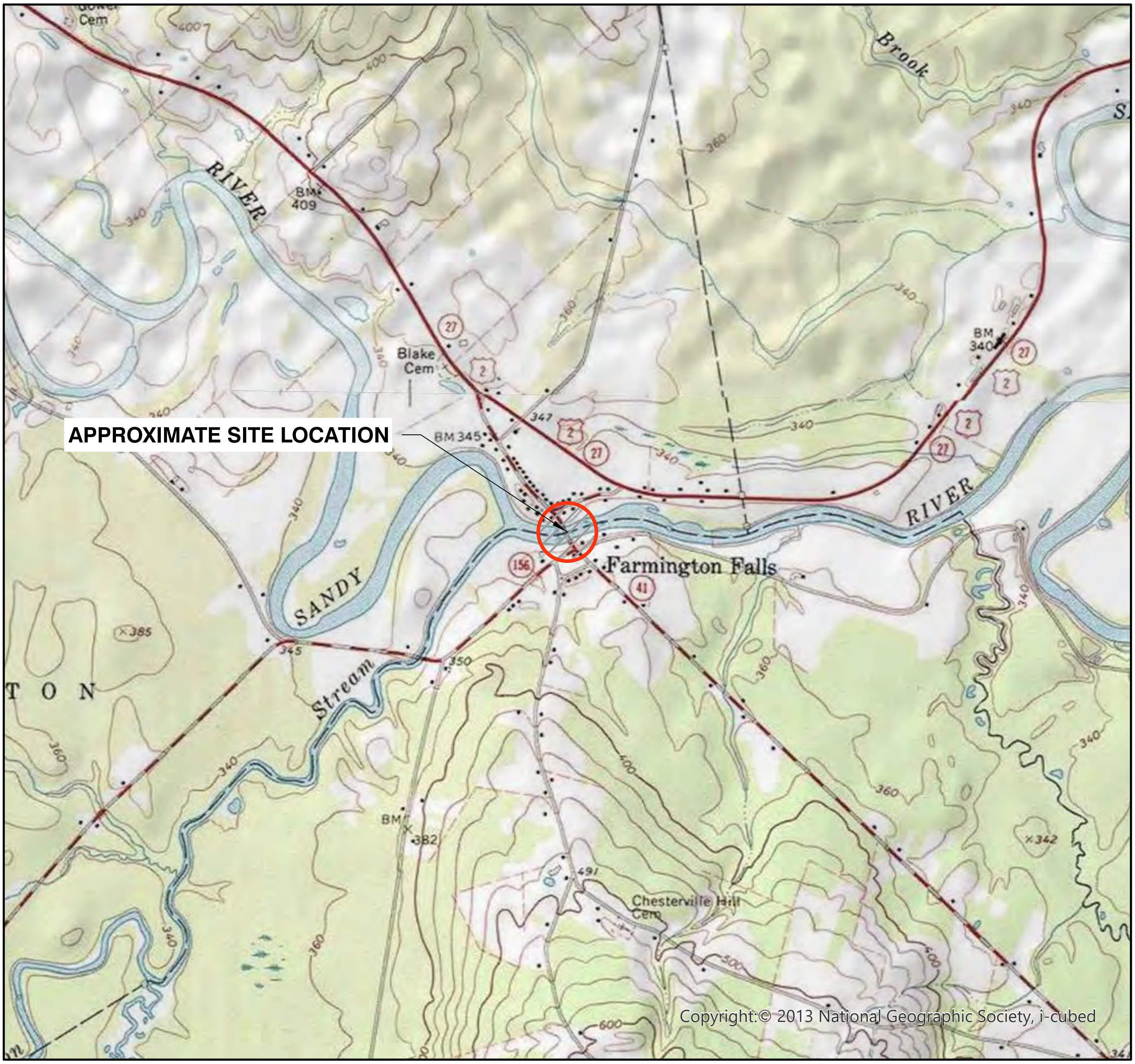
The analyses performed during this investigation and recommendations presented in this report are based in part upon the data obtained from subsurface explorations made at the site. Variations in subsurface conditions may occur between explorations and may not become evident until construction. If variations 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.

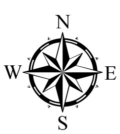
Observations have been made during exploration work to assess site groundwater levels. Fluctuations in water levels will occur due to variations in rainfall, temperature, and other factors.

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.

S.W.COLE ENGINEERING, INC.

APPENDIX B
Figures





2,000 0 2,000 4,000

Scale in Feet

S.W.COLE ENGINEERING, INC.

ERDMAN ANTHONY

SITE LOCATION MAP

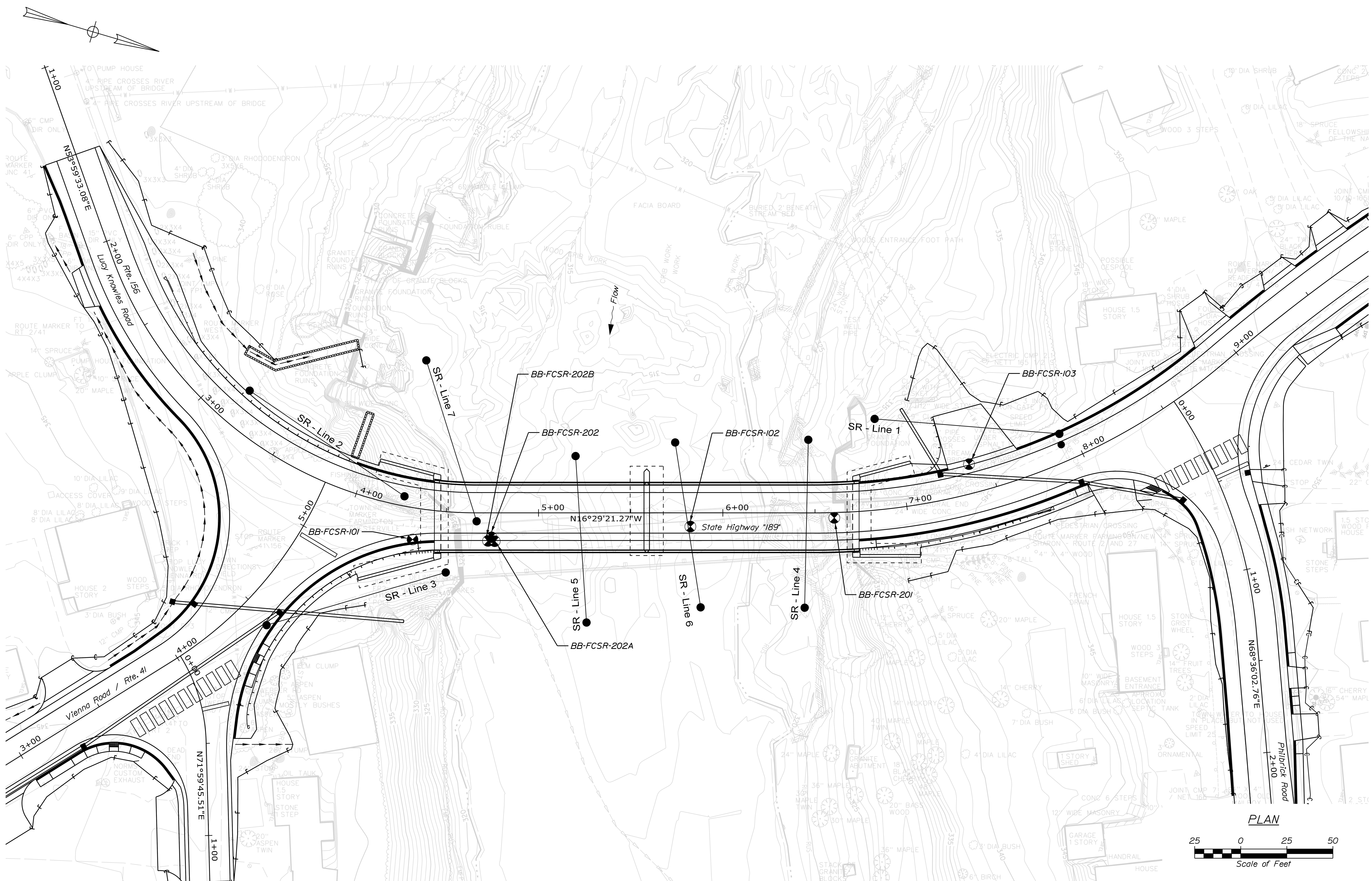
FARMINGTON FALLS BRIDGE #2273 REPLACEMENT
ROUTE 41 OVER SANDY RIVER
FARMINGTON-CHESTERVILLE, MAINE
WIN 022296.00

Job No.	18-0136	Scale	1" = 2000'
Date:	3/18/2020	Sheet	1

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

R:\2018\18-0136\GIS\Project\18-0136.aprx, 3/18/2020 4:12 PM, 1:24,000,

Filename: ...\008_BoringPlan.dgn
Division: BRIDGE
Username: LindoT
Date:6/25/2021



LEGEND

CASED WASH BORING

100' Seismic Refraction Line Location

NOTE

Full seismic refraction survey report is available in the Geotechnical Design Report.

ERDMAN ANTHONY

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

2229600

WIN
22296.00

BRIDGE NO. 2273

BRIDGE PLANS

		BY	DATE
PROJ. MANAGER	MICHAEL WIGHT		
DESIGN-DETAILED	C. SICHAK	R. PARKER	
CHECKED-REVIEWED			
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

SIGNATURE

P.E. NUMBER

DATE

FARMINGTON FALLS BRIDGE
SANDY RIVER
CHESTERVILLE-FARMINGTON FRANKLIN COUNTY

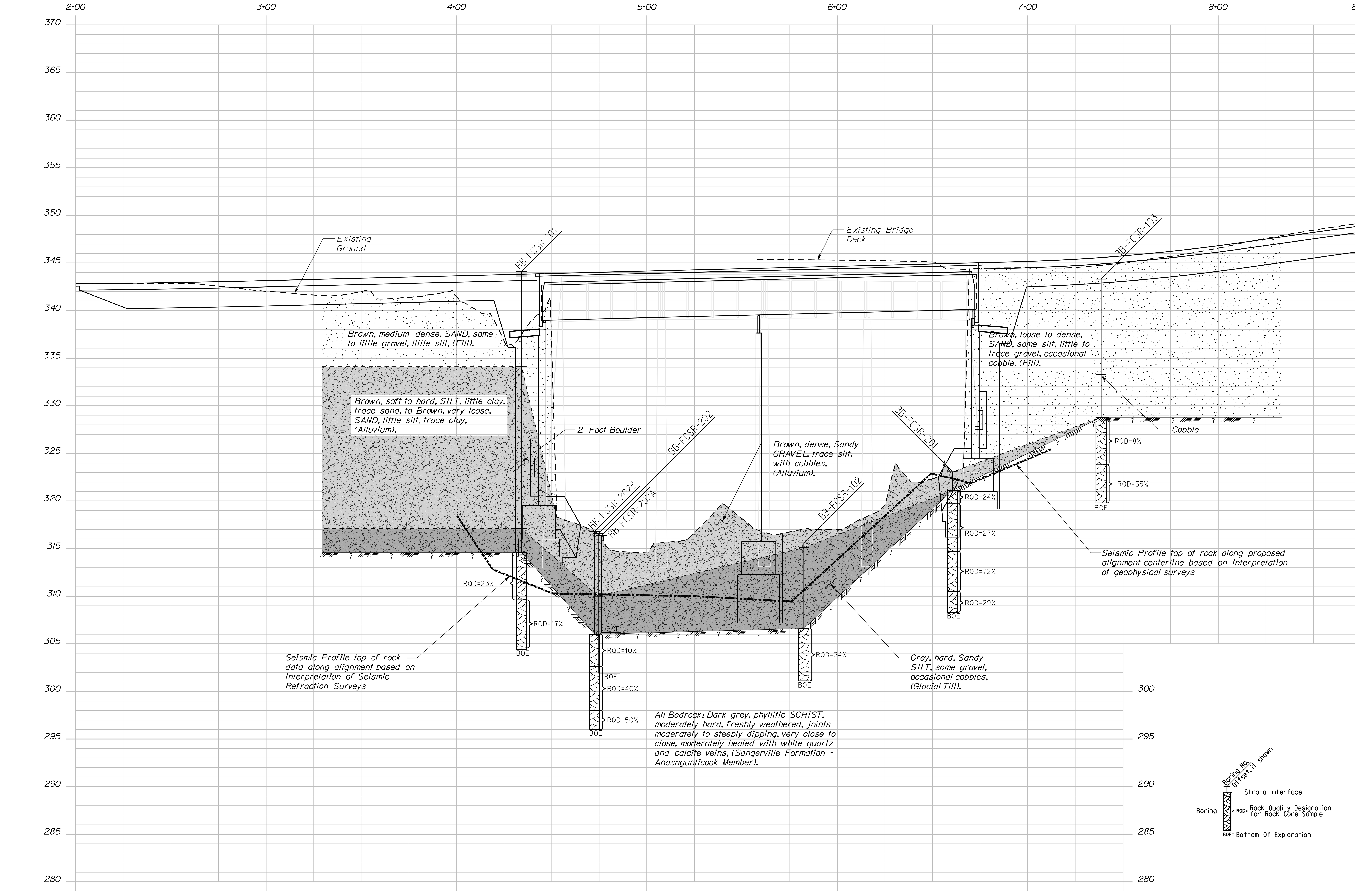
BORING LOCATION PLAN

SHEET NUMBER

2

OF 3

Filename: ...\003_ISProfile.dgn
Division: BRIDGE
Username: LindoT
Date:6/25/2021



HOR. 25 0 25 50
VER. 5 0 5 10
Scale

Note: This generalized interpretive soil profile is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized, and have been developed by interpretations of widely spaced explorations and samples. Actual soil transitions may vary and are probably more erratic. For more specific information refer to the exploration logs.

PROFILE (℄ Construction #1)
VIENNA ROAD (ROUTE 41)
STATE HIGHWAY 189

ERDMAN ANTHONY

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

WIN 22296.00
BRIDGE NO. 2273
BRIDGE PLANS

	BY	DATE
PROJ. MANAGER	MICHAEL WIGHT	
DESIGN-DETAILED	R. PARKER	
CHECKED-REVIEWED	C. SICHAK	
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

SIGNATURE
P.E. NUMBER
DATE

FARMINGTON FALLS BRIDGE
SANDY RIVER
CHESTERVILLE-FARMINGTON FRANKLIN COUNTY

INTERPRETIVE SUBSURFACE PROFILE

SHEET NUMBER
3
OF 3

APPENDIX C

Boring Logs & Key to Soil and Rock Descriptions and Terms

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE-GRAINED SOILS (more than half of material is larger than No. 200 sieve size)	GRAVELS (more than half of coarse fraction is larger than No. 4 sieve size)	CLEAN GRAVELS (little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly-graded gravels, gravel sand mixtures, little or no fines.
		GRAVEL WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS (more than half of coarse fraction is smaller than No. 4 sieve size)	CLEAN SANDS (little or no fines)	SW	Well-graded sands, Gravelly sands, little or no fines
			SP	Poorly-graded sands, Gravelly sand, little or no fines.
		SANDS WITH FINES (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures.
FINE-GRAINED SOILS (more than half of material is smaller than No. 200 sieve size)	SILTS AND CLAYS (liquid limit less than 50)		ML	Inorganic silts and very fine sands, rock flour, Silty or Clayey fine sands, or Clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, Gravelly clays, Sandy clays, Silty clays, lean clays.
			OL	Organic silts and organic Silty clays of low plasticity.
	SILTS AND CLAYS (liquid limit greater than 50)		MH	Inorganic silts, micaceous or diatomaceous fine Sandy or Silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
	HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.

Desired Soil Observations (in this order, if applicable):
Color (Munsell color chart)
Moisture (dry, damp, moist, wet)
Density/Consistency (from above right hand side)
Texture (fine, medium, coarse, etc.)
Name (Sand, Silty Sand, Clay, etc., including portions - trace, little, etc.)
Gradation (well-graded, poorly-graded, uniform, etc.)
Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic)
Structure (layering, fractures, cracks, etc.)
Bonding (well, moderately, loosely, etc.,)
Cementation (weak, moderate, or strong)
Geologic Origin (till, marine clay, alluvium, etc.)
Groundwater level

MODIFIED BURMISTER SYSTEM

Descriptive Term	Portion of Total (%)
trace	0 - 10
little	11 - 20
some	21 - 35
adjective (e.g. Sandy, Clayey)	36 - 50

TERMS DESCRIBING DENSITY/CONSISTENCY

Coarse-grained soils (more than half of material is larger than No. 200 sieve): Includes (1) clean gravels; (2) Silty or Clayey gravels; and (3) Silty, Clayey or Gravelly sands. Density is rated according to standard penetration resistance (N-value).

Density of Cohesionless Soils	Standard Penetration Resistance N-Value (blows per foot)
Very loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

Fine-grained soils (more than half of material is smaller than No. 200 sieve): Includes (1) inorganic and organic silts and clays; (2) Gravelly, Sandy or Silty clays; and (3) Clayey silts. Consistency is rated according to undrained shear strength as indicated.

Consistency of Cohesive soils	SPT N-Value (blows per foot)	Approximate Undrained Shear Strength (psf)	Field Guidelines
Very Soft	WOH, WOR, WOP, <2	0 - 250	Fist easily penetrates
Soft	2 - 4	250 - 500	Thumb easily penetrates
Medium Stiff	5 - 8	500 - 1000	Thumb penetrates with moderate effort
Stiff	9 - 15	1000 - 2000	Indented by thumb with great effort
Very Stiff	16 - 30	2000 - 4000	Indented by thumbnail
Hard	>30	over 4000	Indented by thumbnail with difficulty

Rock Quality Designation (RQD):

$$\text{RQD (\%)} = \frac{\text{sum of the lengths of intact pieces of core* > 4 inches}}{\text{length of core advance}}$$

*Minimum NQ rock core (1.88 in. OD of core)

Rock Quality Based on RQD

Rock Quality	RQD (%)
Very Poor	≤25
Poor	26 - 50
Fair	51 - 75
Good	76 - 90
Excellent	91 - 100

Desired Rock Observations (in this order, if applicable):
Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Rock Type (granite, schist, sandstone, etc.)
Hardness (very hard, hard, mod. hard, etc.)
Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)
Geologic discontinuities/jointing:
- -dip (horiz - 0-5 deg., low angle - 5-35 deg., mod. dipping - 35-55 deg., steep - 55-85 deg., vertical - 85-90 deg.)
- -spacing (very close - <2 inch, close - 2-12 inch, mod. close - 1-3 feet, wide - 3-10 feet, very wide >10 feet)
- -tightness (tight, open, or healed)
- -infilling (grain size, color, etc.)

Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)
RQD and correlation to rock quality (very poor, poor, etc.)
ref: ASTM D6032 and FHWA NHI-16-072 GEC 5 - Geotechnical Site Characterization, Table 4-12
Recovery (inch/inch and percentage)
Rock Core Rate (X.X ft - Y.Y ft (min:sec))

Maine Department of Transportation
Geotechnical Section
Key to Soil and Rock Descriptions and Terms
Field Identification Information

Sample Container Labeling Requirements:

WIN	Blow Counts
Bridge Name / Town	Sample Recovery
Boring Number	Date
Sample Number	Personnel Initials
Sample Depth	

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Farmington Falls Bridge #2273 carries Route 41 over Sandy River
Location: Farmington-Chesterville, Maine

Boring No.: BB-FCSR-101
WIN: 022296

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	344.1	**Auger ID/OD:**	5" Solid Stem
Operator:	J. Lee	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	J. McElroy	**Rig Type:**	CME 850 Track-mounted	**Hammer Wt./Fall:**	140 lbs/30 inch
Date Start/Finish:	1/20/2020	**Drilling Method:**	Cased Wash	**Core Barrel:**	BX (1.6")
Boring Location:	Sta. 4+34.2, 17.1 ft Rt.	**Casing ID/OD:**	HW 4"/4.5" / NW 3"/3.5"	**Water Level*:**	9.5 feet (after drilling)
Hammer Efficiency Factor:	0.801	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	343.5		7 inches of Pavement. 0.6	
	1D	24/16	2.00 - 4.00	6/7/8/10	15	20				Brown, moist, medium dense, SAND, some gravel, little silt, (Fill).	
5	2D	24/12	5.00 - 7.00	5/6/4/22	10	13				Brown, moist, medium dense, SAND, little gravel, little silt, (Fill).	GTX #551303 A-1-b, SM WC = 9.8%
								334.1		10.0	
10	3D	24/12	10.00 - 12.00	1/1/2/1	3	4	18			Brown, moist, soft, SILT, some fine sand, little clay, trace gravel, (Alluvium).	GTX #551305 A-4, ML WC = 28.0%
							17				
							20				
							24				
							20				
15	4D	24/6	15.00 - 17.00	1/1/1/1	2	3	5			Brown, moist to wet, very loose, fine SAND, little silt, trace clay, (Alluvium).	
							7				
							14				
							37				
							80				
20	R1	32/24	20.00 - 22.67				NQ			2 foot boulder. Advanced by rock core through boulder and set NW casing.	
							OPEN				
25											

Remarks:

Autohammer SN 295792 Calibrated 7/29/2019.
bgs = below ground surface.
±2 feet of frost.
Water level measured at end of drilling and prior to removal of casing.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Farmington Falls Bridge #2273 carries Route 41 over Sandy River
Location: Farmington-Chesterville, Maine

Boring No.: BB-FCSR-101
WIN: 022296

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	344.1	**Auger ID/OD:**	5" Solid Stem
Operator:	J. Lee	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	J. McElroy	**Rig Type:**	CME 850 Track-mounted	**Hammer Wt./Fall:**	140 lbs/30 inch
Date Start/Finish:	1/20/2020	**Drilling Method:**	Cased Wash	**Core Barrel:**	BX (1.6")
Boring Location:	Sta. 4+34.2, 17.1 ft Rt.	**Casing ID/OD:**	HW 4"/4.5" / NW 3"/3.5"	**Water Level*:**	9.5 feet (after drilling)
Hammer Efficiency Factor:	0.801	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
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MV = Unsuccessful Field Vane Shear Test Attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140 lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25	5D	24/16	25.00 - 27.00	9/15/23/40	38	51	27			Grey, wet, hard, SILT, little clay, trace fine sand with fine to medium sand partings, (Alluvium).	GTX #551304 A-4, ML WC = 26.4%
							43				
							170	317.1		27.0 Drill rig action suggests gravelly soils.	
							219				
	R2	60/21	29.70 - 34.70	RQD = 23%			BX	314.6		29.5 Top of Bedrock at Elev 314.6 feet. Advanced by rollercone to 29.7 feet bgs. R2:Bedrock: Dark grey, phyllitic SCHIST, some garnets, moderately hard freshly weathered, joints are moderate to steeply dipping, very close to close, moderately healed with white quartz and calcite vein infilling, (Sangerville Formation - Anasagunticook Member). Rock Mass Quality = Very Poor. R2:Core Times (min:sec) 29.7-30.7 ft (3:00) 30.7-31.7 ft (1:30) 31.7-32.7 ft (1:34) 32.7-33.7 ft (1:20) 33.7-34.7 ft (1:15) 35% Recovery.	
30											
35	R3	60/60	34.70 - 39.70	RQD = 17%						R3:Bedrock: Similar to R2. Rock Mass Quality = Very Poor. R3:Core Times (min:sec) 34.7-35.7 ft (2:25) 35.7-36.7 ft (3:05) 36.7-37.7 ft (4:04) 37.7-38.7 ft (2:48) 38.7-39.7 ft (4:11) 100% Recovery.	
40								304.4		39.7 **Bottom of Exploration at 39.7 feet below ground surface.**	
45											
50											

Remarks:

Autohammer SN 295792 Calibrated 7/29/2019.
bgs = below ground surface.
±2 feet of frost.
Water level measured at end of drilling and prior to removal of casing.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Farmington Falls Bridge #2273 carries Route 41 over Sandy River
Location: Farmington-Chesterville, Maine

Boring No.: BB-FCSR-102
WIN: 022296

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	315.6	**Auger ID/OD:**	5" Solid Stem
Operator:	J. Lee	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	J. McElroy	**Rig Type:**	CME 850 Track-mounted	**Hammer Wt./Fall:**	140 lbs/30 inch
Date Start/Finish:	1/23/2020	**Drilling Method:**	Cased Wash	**Core Barrel:**	NQ2 (2")
Boring Location:	Sta. 5+82.4, 7.4 ft Rt.	**Casing ID/OD:**	HW 4"/4.5"	**Water Level*:**	26 feet below bridge deck
Hammer Efficiency Factor:	0.801	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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MV = Unsuccessful Field Vane Shear Test Attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/12	0.00 - 2.00	14/18/12/10	30	40	42	315.1		Brown, wet, dense, Sandy GRAVEL, trace silt, (Alluvium). — 0.5	
							84				
							118				
							85				
							77				
5							135				
	2D	24/16	6.00 - 8.00	49/53/57/51	110	147	80			Grey, moist, hard, Sandy SILT, some gravel, (Glacial Till).	
							78				
							54			— 9.0	
	R1	60/48	9.50 - 14.50	RQD = 34%			NX2	306.6		Top of Bedrock at Elev 306.6 feet. Advanced by rollercone to 9.5 feet. R1:Bedrock: Dark grey, phyllitic SCHIST, some garnets, moderately hard freshly weathered, joints are moderate to steeply dipping, close to moderately close, moderately healed with white quartz and calcite vein infilling, (Sangerville Formation - Anasagunticook Member) Rock Mass Quality = Poor. R1:Core Times (min:sec) 9.5-10.5 ft (3:50) 10.5-11.5 ft (2:00) 11.5-12.5 ft (1:46) 12.5-13.5 ft (1:14) 13.4-14.5 ft (1:28) 80% Recovery.	GTX #311468 UTC qp = 17, 030 psi
10											
								301.1		— 14.5	
15										**Bottom of Exploration at 14.5 feet below ground surface.**	
20											
25											

Remarks:

Autohammer SN 295792 Calibrated 7/29/2019.
bgs = below ground surface.
Reinforced concrete deck was 12.75 inches thick. Lower 3 inches broke off.
29 feet of unsupported HW casing through bridge deck at Elev 344.6 feet to bottom of stream at Elev 315.6 feet.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Boring No.: BB-FCSR-102

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Farmington Falls Bridge #2273 carries Route 41 over Sandy River
Location: Farmington-Chesterville, Maine

Boring No.: BB-FCSR-103
WIN: 022296

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	343.3	**Auger ID/OD:**	5" Solid Stem
Operator:	J. Lee	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	J. McElroy	**Rig Type:**	CME 850 Track-mounted	**Hammer Wt./Fall:**	140 lbs/30 inch
Date Start/Finish:	1/21/2020	**Drilling Method:**	Cased Wash	**Core Barrel:**	NQ2 (2")
Boring Location:	Sta. 7+38.5, 15.8 ft Lt.	**Casing ID/OD:**	HW 4"/4.5"	**Water Level*:**	9.0 feet (after drilling)
Hammer Efficiency Factor:	0.801	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/12	0.00 - 2.00	14/18/12/10	30	40	SSA			Brown, frozen, dense, SAND, some silt, little gravel, (Fill).	
5	2D	24/6	5.00 - 7.00	2/2/3/3	5	7				Brown, moist, loose, SAND, some silt, trace gravel, (Fill).	
10	3D	24/17	10.00 - 12.00	7/14/15/19	29	39	78			Brown, moist, dense, SAND, some silt, little gravel, trace clay, (Fill).	GTX #551306 A-2-4, SM WC = 11.2%
							127				
							85			10 inch cobble. Rollercone through cobble and set NW casing.	
	MD	25/0	14.50 - 16.58	25-0"	- -		BX	328.8		14.5	
15	R1	60/21	14.60 - 19.60	RQD = 8%						Top of Bedrock at Elev 328.8 feet. Advanced by rollercone to 14.6 feet. R1:Bedrock: Dark grey, phyllitic SCHIST, some garnets, moderately hard, freshly weathered, joints are moderate to steeply dipping, very close to close, moderately healed with white quartz and calcite vein infilling, (Sangerville Formation - Anasagunticook Member). Rock Mass Quality = Very Poor. R1:Core Times (min:sec) 14.6-15.6 ft (2:30) 15.6-16.6 ft (1:45) 16.6-17.6 ft (3:15) 17.6-18.6 ft (4:00) 18.6-19.6 ft (5:33) 50% Recovery.	
20	R2	47/36	19.60 - 23.52	RQD = 35%						R2:Bedrock: Similar to R1 except joints are close to moderately close. Rock Mass Quality = Poor. R2:Core Times (min:sec) 19.6-20.6 ft (3:00) 20.6-21.6 ft (3:04) 21.6-22.6 ft (5:37) 22.6-23.5 ft (2:32) 38.7-39.7 ft (4:11) 77% Recovery.	GTX #311468 UCT qp = 9, 200 psi
								319.8		23.5	
25											

Remarks:

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±2 feet of frost.
Water level measured at end of drilling and prior to removal of casing.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Farmington Falls Bridge #2273 carries Route 41 over Sandy River **Location:** Farmington-Chesterville, Maine	**Boring No.:** BB-FCSR-103 **WIN:** 022296

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	343.3	**Auger ID/OD:**	5" Solid Stem
Operator:	J. Lee	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	J. McElroy	**Rig Type:**	CME 850 Track-mounted	**Hammer Wt./Fall:**	140 lbs/30 inch
Date Start/Finish:	1/21/2020	**Drilling Method:**	Cased Wash	**Core Barrel:**	NQ2 (2")
Boring Location:	Sta. 7+38.5, 15.8 ft Lt.	**Casing ID/OD:**	HW 4"/4.5"	**Water Level*:**	9.0 feet (after drilling)
Hammer Efficiency Factor:	0.801	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
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WC = Water Content, percent
LL = Liquid Limit
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G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										**Bottom of Exploration at 23.5 feet below ground surface.**	
30											
35											
40											
45											
50											

Remarks:

Autohammer SN 295792 Calibrated 7/29/2019.
bgs = below ground surface.
±2 feet of frost.
Water level measured at end of drilling and prior to removal of casing.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Farmington Falls Bridge #2273 carries Route 41 over Sandy River
Location: Farmington-Chesterville, Maine

Boring No.: BB-FCSR-201
WIN: 022296

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	323.0	**Auger ID/OD:**	N/A
Operator:	K. Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	M. St. Pierre	**Rig Type:**	Diedrich D-50 track-mounted	**Hammer Wt./Fall:**	140 lbs/30 inch
Date Start/Finish:	11/20/2020	**Drilling Method:**	Cased Wash	**Core Barrel:**	NQ2 (2")
Boring Location:	Sta. 6+60.4, 2.8 ft Rt.	**Casing ID/OD:**	NW 3"/3.5"	**Water Level*:**	Ground surface (after drilling)
Hammer Efficiency Factor:	0.995	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	23/14	0.00 - 1.92	8/9/26/50-5"	35	58				Brown, moist, very dense, SAND, some gravel, little silt, (Alluvium).	
	R1	17/12	1.92 - 3.34	RQD = 24%			NQ2	321.1		1.9	
										Top of Bedrock at Elev 321.1 feet.	
	R2	60/35	3.34 - 8.34	RQD = 27%						R1:Bedrock: Dark grey, phyllitic SCHIST, some garnets, moderately hard, freshly weathered, joints are moderate to steeply dipping, close to moderately close, moderately healed with white quartz and calcite vein infilling, (Sangerville Formation - Anasagunticook Member). Rock Mass Quality = Poor. R1:Core Times (min:sec) 1.9-2.2 ft (1:14) 2.2-3.2 ft (3:18) 3.2-3.3 ft (0:27) 12.5-13.5 ft (1:14) 13.4-14.5 ft (1:28) 80% Recovery.	
5											
	R3	50/50	8.34 - 12.51	RQD = 72%						R2:Bedrock: Similar to R1. Rock Mass Quality = Poor. R2:Core Times (min:sec) 3.3-4.3 ft (3:34) 4.3-5.3 ft (2:31) 5.3-6.3 ft (2:47) 6.3-7.3 ft (3:13) 7.3-8.3 ft (2:09) 58% Recovery.	
10											
	R4	26/23	12.51 - 14.68	RQD = 29%						R3:Bedrock: Similar to R2. Rock Mass Quality = Fair. R3:Core Times (min:sec) 8.3-9.3 ft (2:13) 9.3-10.3 ft (2:31) 10.3-11.3 ft (2:25) 11.3-12.3 ft (2:44) 12.3-12.5 ft (0:26) 100% Recovery.	
15								308.3		R4:Bedrock: Similar to R3. Rock Mass Quality = Poor. R4:Core Times (min:sec) 12.5-13.5 ft (2:12) 13.5-14.5 ft (1:55) 14.5-14.7 ft (0:32) 88% Recovery.	
										14.7	
										Bottom of Exploration at 14.7 feet below ground surface.	
20											
25											

Remarks:

Autohammer SN 367 Calibrated 9/9/2020.
bgs = below ground surface.
Reinforced concrete deck was 11 inches thick.
21.7 feet of unsupported NW casing through bridge deck at Elev 344.7 feet to bottom of stream at Elev 323.0 feet.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Boring No.: BB-FCSR-201

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Farmington Falls Bridge #2273 carries Route 41 over Sandy River
Location: Farmington-Chesterville, Maine

Boring No.: BB-FCSR-202
WIN: 022296

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	315.7	**Auger ID/OD:**	N/A
Operator:	K. Hanscom	**Datum:**	NAVD88	**Sampler:**	Standard Split-Spoon
Logged By:	M. St. Pierre	**Rig Type:**	Diedrich D-50 track-mounted	**Hammer Wt./Fall:**	140 lbs/30 inch
Date Start/Finish:	11/20/2020 - 11/21/2020	**Drilling Method:**	Cased Wash	**Core Barrel:**	NQ2 (2")
Boring Location:	Sta. 4+74.5, 13.3 ft Rt.	**Casing ID/OD:**	HW 4"/4.5" NW 3"/3.5"	**Water Level*:**	Level of Sandy River
Hammer Efficiency Factor:	0.995	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/17	0.00 - 2.00	21/33/11/12	44	73	116	315.0		Grey and brown, saturated, very dense, GRAVEL, some sand, trace silt, (Alluvium). 0.7	
							87			Grey, saturated, very stiff, SILT, little clay, with fine sand partings, (Alluvium).	
							104				
							189				
							232				
5	2D	9/5	5.00 - 5.75	67/50-3"	- -		193	310.0		Grey, wet, hard, SILT, little sand, little clay, (Alluvium). 5.7	
	R1	14/12	6.50 - 7.67				NQ2			Advanced by rock core through ±2 foot boulder.	
	R2	28/11	7.70 - 10.03								
10	R3	6/0	10.00 - 10.50					305.2		Cuttings in core barrel: Grey, wet, SAND, some silt, little gravel, (Glacial Till). 10.5	
										Bottom of Exploration at 10.5 feet below ground surface. DA suggests bedrock at 10.5 feet.	
15											
20											
25											

Remarks:

Autohammer SN 367 Calibrated 9/9/2020.
bgs = below ground surface. Reinforced concrete deck was 12 inches thick.
29.0 feet of unsupported casing through bridge deck at Elev 344.7 feet to bottom of stream at Elev 315.7 feet. Sandy River 5.2 feet deep at boring location on 11/20/2020.
Casing broken and not observed upon arriving on site 11/21/2020. Anticipate ±10 feet of casing abandoned in ground.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Boring No.: BB-FCSR-202

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Farmington Falls Bridge #2273 carries Route 41 over Sandy River
Location: Farmington-Chesterville, Maine

Boring No.: BB-FCSR-202A
WIN: 022296

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	315.5	**Auger ID/OD:**	N/A
Operator:	K. Hanscom	**Datum:**	NAVD88	**Sampler:**	N/A
Logged By:	N. Strout	**Rig Type:**	Diedrich D-50 track-mounted	**Hammer Wt./Fall:**	N/A
Date Start/Finish:	12/9/2020	**Drilling Method:**	Cased Wash	**Core Barrel:**	N/A
Boring Location:	Sta. 4+76.2, 15.4 ft Rt.	**Casing ID/OD:**	HW 4"/4.5"	**Water Level*:**	Level of Sandy River
Hammer Efficiency Factor:	0.995	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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MD = Unsuccessful Split Spoon Sample Attempt
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MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							HW			Advanced HW casing to 10.2 feet bgs. Soils similar to boring BB-FCSR-202 from 0 to 10.2 feet.	
5											
10								305.3		10.2 **Bottom of Exploration at 10.2 feet below ground surface.** Hole abandoned. Encountered probable casing from BB-FCSR-202.	
15											
20											
25											

Remarks:

Autohammer SN 367 Calibrated 9/9/2020.
bgs = below ground surface.
Reinforced concrete deck was 12 inches thick.
28.7 feet of unsupported casing through bridge deck at Elev 344.2 feet to bottom of stream at Elev 315.5 feet.
Sandy River 4.8 feet deep at boring location on 12/9/2020.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Boring No.: BB-FCSR-202A

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	**Project:** Farmington Falls Bridge #2273 carries Route 41 over Sandy River **Location:** Farmington-Chesterville, Maine	**Boring No.:** BB-FCSR-202B **WIN:** 022296

Driller:	S. W. Cole Explorations, LLC	**Elevation (ft.)**	315.9	**Auger ID/OD:**	N/A
Operator:	K. Hanscom	**Datum:**	NAVD88	**Sampler:**	N/A
Logged By:	N. Strout	**Rig Type:**	Diedrich D-50 track-mounted	**Hammer Wt./Fall:**	
Date Start/Finish:	12/9/2020	**Drilling Method:**	Cased Wash	**Core Barrel:**	NQ2 (2")
Boring Location:	Sta. 4+72.6, 15.2 ft Rt.	**Casing ID/OD:**	HW 4"/4.5" NW 3"/3.5"	**Water Level*:**	Level of Sandy River
Hammer Efficiency Factor: 0.995		**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample Attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample Attempt
V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt
R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							HW			Advanced HW casing to 9.9 feet bgs. Soils similar to boring BB-FCSR-202 from 0 to 9.9 feet.	
5											
	R1	41/29	9.90 - 13.32	RQD = 10%			NQ2				
10								306.0		9.9 Top of Bedrock at Elev 306.0 feet. R1:Bedrock: Dark grey, phyllitic SCHIST, some garnets, moderately hard, freshly weathered, joints are moderate to steeply dipping, close to moderately close, moderately healed with white quartz and calcite vein infilling, (Sangerville Formation - Anasagunticook Member). Rock Mass Quality = Very Poor. R1:Core Times (min:sec) 9.9-10.9 ft (1:59) 10.9-11.9 ft (2:38) 11.9-12.9 ft (2:41) 12.9-13.3 ft (1:57) 71% Recovery.	
	R2	55/49	13.32 - 17.90	RQD = 40%						R2:Bedrock: Similar to R1. Rock Mass Quality = Poor. R2:Core Times (min:sec) 13.3-13.9 ft (1:03) 13.9-14.9 ft (1:48) 14.9-15.9 ft (1:34) 15.9-16.9 ft (1:50) 16.9-17.9 ft (1:46) 89% Recovery.	
15											
	R3	24/24	17.90 - 19.90	RQD = 50%						R3:Bedrock: Similar to R2. Rock Mass Quality = Poor. R3:Core Times (min:sec) 17.9-18.9 ft (1:27) 18.9-19.9 ft (1:22) 100% Recovery.	
20								296.0		19.9 **Bottom of Exploration at 19.9 feet below ground surface.**	
25											

Remarks:

Autohammer SN 367 Calibrated 9/9/2020.
bgs = below ground surface.
Reinforced concrete deck was 12 inches thick.
29.1 feet of unsupported casing through bridge deck at Elev 345.0 feet to bottom of stream at Elev 315.9 feet.
Sandy River 4.6 feet deep at boring location on 12/9/2020.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Boring No.: BB-FCSR-202B

S.W.COLE
ENGINEERING, INC.

APPENDIX D
Laboratory Test Results

Client:	S.W. Cole Engineering, Inc.				
Project:	Farmington Falls Br. 2273 Replace.				
Location:	Farmington-Chesterville, ME			Project No:	GTX-311468
Boring ID:	---	Sample Type:	---	Tested By:	ckg
Sample ID:	---	Test Date:	03/21/20	Checked By:	bfs
Depth :	---	Test Id:	551310		

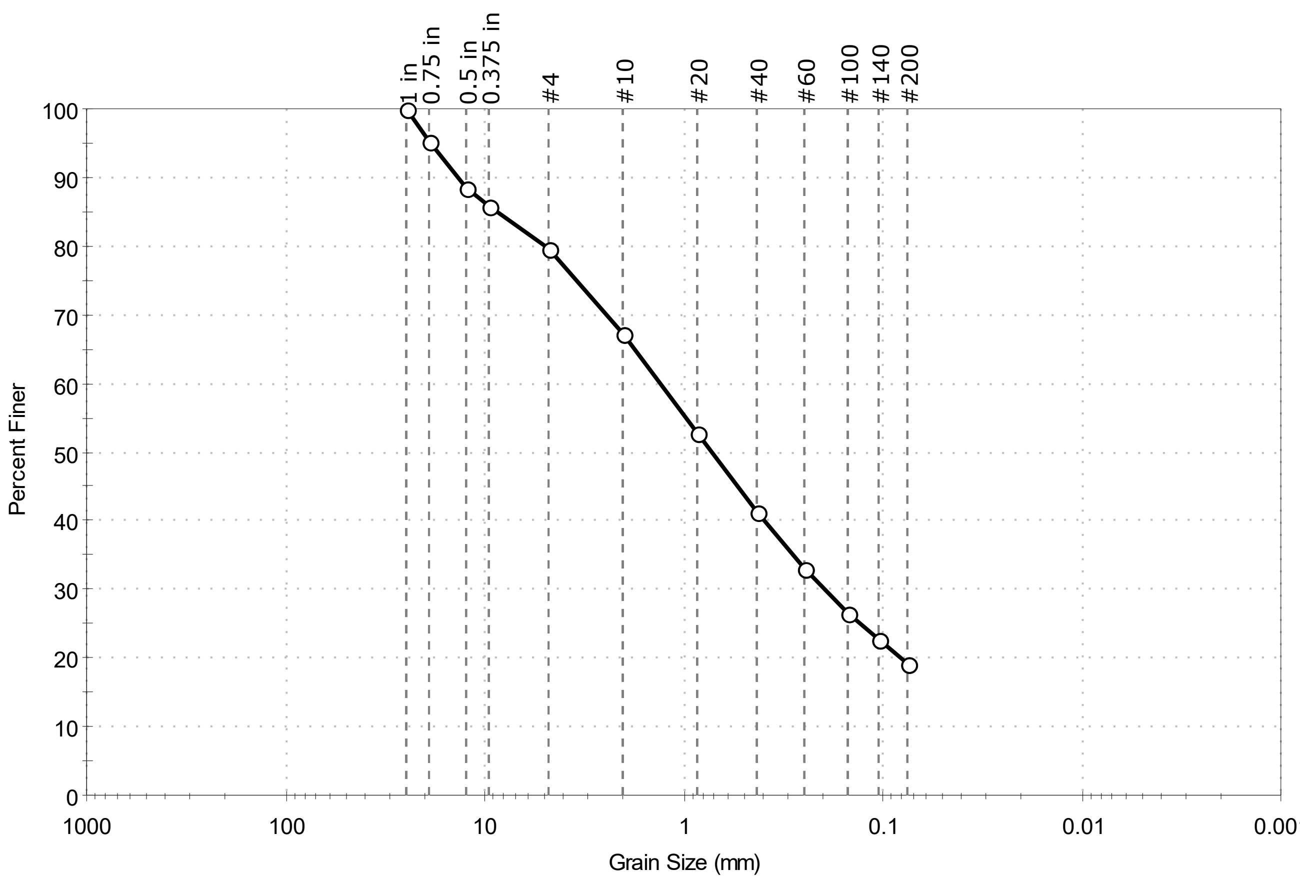
Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content,%
BB-FCSR-101	2D	5-7	Moist, dark grayish brown silty sand with gravel	9.8
BB-FCSR-101	3D	10-12	Moist, dark grayish brown sandy silt	28.0
BB-FCSR-101	5D	25-27	Moist, dark greenish gray clay	26.4
BB-FCSR-103	3D	10-12	Moist, dark grayish brown silty sand with gravel	11.2

Notes: Temperature of Drying : 110º Celsius

Client:	S.W. Cole Engineering, Inc.		
Project:	Farmington Falls Br. 2273 Replace.		
Location:	Farmington-Chesterville, ME	Project No:	GTX-311468
Boring ID: BB-FCSR-101	Sample Type: jar	Tested By: ckg	
Sample ID: 2D	Test Date: 03/25/20	Checked By: bfs	
Depth : 5-7	Test Id: 551303		
Test Comment:	---		
Visual Description:	Moist, dark grayish brown silty sand with gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	20.2	60.6	19.2

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	95		
0.5 in	12.50	88		
0.375 in	9.50	86		
#4	4.75	80		
#10	2.00	67		
#20	0.85	53		
#40	0.42	41		
#60	0.25	33		
#100	0.15	26		
#140	0.11	23		
#200	0.075	19		

Coefficients

D_{85} =8.7573 mm　D_{30} =0.1979 mm

D_{60} =1.2984 mm　D_{15} =N/A

D_{50} =0.7134 mm　D_{10} =N/A

C_u =N/A　C_c =N/A

Classification

ASTM　N/A

AASHTO　Stone Fragments, Gravel and Sand (A-1-b (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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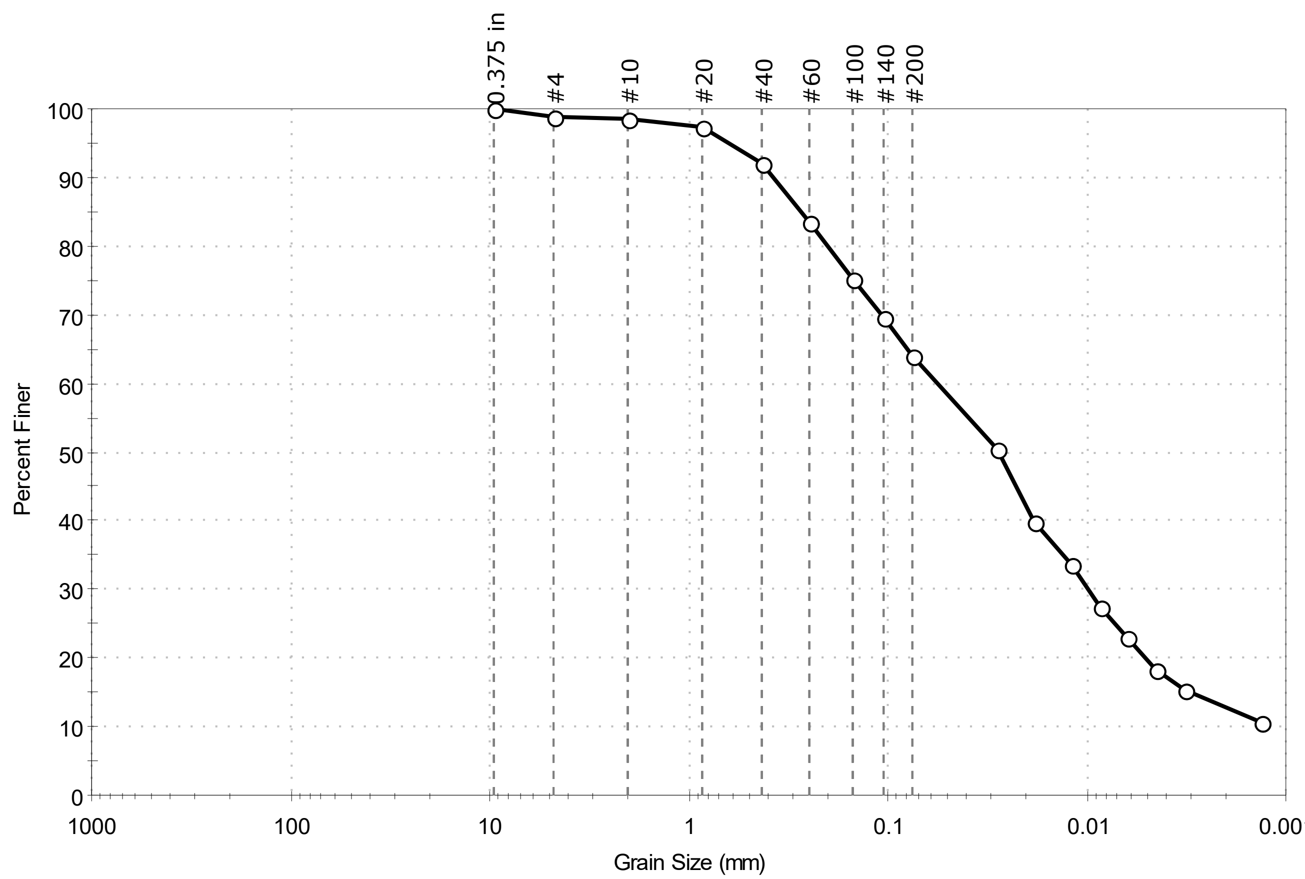
GeoTesting EXPRESS

Client: S.W. Cole Engineering, Inc.			
Project: Farmington Falls Br. 2273 Replace.			
Location: Farmington-Chesterville, ME		Project No:	GTX-311468
Boring ID: BB-FCSR-101	Sample Type: jar	Tested By:	ckg
Sample ID: 3D	Test Date: 03/25/20	Checked By:	bfs
Depth : 10-12	Test Id: 551305		

Test Comment: ---
Visual Description: Moist, dark grayish brown sandy silt
Sample Comment: ---

Particle Size Analysis - ASTM D6913/D7928



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	1.2	34.7	64.1

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	99		
#10	2.00	98		
#20	0.85	97		
#40	0.42	92		
#60	0.25	84		
#100	0.15	75		
#140	0.11	70		
#200	0.075	64		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0279	50		
---	0.0182	40		
---	0.0119	34		
---	0.0085	28		
---	0.0062	23		
---	0.0045	18		
---	0.0032	15		
---	0.0013	11		

Coefficients

D_{85} =0.2736 mm D_{30} =0.0098 mm
D_{60} =0.0558 mm D_{15} =0.0030 mm
D_{50} =0.0274 mm D_{10} =N/A
C_u =N/A C_c =N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---
Sand/Gravel Hardness : ---
Dispersion Device : Apparatus A - Mech Mixer
Dispersion Period : 1 minute
Est. Specific Gravity : 2.65
Separation of Sample: #200 Sieve

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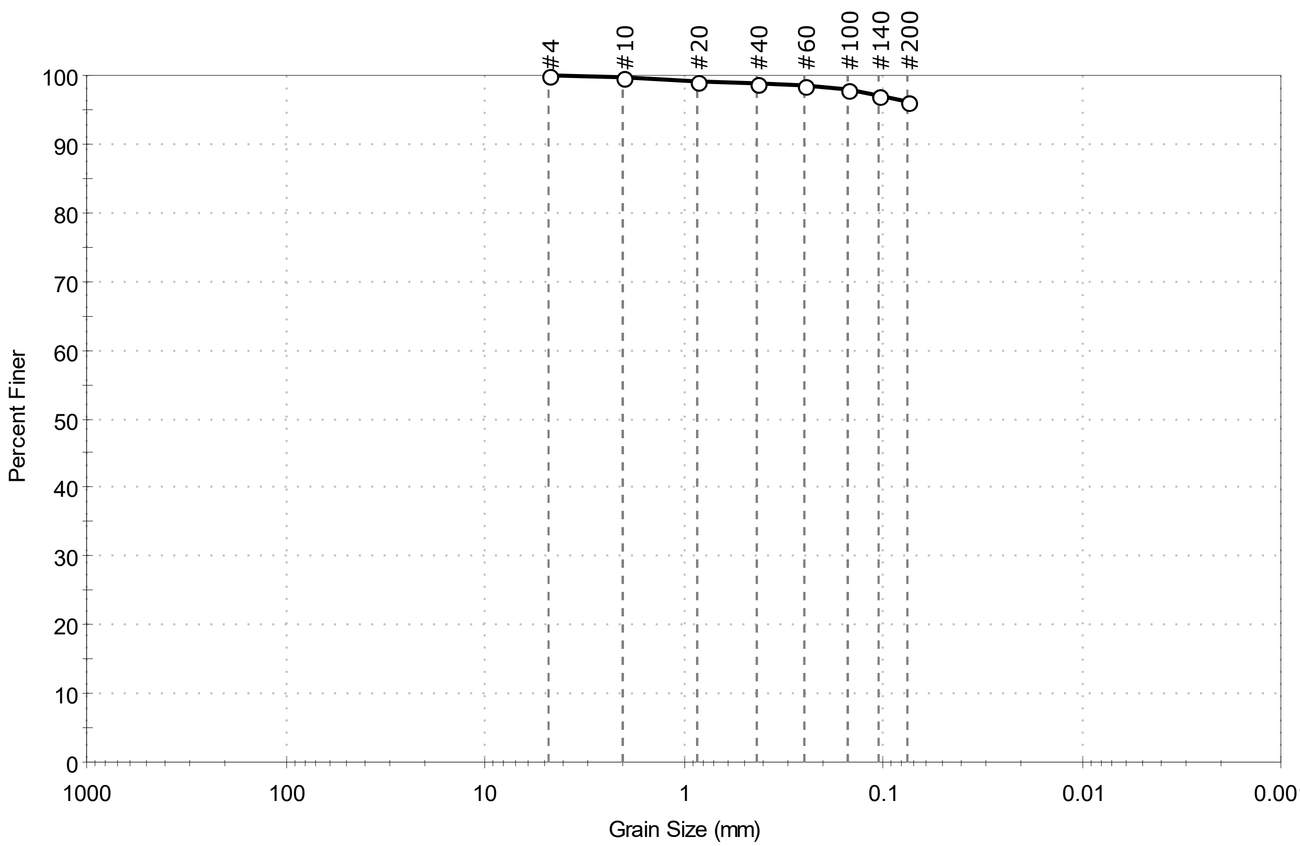
GeoTesting EXPRESS

Client:	S.W. Cole Engineering, Inc.		
Project:	Farmington Falls Br. 2273 Replace.		
Location:	Farmington-Chesterville, ME	Project No:	GTX-311468
Boring ID: BB-FCSR-101	Sample Type: jar	Tested By:	ckg
Sample ID: 5D	Test Date: 03/25/20	Checked By:	bfs
Depth : 25-27	Test Id: 551304		
Test Comment:	---		
Visual Description:	Moist, dark greenish gray clay		
Sample Comment:	---		

Particle Size Analysis - ASTM D6913



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	0.0	3.9	96.1

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	99		
#40	0.42	99		
#60	0.25	98		
#100	0.15	98		
#140	0.11	97		
#200	0.075	96		

Coefficients

D_{85} =N/A	D_{30} =N/A
D_{60} =N/A	D_{15} =N/A
D_{50} =N/A	D_{10} =N/A
C_u =N/A	C_c =N/A

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

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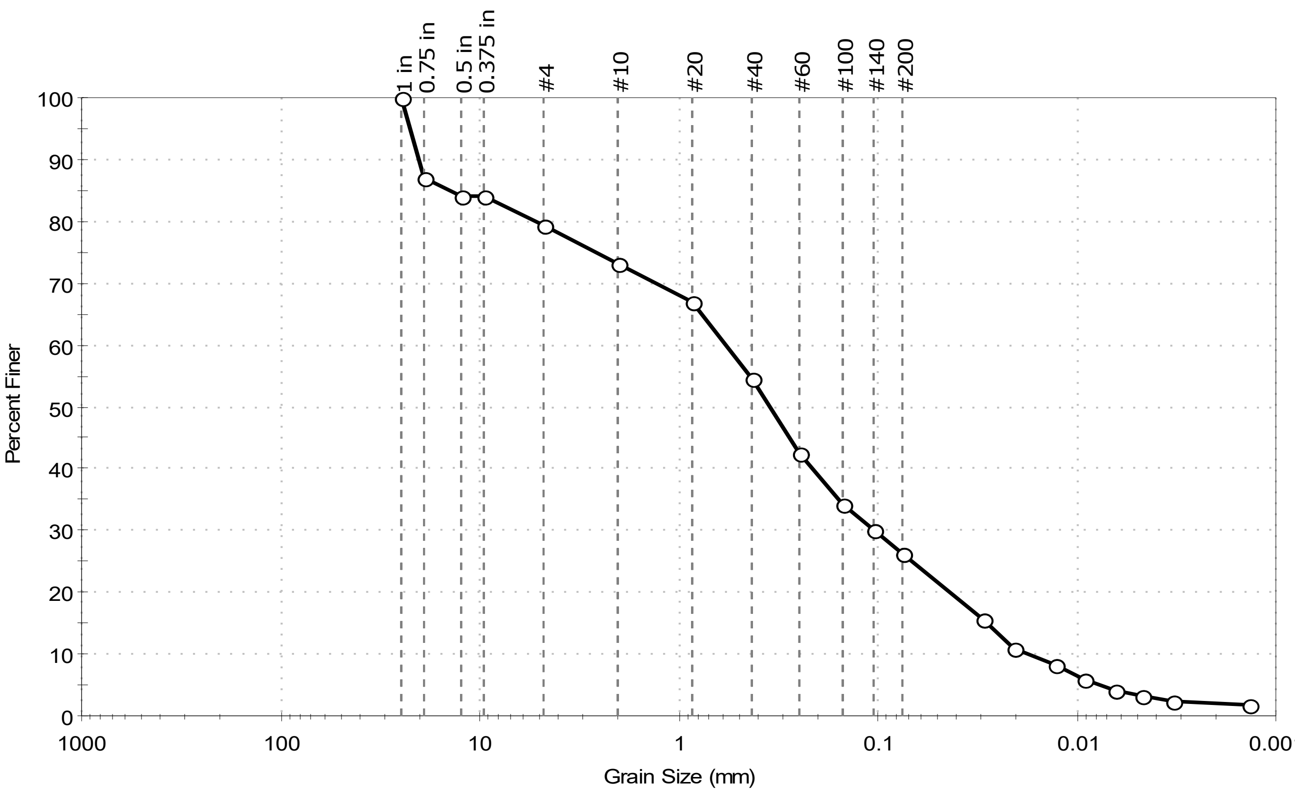
Client:	S.W. Cole Engineering, Inc.		
Project:	Farmington Falls Br. 2273 Replace.		
Location:	Farmington-Chesterville, ME	Project No:	GTX-311468
Boring ID: BB-FCSR-103	Sample Type: jar	Tested By:	ckg
Sample ID: 3D	Test Date: 03/26/20	Checked By:	bfs
Depth : 10-12	Test Id: 551306		
Test Comment:	---		
Visual Description:	Moist, dark grayish brown silty sand with gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D6913/D7928



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	20.6	53.2	26.2

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	87		
0.5 in	12.50	84		
0.375 in	9.50	84		
#4	4.75	79		
#10	2.00	73		
#20	0.85	67		
#40	0.42	55		
#60	0.25	42		
#100	0.15	34		
#140	0.11	30		
#200	0.075	26		
Hydrometer	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0297	16		
---	0.0205	11		
---	0.0128	8		
---	0.0093	6		
---	0.0064	4		
---	0.0048	3		
---	0.0033	2		
---	0.0014	2		

Coefficients

D_{85} =14.5037 mm | D_{30} =0.1041 mm

D_{60} =0.5729 mm | D_{15} =0.0281 mm

D_{50} =0.3474 mm | D_{10} =0.0178 mm

C_u =32.185 | C_c =1.063

Classification

ASTM N/A

AASHTO Silty Gravel and Sand (A-2-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Dispersion Device : Apparatus A - Mech Mixer

Dispersion Period : 1 minute

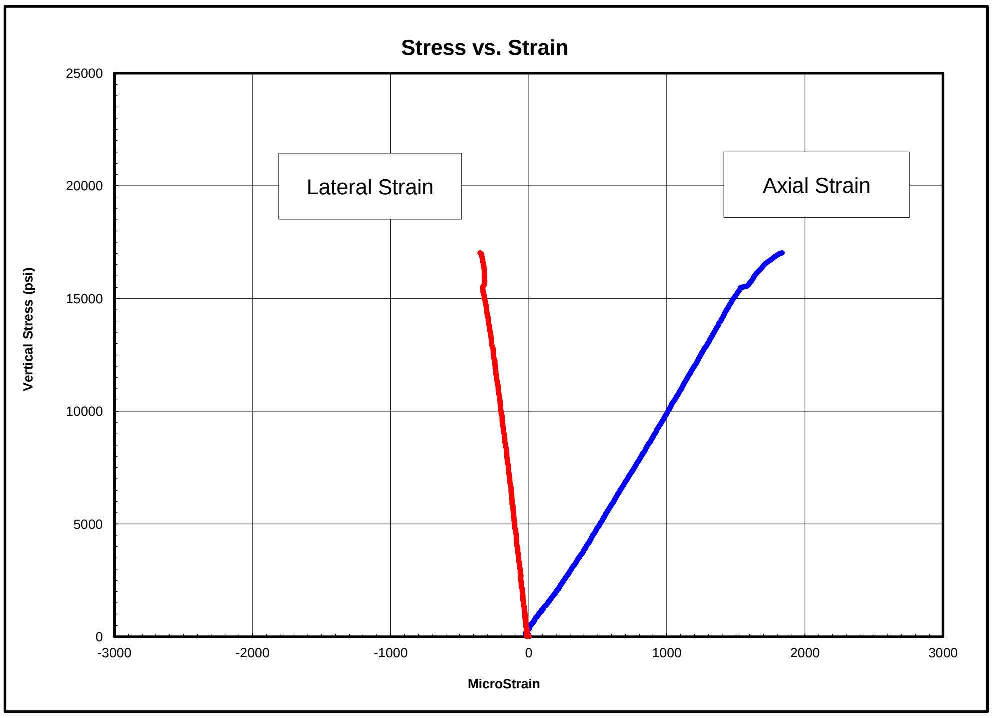
Est. Specific Gravity : 2.65

Separation of Sample: #200 Sieve

printed 3/26/2020 10:17:18 AM

Client:	S.W.Cole Engineering, Inc.
Project Name:	Farmington Falls Br. 2273 Replace
Project Location:	Farmington-Chesterville, ME
GTX #:	311468
Test Date:	3/25/2020
Tested By:	cmh
Checked By:	jsc
Boring ID:	BB-FCSR-102
Sample ID:	R1
Depth, ft:	9.55-9.91
Sample Type:	rock core
Sample Description:	See photographs Discontinuity failure

Compressive Strength and Elastic Moduli of Rock by ASTM D7012 - Method D



Peak Compressive Stress: 17,026 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1700-6200	9,560,000	0.18
6200-10800	10,200,000	0.21
10800-15300	10,500,000	0.26

Notes: Test specimen tested at the approximate as-received moisture content and at standard laboratory temperature.
The axial load was applied continuously at a stress rate that produced failure in a test time between 2 and 15 minutes.
Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed.
Calculations assume samples are isotropic, which is not necessarily the case.

GeoTesting EXPRESS

Client:	S.W. Cole Engineering, Inc.	Test Date:	3/20/2020
Project Name:	Farmington Falls Br. 2273 Replace	Tested By:	cmh/kdp
Project Location:	Farmington-Chesterville, ME	Checked By:	smd
GTX #:	311468		
Boring ID:	BB-FCSR-102		
Sample ID:	R1		
Depth:	9.55-9.91 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

BULK DENSITY

	1	2	Average	
Specimen Length, in:	4.20	4.20	4.20	
Specimen Diameter, in:	1.98	1.98	1.98	
Specimen Mass, g:	609.52			
Bulk Density, lb/ft^3	179	**Minimum Diameter Tolerence Met?**		YES
Length to Diameter Ratio:	2.1	**Length to Diameter Ratio Tolerance Met?**		YES

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:

Is the maximum gap $\leq$ 0.02 in.? NO

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? NO

END FLATNESS AND PARALLELISM (Procedure FP1)

END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00050	-0.00040	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00020	0.00020	0.00020
Diameter 2, in (rotated 90°)	-0.00030	-0.00030	-0.00020	-0.00020	-0.00010	-0.00010	0.00000	0.00000	0.00010	0.00010	0.00010	0.00010	0.00010	0.00020	0.00020

Difference between max and min readings, in:

0° = 0.00070 90° = 0.00050

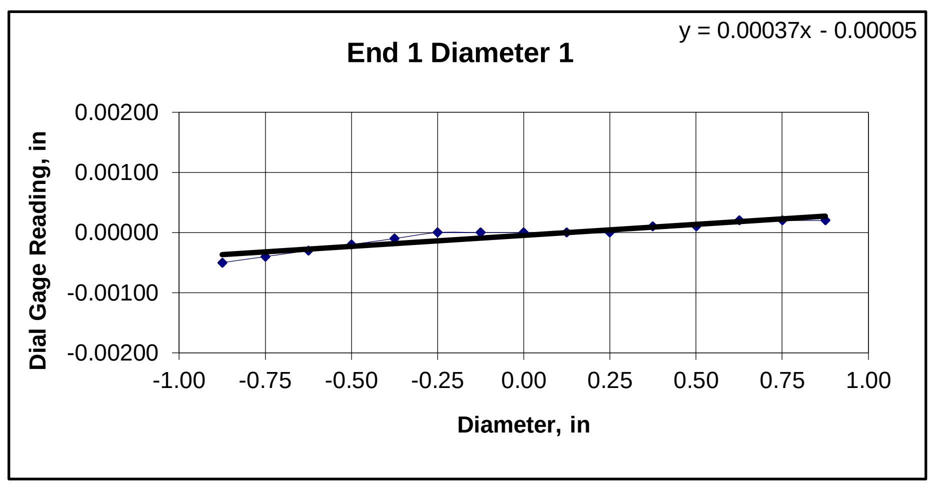
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00050	-0.00040	-0.00030	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010
Diameter 2, in (rotated 90°)	0.00020	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00010	-0.00020	-0.00030	-0.00030	-0.00040

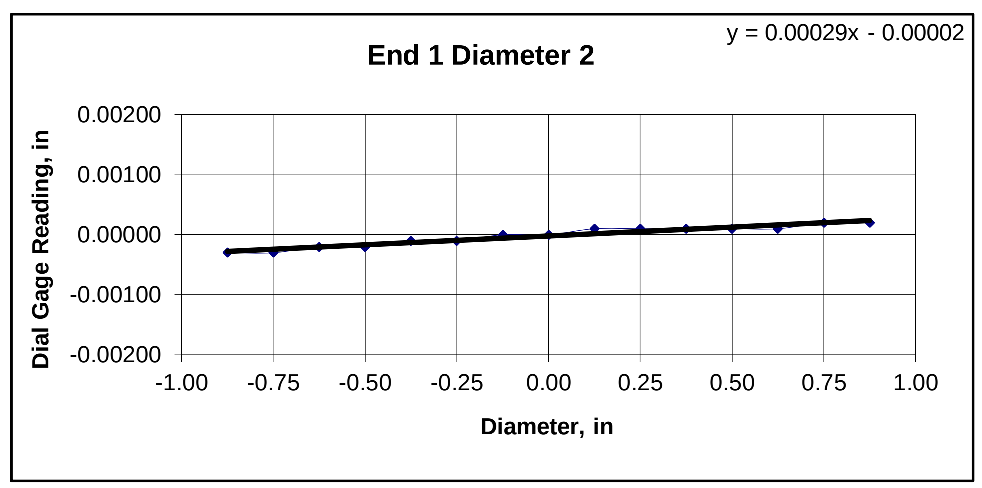
Difference between max and min readings, in:

0° = 0.0006 90° = 0.0006

Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00035

Flatness Tolerance Met? YES





DIAMETER 1

End 1:

Slope of Best Fit Line	0.00037
Angle of Best Fit Line:	0.02095

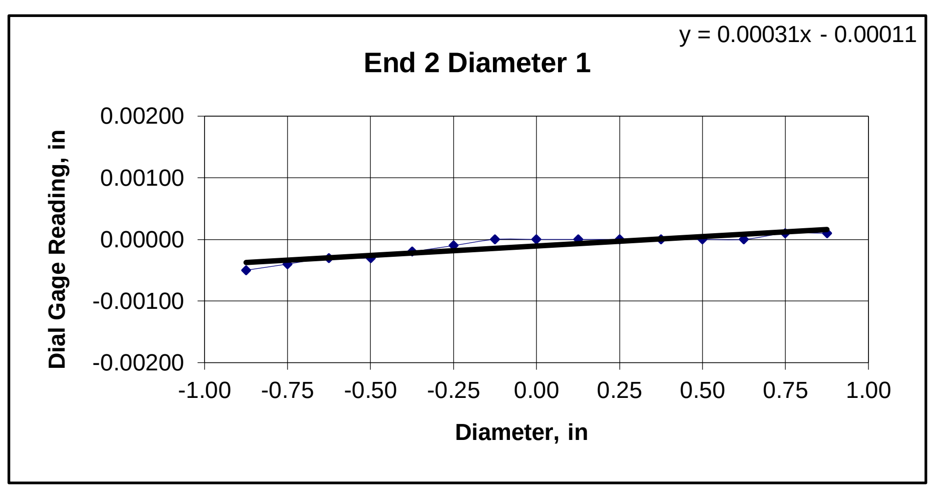
End 2:

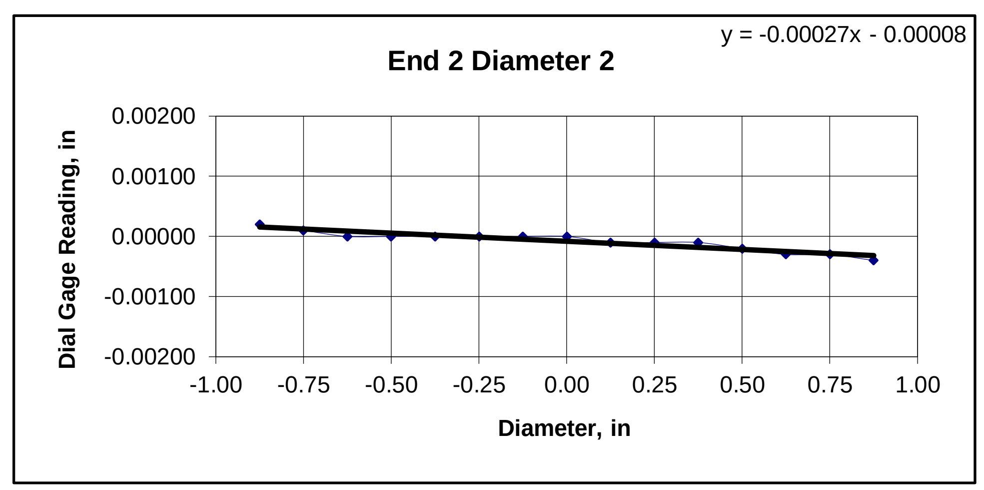
Slope of Best Fit Line	0.00031
Angle of Best Fit Line:	0.01752

Maximum Angular Difference: 0.00344

Parallelism Tolerance Met? YES

Spherically Seated





DIAMETER 2

End 1:

Slope of Best Fit Line	0.00029
Angle of Best Fit Line:	0.01686

End 2:

Slope of Best Fit Line	0.00027
Angle of Best Fit Line:	0.01555

Maximum Angular Difference: 0.00131

Parallelism Tolerance Met? YES

Spherically Seated

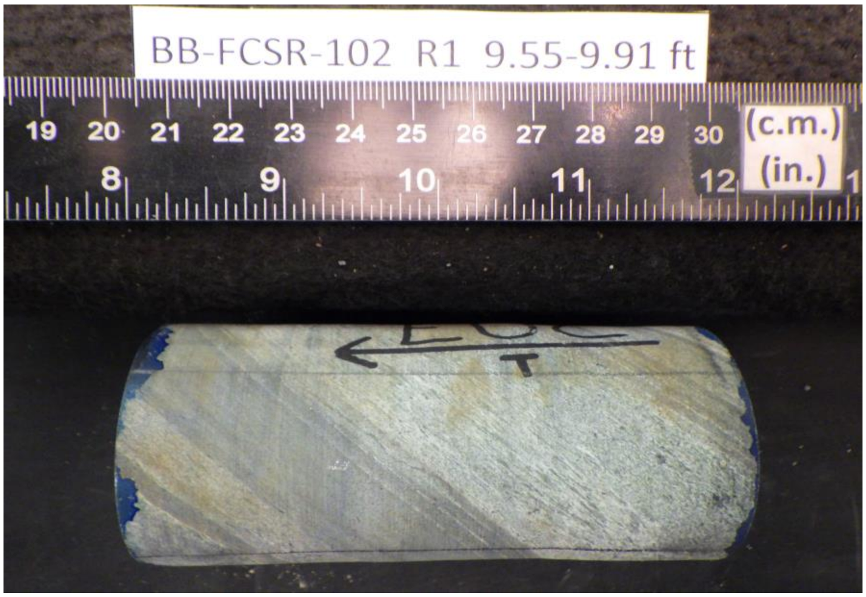
PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00070	1.980	0.00035	0.020	YES
Diameter 2, in (rotated 90°)	0.00050	1.980	0.00025	0.014	YES
END 2					
Diameter 1, in	0.00060	1.980	0.00030	0.017	YES
Diameter 2, in (rotated 90°)	0.00060	1.980	0.00030	0.017	YES

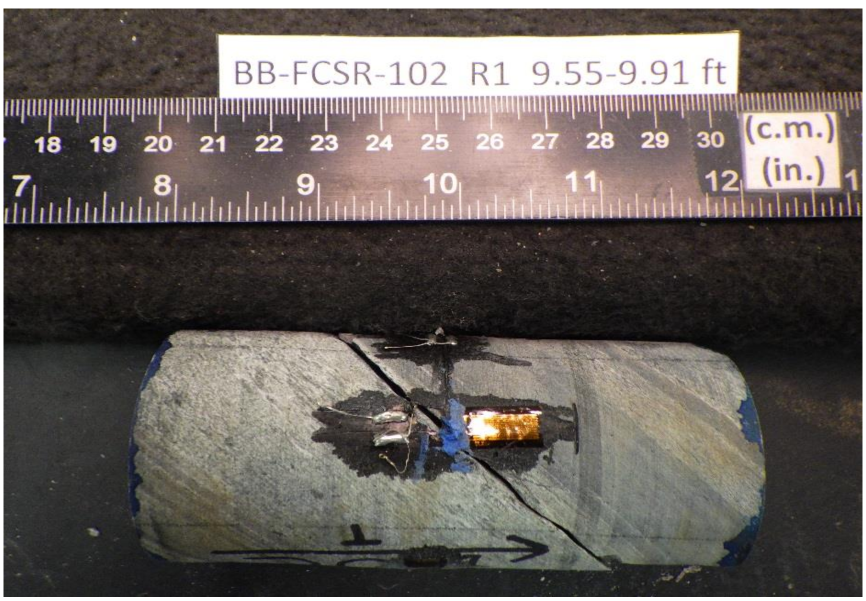
Maximum angle of departure must be $\leq$ 0.25°

Perpendicularity Tolerance Met? YES

Client:	S.W.Cole Engineering, Inc.
Project Name:	Farmington Falls Br. 2273 Replace
Project Location:	Farmington-Chesterville, ME
GTX #:	311468
Test Date:	3/25/2020
Tested By:	cmh
Checked By:	smd
Boring ID:	BB-FCSR-102
Sample ID:	R1
Depth, ft:	9.55-9.91



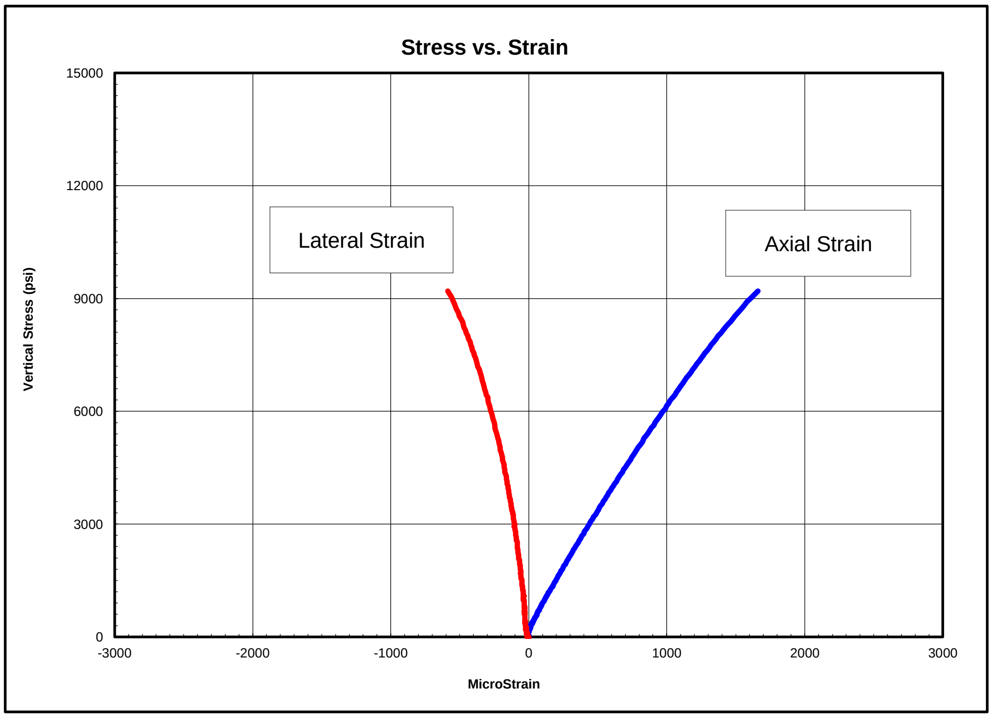
After cutting and grinding



After break

Client:	S.W. Cole Engineering, Inc.
Project Name:	Farmington Falls Br. 2273 Replace
Project Location:	Farmington-Chesterville, ME
GTX #:	311468
Test Date:	3/25/2020
Tested By:	cmh
Checked By:	jsc
Boring ID:	BB-FCSR-103
Sample ID:	R2
Depth, ft:	20.97-21.35
Sample Type:	rock core
Sample Description:	See photographs Intact material and discontinuity failure

Compressive Strength and Elastic Moduli of Rock by ASTM D7012 - Method D



Peak Compressive Stress: 9,198 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
900-3400	6,200,000	0.22
3400-5800	5,610,000	0.32
5800-8300	4,990,000	0.42

Notes: Test specimen tested at the approximate as-received moisture content and at standard laboratory temperature.
The axial load was applied continuously at a stress rate that produced failure in a test time between 2 and 15 minutes.
Young's Modulus and Poisson's Ratio calculated using the tangent to the line in the stress range listed.
Calculations assume samples are isotropic, which is not necessarily the case.

GeoTesting EXPRESS

Client:	S.W. Cole Engineering, Inc.	Test Date:	3/20/2020
Project Name:	Farmington Falls Br. 2273 Replace.	Tested By:	cmh/kdp
Project Location:	Farmington-Chesterville, ME	Checked By:	smd
GTX #:	311468		
Boring ID:	BB-FCSR-103		
Sample ID:	R2		
Depth:	20.97-21.35 ft		
Visual Description:	See photographs		

UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

BULK DENSITY

	1	2	Average	
Specimen Length, in:	4.37	4.37	4.37	
Specimen Diameter, in:	1.99	1.98	1.99	
Specimen Mass, g:	626.01			
Bulk Density, lb/ft^3	176	**Minimum Diameter Tolerence Met?**		**YES**
Length to Diameter Ratio:	2.2	**Length to Diameter Ratio Tolerance Met?**		**YES**

DEVIATION FROM STRAIGHTNESS (Procedure S1)

Maximum gap between side of core and reference surface plate:
Is the maximum gap $\leq$ 0.02 in.? YES

Maximum difference must be < 0.020 in.
Straightness Tolerance Met? **YES**

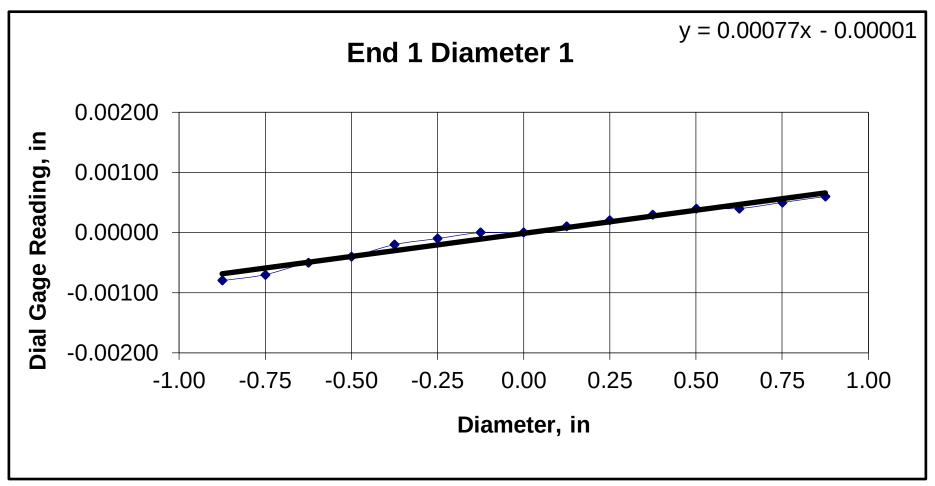
END FLATNESS AND PARALLELISM (Procedure FP1)

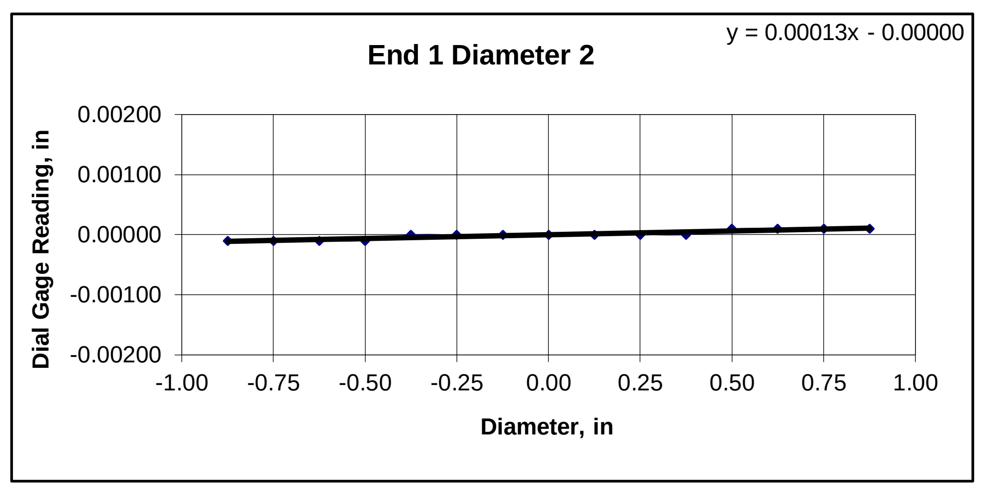
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00080	-0.00070	-0.00050	-0.00040	-0.00020	-0.00010	0.00000	0.00000	0.00010	0.00020	0.00030	0.00040	0.00040	0.00050	0.00060
Diameter 2, in (rotated 90°)	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010	0.00010

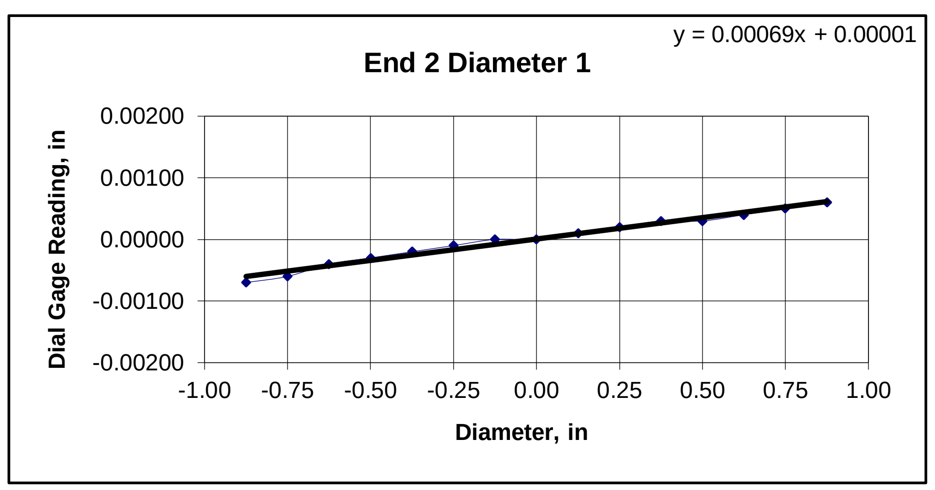
Difference between max and min readings, in:
0° = 0.00140 90° = 0.00020

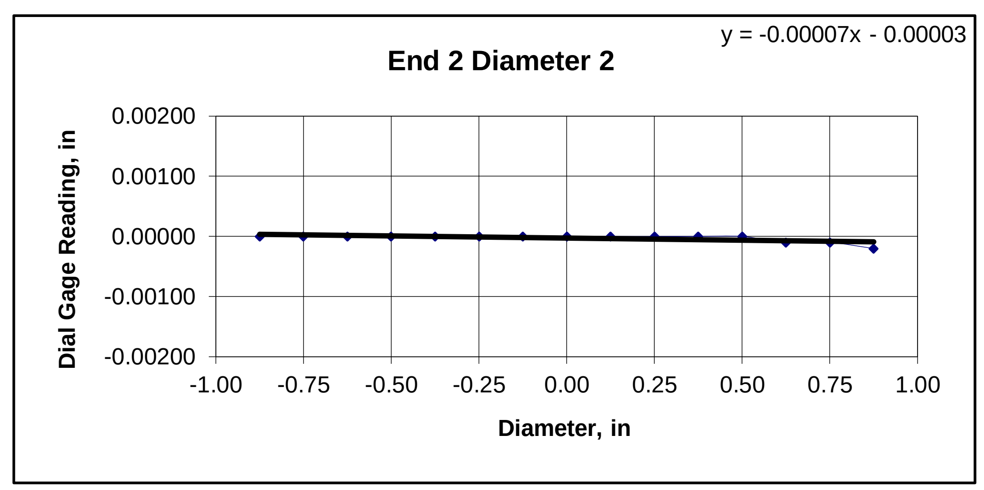
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00070	-0.00060	-0.00040	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00010	0.00020	0.00030	0.00030	0.00040	0.00050	0.00060
Diameter 2, in (rotated 90°)	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00020

Difference between max and min readings, in:
0° = 0.0013 90° = 0.0002
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00070
Flatness Tolerance Met? **YES**









DIAMETER 1

End 1:
Slope of Best Fit Line 0.00077
Angle of Best Fit Line: 0.04404

End 2:
Slope of Best Fit Line 0.00069
Angle of Best Fit Line: 0.03978

Maximum Angular Difference: 0.00426

Parallelism Tolerance Met? **YES**
Spherically Seated

DIAMETER 2

End 1:
Slope of Best Fit Line 0.00013
Angle of Best Fit Line: 0.00720

End 2:
Slope of Best Fit Line 0.00007
Angle of Best Fit Line: 0.00409

Maximum Angular Difference: 0.00311

Parallelism Tolerance Met? **YES**
Spherically Seated

PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00140	1.985	0.00071	0.040	YES
Diameter 2, in (rotated 90°)	0.00020	1.985	0.00010	0.006	YES
END 2					
Diameter 1, in	0.00130	1.985	0.00065	0.038	YES
Diameter 2, in (rotated 90°)	0.00020	1.985	0.00010	0.006	YES

Maximum angle of departure must be $\leq$ 0.25°

Perpendicularity Tolerance Met? **YES**

Client:	S.W.Cole Engineering, Inc.
Project Name:	Farmington Falls Br. 2273 Replace
Project Location:	Farmington-Chesterville, ME
GTX #:	311468
Test Date:	3/25/2020
Tested By:	cmh
Checked By:	smd
Boring ID:	BB-FCSR-103
Sample ID:	R2
Depth, ft:	20.97-21.35

After cutting and grinding

After break

S.W.COLE ENGINEERING, INC.

APPENDIX E
Geophysical Test Report

Seismic Refraction
Geophysical Investigation
Farmington Falls Bridge over Sandy River

Farmington, Maine

Prepared for

S.W. Cole Engineering, Inc

December, 2020

We Save Structures™

NDTCorporation.com

NDT CORPORATION
Concrete & Geophysical Testing

NDT CORPORATION
153 Clinton Road, Sterling, MA 01564
Main: 978-563-1327 | Fax: 978-563-1340
eMail: Info@NDTCorporation.com

December 3, 2020

Michael St. Pierre, P.E.
Senior Geotechnical Engineer
S. W. Cole Engineering, Inc.
26 Coles Crossing Drive, Sidney, ME 04330

Subject: Report for Geophysical Testing to determine bedrock elevations for Farmington Falls Bridge over Sandy River Replacement Project in Farmington, Maine

Dear Mr. St. Pierre:

NDT Corporation conducted a seismic refraction survey at the existing Farmington Falls Bridge crossing the Sandy River in Farmington, Maine. Fieldwork was conducted on November 11th, 2020. The seismic refraction data was collected along four (4) 100 foot lines of coverage in the river, two (2) 100 foot lines along the South approach and one (1) 100 foot line along the North approach with the objective to profile the top of rock along the proposed replacement bridge alignment. The results of this testing will assist S.W. Cole Engineering with their bridge replacement designs.

We thank you for the opportunity to perform this work and look forward to being of service to you in the future. If you have any questions or require additional information, call the undersigned at 978-573-1327.

Sincerely,
NDT Corporation
William Horne

William Horne

TABLE OF CONTENTS

1.0 SUMMARY:

Seismic Refraction results indicate that a 14,000+/- ft/sec bedrock is overlain by 1,500 ft/sec soils/fills over 2,500 ft/sec dense soils/ablation till layer, both of which when saturated indicate a 5,000 ft/sec. The top of rock profile along the center of the proposed alignment (Figure 3) is approximately as follows:

- Sta. 3+50 to 4+50 the top of rock dips from elevation 330-320+/- to elevation 311+/-
- Sta. 4+50 to 5+75/6+00 top of rock remains relatively flat at elevation 310-312+/-
- Sta. 5+75/6+00 to 7+50 top of rock slopes up to elevation 326+/-
- Sta. 7+50 to 8+00 top of rock dips down to elevation 316+/-

2.0 INTRODUCTION AND PURPOSE:

NDT Corporation conducted a seismic refraction survey at the existing Farmington Falls Bridge crossing the Sandy River in Farmington, Maine. Fieldwork was conducted on November 11th, 2020. The seismic refraction data was collected along four (4) 100 foot lines of coverage in the river, two (2) 100 foot lines along the South approach and one (1) 100 foot line along the North approach with the objective to profile the top of rock along the proposed replacement bridge alignment. The results of this testing will assist S.W. Cole Engineering with their bridge replacement designs.

3.0 LOCATION AND SURVEY CONTROL

The general location of the project area is shown in Figure 1; also shown on Figure 1 is the seismic line locations overlain on a satellite photo of the area. Seismic lines of data were collected along seven (7) 100 foot long lines four (4) were located in the streambed and along the north and south approaches along RT 156/41 – Farmington Falls Bridge crossing the Sandy River in Farmington, Maine. The seismic lines of coverage are plotted Figure 1 and on the proposed project plan (provide to NDT by S.W. Cole) in Figure 2.

Elevation data (obtained from plans provided by S. W. Cole) is referenced to top of road surface ELEV 345. The river/water surface at the time of the survey was measured to be 25 feet below the top of roadway at ELEV 320.

4.0 METHODS OF INVESTIGATION

4.1 SEISMIC REFRACTION:

Seismic refraction data was acquired with a 12 channel system with 5 and 10 foot geophone spacing and seismic energy generated approximately every 50 feet with a "seisgun" or sledge hammer energy source. Seismic Refraction utilizes the natural energy transmitting properties of the soils and rocks and is based on the principle that the velocity at which seismic waves travel through the earth is a function of the physical properties (elastic moduli and Poisson's ratio) of the materials. Refracted compressional wave data are used to evaluate material types and thickness, profile top of bedrock, and to determine the approximate depth to layer interfaces. A more complete discussion of the seismic refraction survey method is included in Appendix 1.

The seismic refraction data were interpreted using the critical distance method. Delayed bedrock wave arrivals were used to portray the bedrock surface more accurately between critical distance depth calculations. The delayed arrivals at individual geophone locations are an indicator of variability in the rock surface. Delayed arrivals indicate thicker overburden over the bedrock. Variations of 3-5 feet are not accurately profiled, particularly in shallow (less than 10 feet deep) bedrock areas.

Overburden with a 1,000-1,500+/- ft/sec velocity is consistent with normally consolidated soils/sands/fill material typical of natural soils, fluvial deposits, and/or construction fill. Till with a 2,000 to 2,600 ft/sec velocity value is consistent with unstratified glacial drift or ground moraine. These tills are typically deposited by receding glaciers consisting of an admixture of clays, sands and gravels with occasional and sometimes frequent boulders associated with an ablation till. Overburden and Till layers with "dry" velocities of less than 5,000 ft/sec are indistinguishable when saturated and assume the velocity of water 5,000+/- ft/sec when saturated. These overburden/soils/tills are susceptible to scour.

Bedrock velocities of less than 10,000 ft/sec are indicative of highly weathered and/or fractured rock typical of sedimentary and low-grade metamorphic rocks such as shales, silt stones and schists. Bedrock with a velocity of 10,000 to 15,000 ft/sec is indicative of competent bedrock that will require drilling and blasting for removal. This velocity range is typical of competent sedimentary and metamorphic rocks such as sandstones, limestones, schists, and gneisses. Bedrock velocities greater than 15,000 ft/sec are indicative of massive bedrock typical in metamorphic and igneous rocks such as gneisses, granites and basalts.

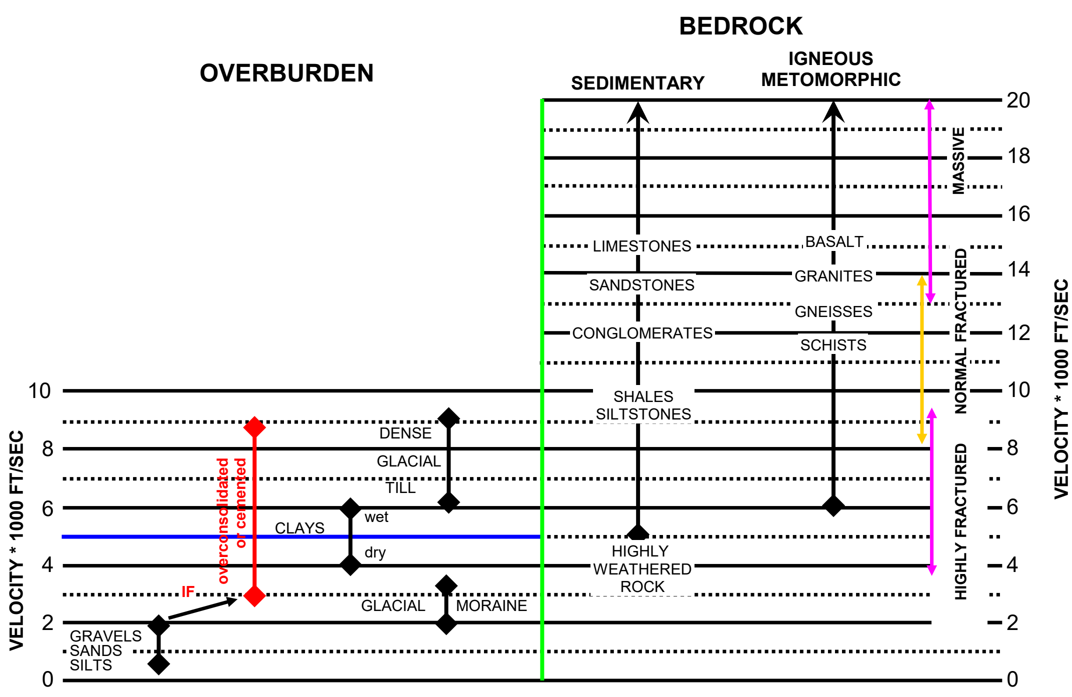


Chart 1: Seismic Velocity by Soil/Rock Types

5.0 RESULTS:

The water surface was measured to be 25 feet (Elev. 320) below the top of roadway (Elev. 345) with water depths ranging from 0 to 5 +/- feet. The stream bed consisted of sands and cobbles with outcropping bedrock between Pier 3 and the North Abutment. Top of rock elevations from the seismic data is plotted on the proposed alignment plan in Figure 2 with top of rock elevations shown in purple. The top of rock data is also plotted on the boring profile along the center line of the proposed replacement bridge alignment (provided by S. W. Cole) in Figure 3. Data plotted on Figure 3 which is along the alignment is plotted as a green dashed line, while data plotted which is West of the alignment is plotted as a blue dashed line and data which is East of the alignment is plotted as a red dashed line. The data was plotted this way because along the South approach data indicates shallow rock west of the alignment along Rt. 156 and data indicates deep rock east of the alignment along Rt. 41.

Post data collection inspection of the alignment plan provided by S.W. Cole indicates buried concrete foundations on the Anstiss Morrill property coincident with the Seismic Line 2 at the south approach offset West along Rt. 156. There is a high potential that if the seismic line crossed over an intact/thick concrete slab there is a potential this would indicate an artificial shallow top of rock.

The seismic results are presented as separate profiles along each seismic line and are shown in Figures 4 -10. These profiles show varying thicknesses of soil/fill materials with a 1,500 ft/sec seismic velocity overlying dense soils/ablation till (south approach) with a seismic velocity of 2,500 ft/sec. Data indicates that along the north and south approach a water table at elevation 320-325+/- shows water saturated overburden/soils/till 5,000 ft/sec. The data indicates a competent bedrock with a seismic velocity of approximately 14,000+/- ft/sec.

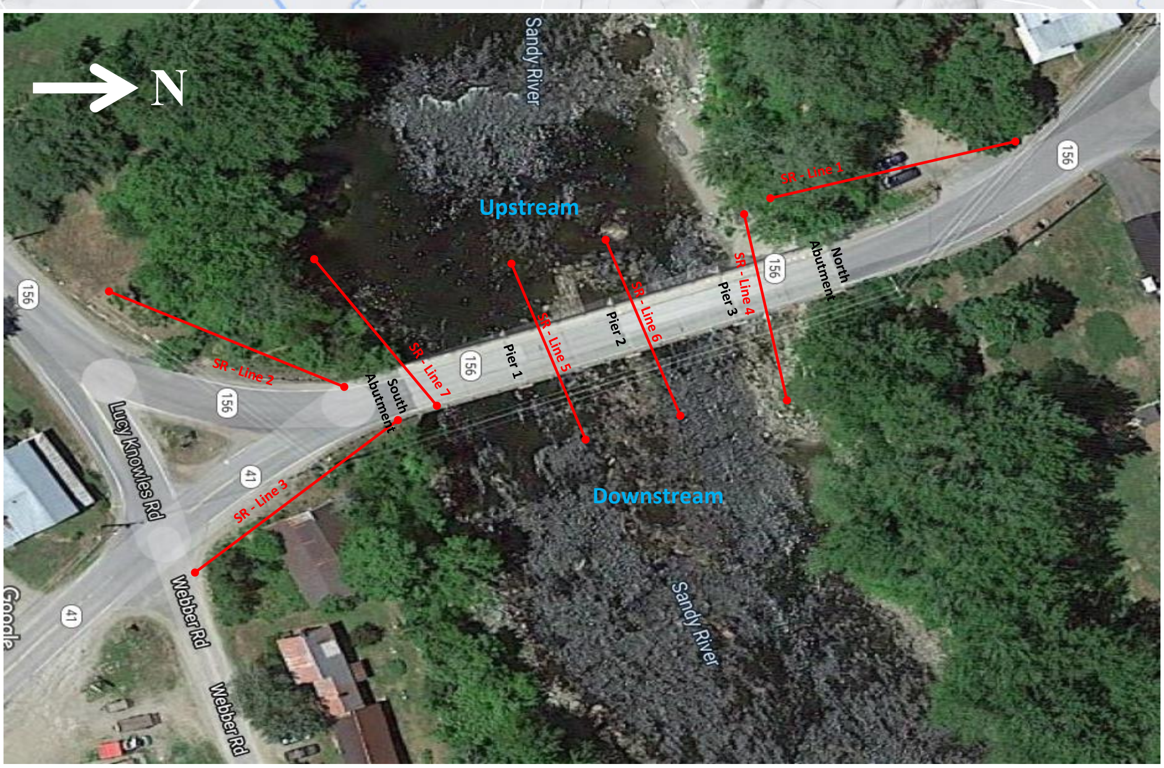
The rock outcropping, visible between Pier 3 and the North Abutment and shown in the photograph below, shows a highly irregular surface of the near vertical dipping bedrock. Top of bedrock surface shown on the profile sections is an average rock surface, localized high and low areas exist. Definition of high and low areas is a function of the seismic spread length, number of “shots” taken, geophone spacing, velocity contrast, and the irregularity of the rock surface. Variations of 3-5+/- feet are not accurately profiled. This would account for the 3-4-foot variation between the seismic data and boring data.

Photo: Visible outcropping bedrock between Pier 3 and North Abutment:
(note near vertical dip and variation in elevation)

FIGURES

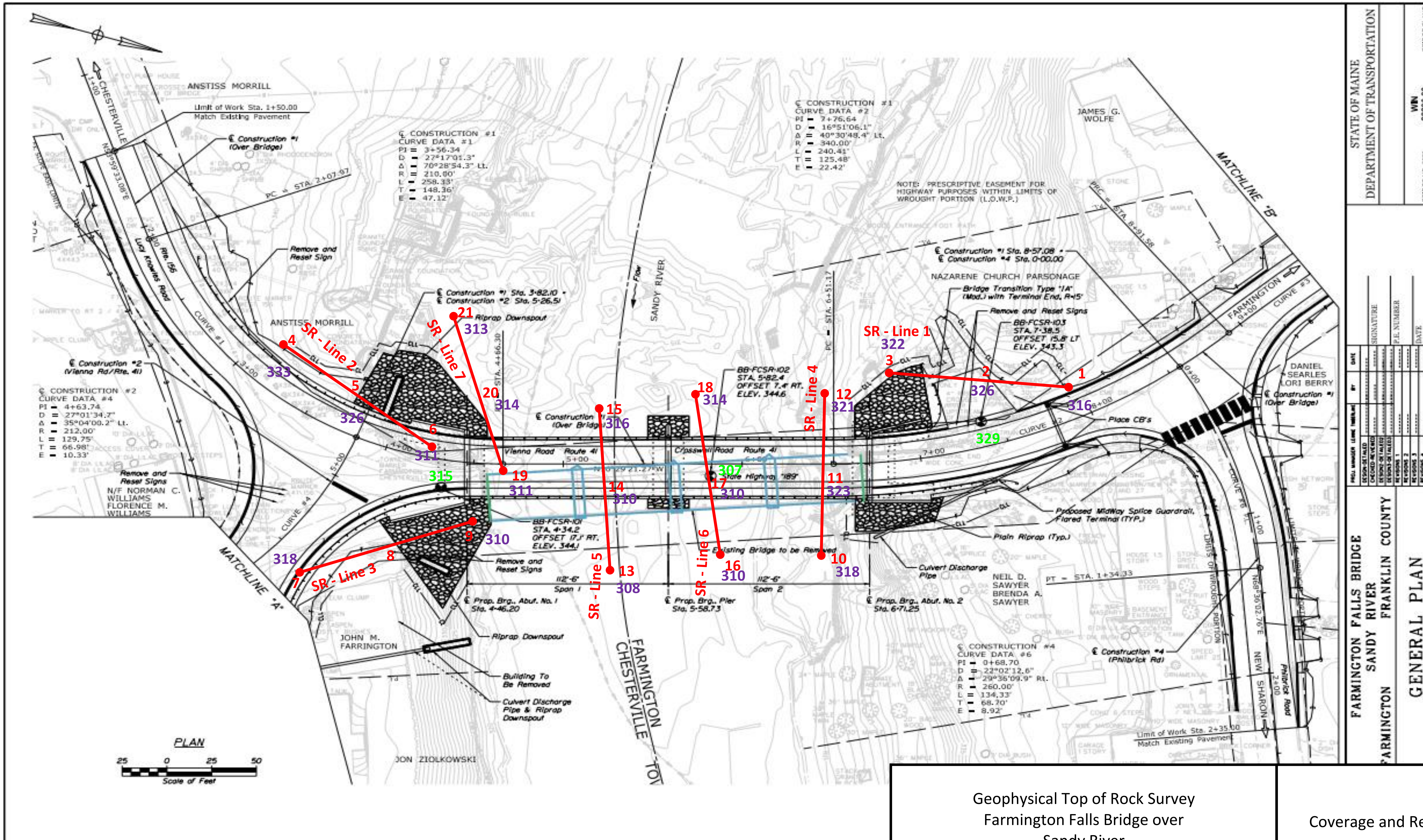
Farmington Falls Bridge over Sandy River Farmington, ME

South



North

Geophysical Top of Rock Survey Farmington Falls Bridge over Sandy River Farmington, ME prepared for S. W. Cole Engineering, Inc. by NDT Corporation	Area of Investigation	
	December 2020	Figure 1



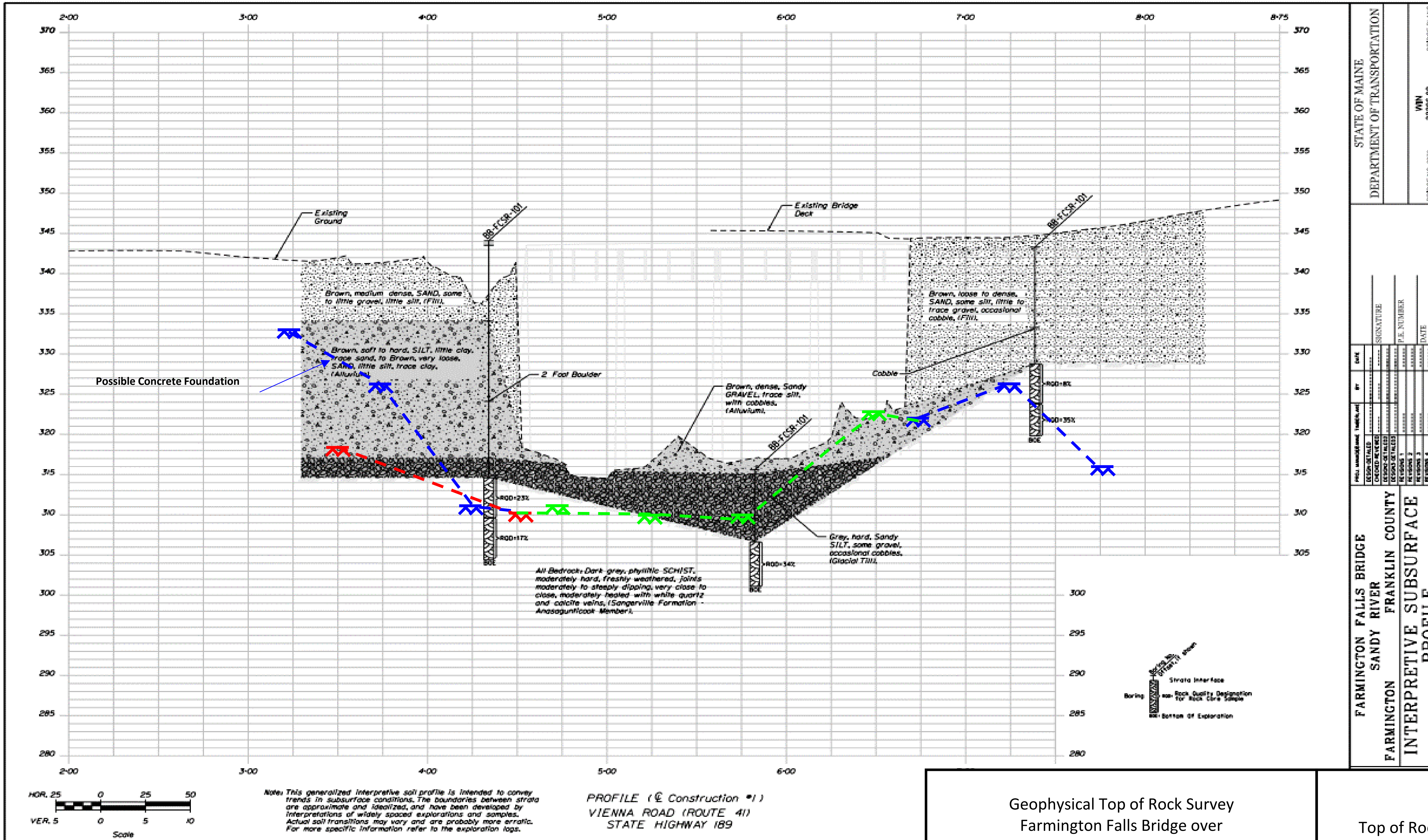
1 Seismic Shot #

100' Seismic Refraction Line

318 Seismic Refraction - Top of Rock Elevation (ft)

315 Boring - Top of Rock Elevation (ft)

Geophysical Top of Rock Survey Farmington Falls Bridge over Sandy River Farmington, ME prepared for S. W. Cole Engineering, Inc. by NDT Corporation	Coverage and Results	
	December 2020	Figure 2



Seismic Profile Top of Rock - Data offset West of Alignment

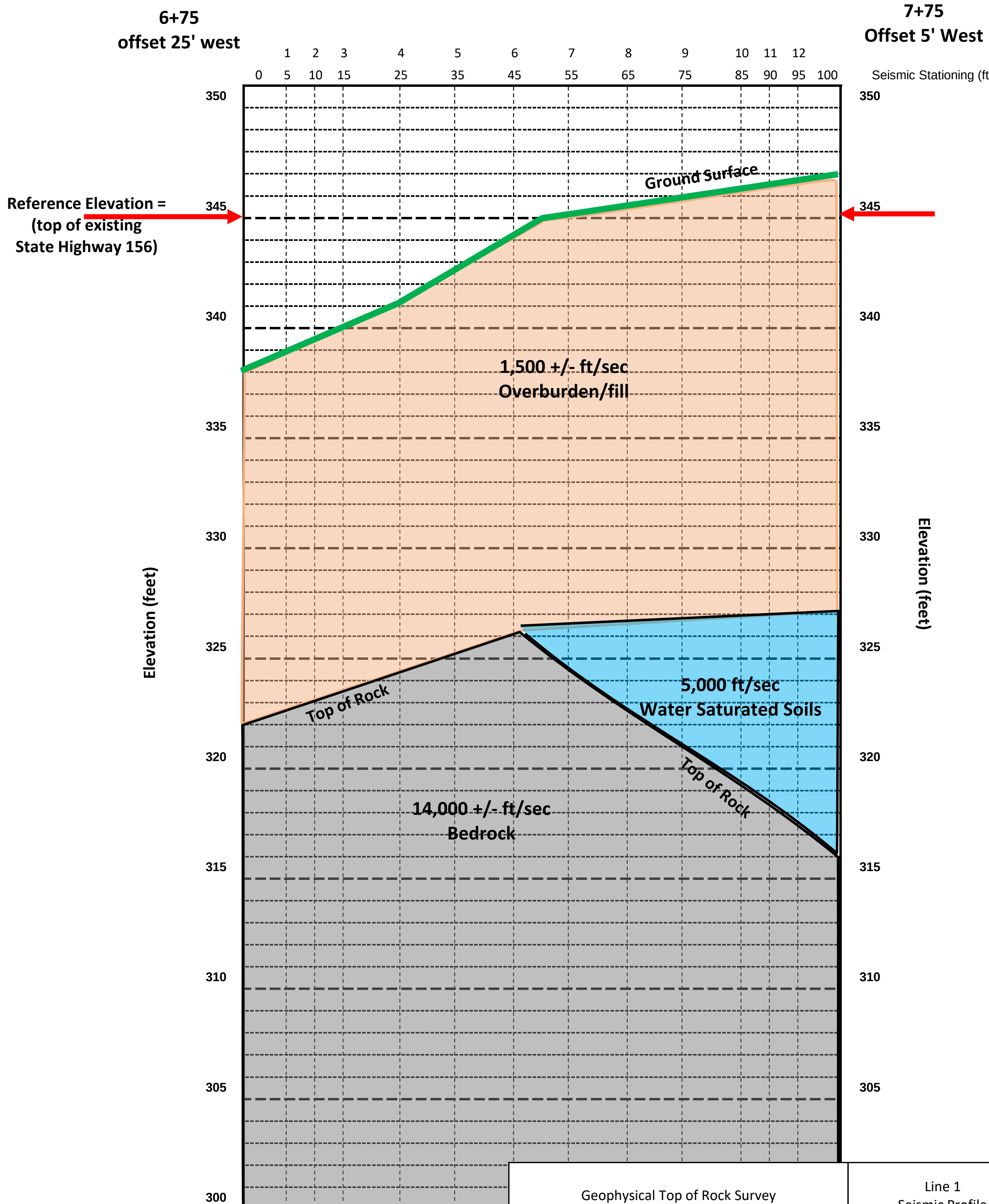
Seismic Profile Top of Rock - Data along Alignment

Seismic Profile Top of Rock - Data offset East of Alignment

Geophysical Top of Rock Survey Farmington Falls Bridge over Sandy River Farmington, ME prepared for S. W. Cole Engineering, Inc. by NDT Corporation	Top of Rock Profile	
	December 2020	Figure 3

Farmington Falls Bridge over Sandy River Farmington ME

Line 1 - Project Sta. 6+75-7+75 **Offset - 20+/- feet** **Date of Survey: 11/6/20**
West of existing EOR

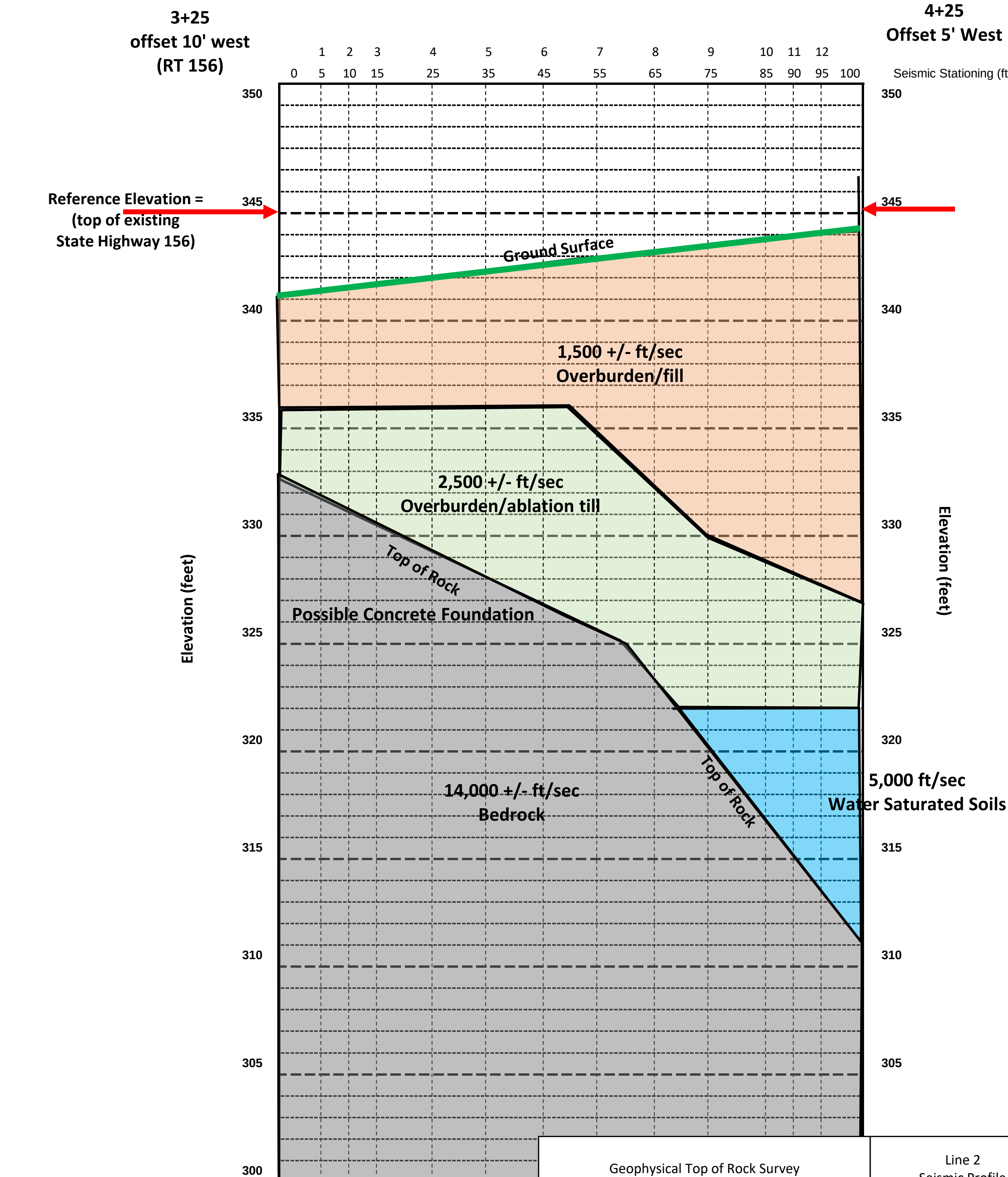


Geophysical Top of Rock Survey Farmington Falls Bridge over Sandy River Farmington, ME prepared for S. W. Cole Engineering, Inc. by NDT Corporation	Line 1 Seismic Profile 6+75 - 7+75 offset West	
	December 2020	Figure 4

Farmington Falls Bridge over Sandy River Farmington ME

Line 2 - Project Sta. 3+25 - 4+25
5+/- feet West of existing EOR

Offset - Date of Survey: 11/6/20

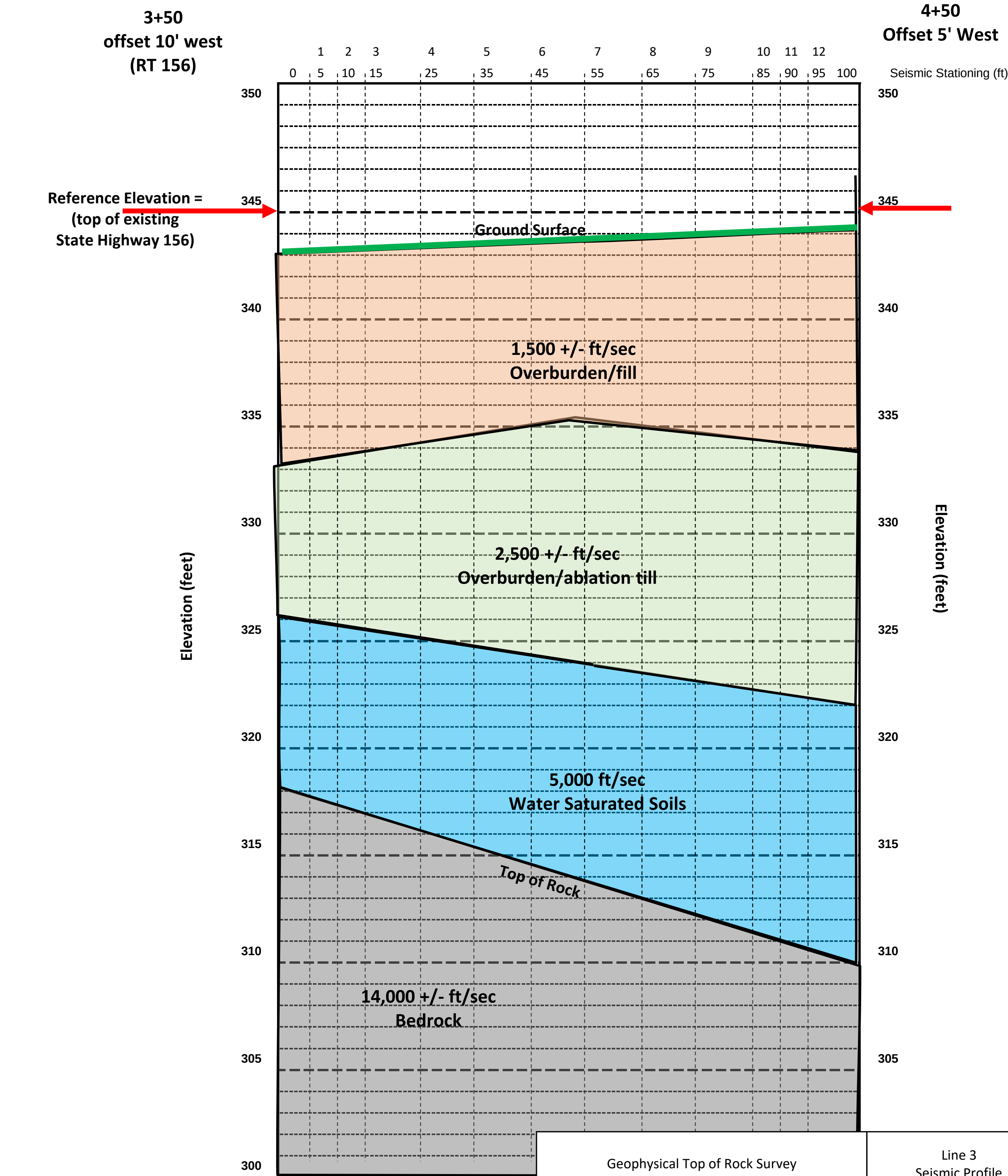


Geophysical Top of Rock Survey Farmington Falls Bridge over Sandy River Farmington, ME prepared for S. W. Cole Engineering, Inc. by NDT Corporation	Line 2 Seismic Profile 3+25 - 4+25 offset West	
	December 2020	Figure 5

Farmington Falls Bridge over Sandy River Farmington ME

Line 3 - Project Sta. 3+50 - 4+50
East of existing EOR

Offset - 5+/- feet **Date of Survey: 11/6/20**

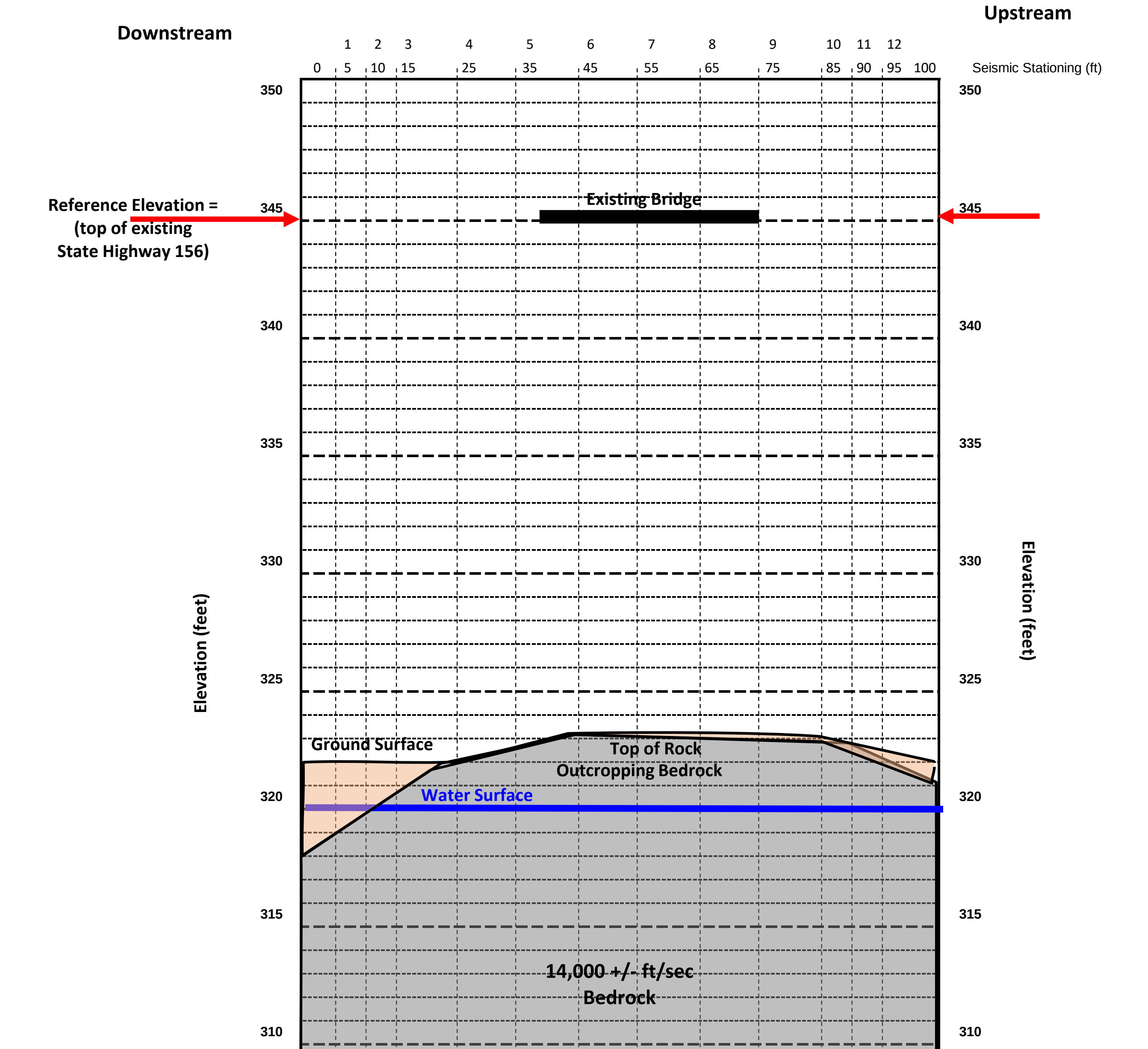


Geophysical Top of Rock Survey Farmington Falls Bridge over Sandy River Farmington, ME prepared for S. W. Cole Engineering, Inc. by NDT Corporation	Line 3 Seismic Profile 3+50 - 4+50 offset East	
	December 2020	Figure 6

Farmington Falls Bridge over Sandy River Farmington ME

Line 4 - Project Sta. 6+50 (between Pier 3 and North Abutment)

Date of Survey: 11/6/20

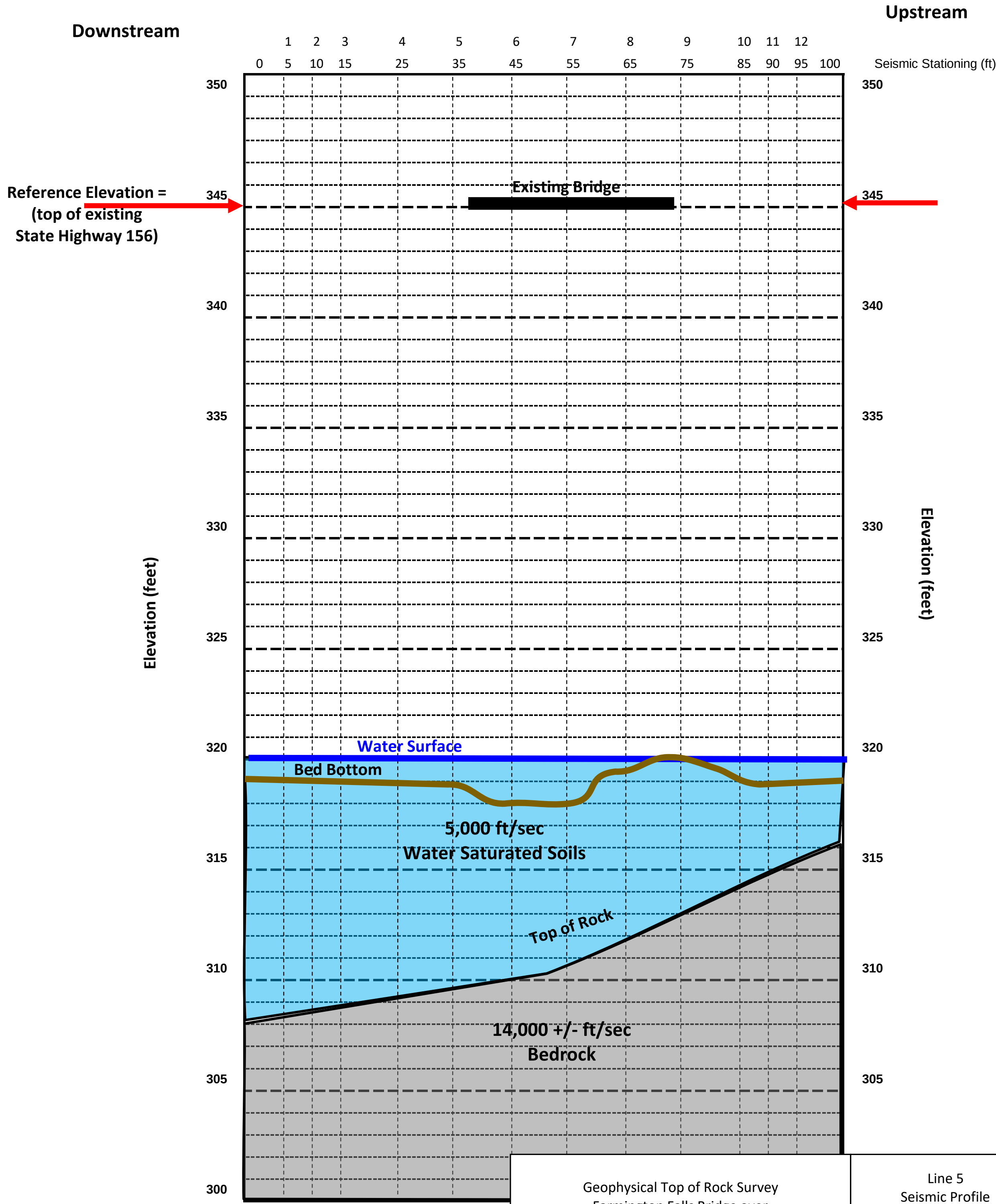


Geophysical Top of Rock Survey Farmington Falls Bridge over Sandy River Farmington, ME prepared for S. W. Cole Engineering, Inc. by NDT Corporation	Line 4 Seismic Profile 6+50 Between Pier 3 and North Abutment	
	December 2020	Figure 7

Farmington Falls Bridge over Sandy River Farmington ME

Line 5 - Project Sta. 5+25 (between Pier 1 and Pier 2)

Date of Survey: 11/6/20

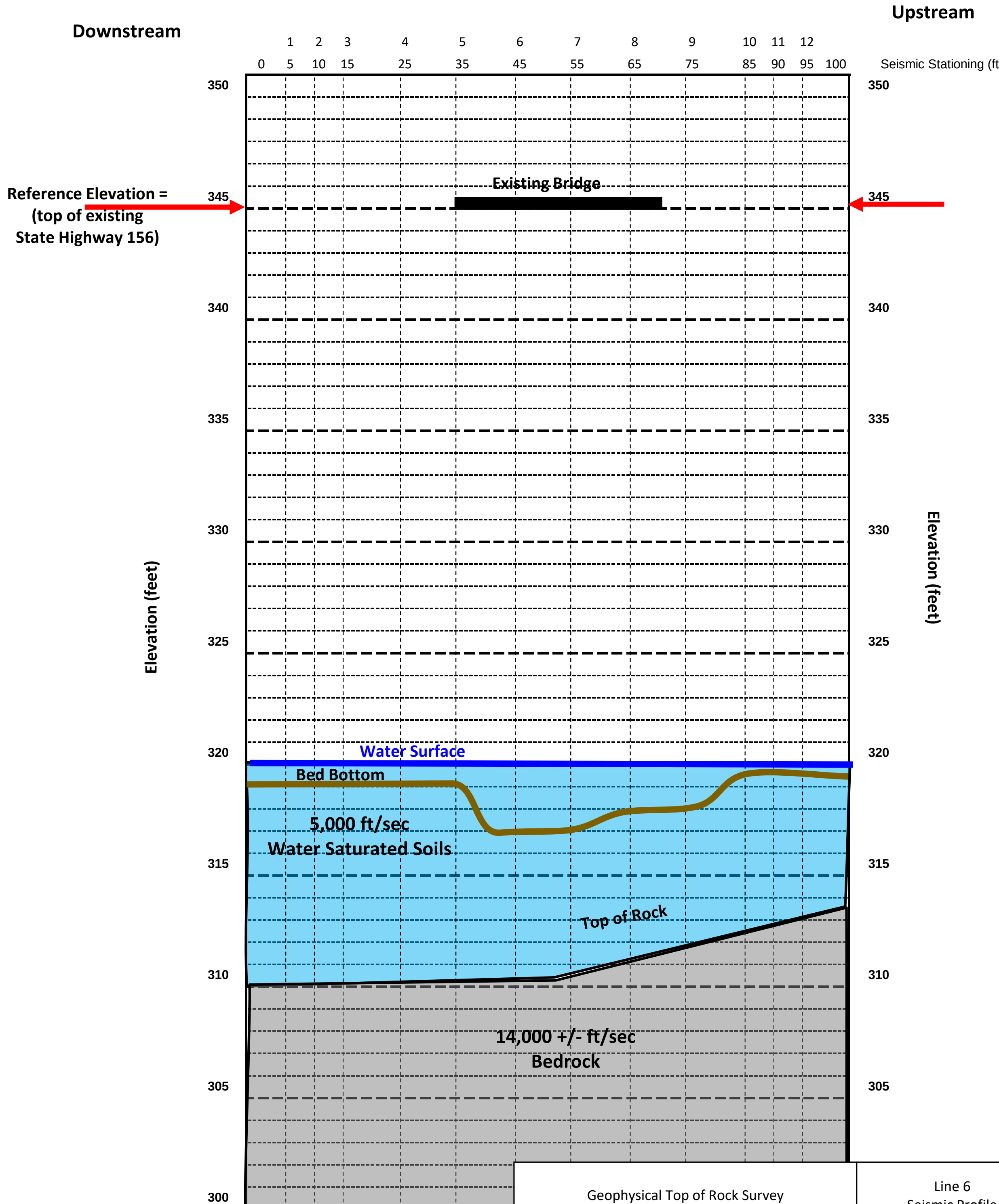


Geophysical Top of Rock Survey Farmington Falls Bridge over Sandy River Farmington, ME prepared for S. W. Cole Engineering, Inc. by NDT Corporation	Line 5 Seismic Profile 5+25 Between Pier 1 and Pier 2	
	December 2020	Figure 8

Farmington Falls Bridge over Sandy River Farmington ME

Line 6 - Project Sta. 5+75 (between Pier 2 and Pier 3)

Date of Survey: 11/6/20

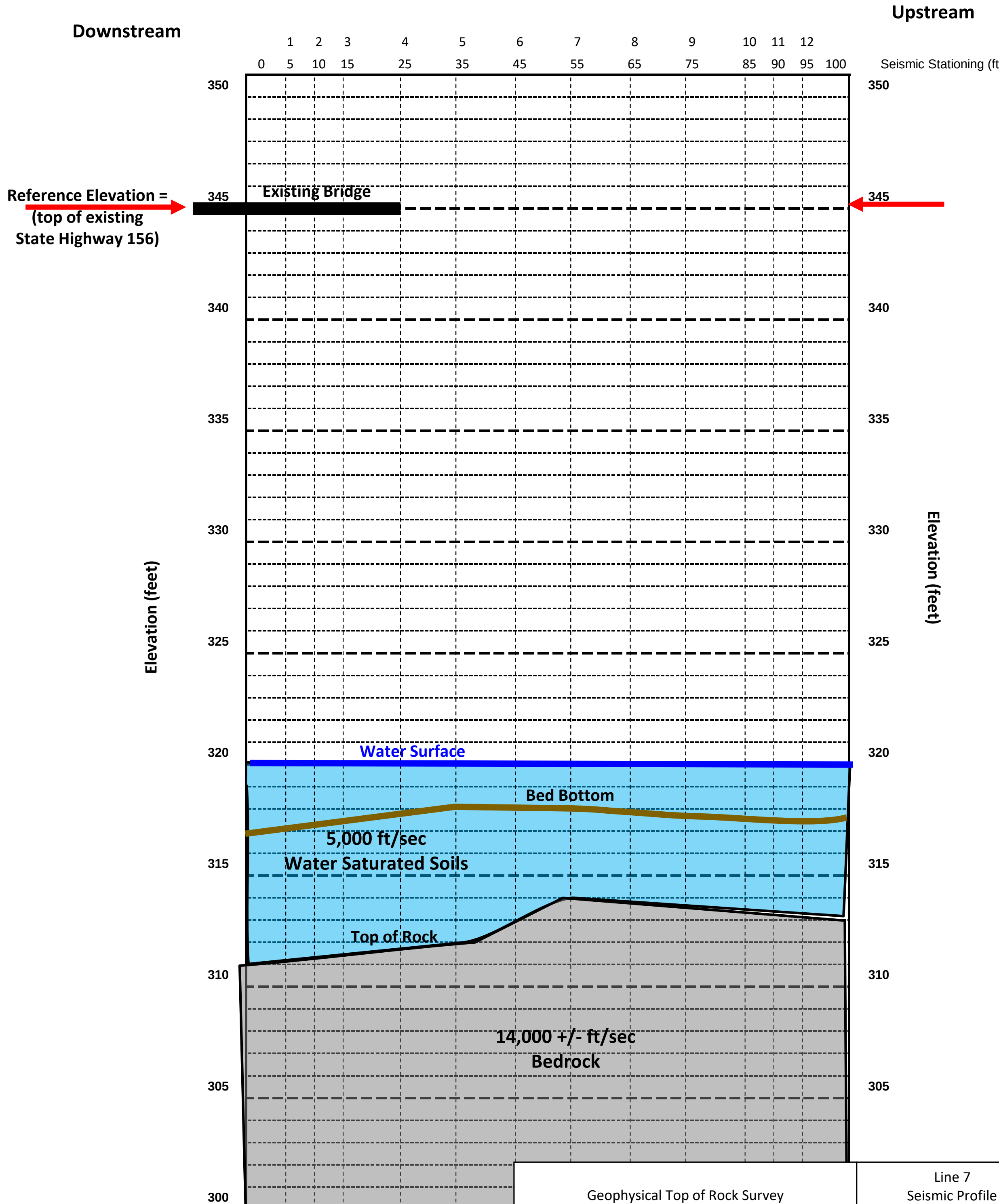


Geophysical Top of Rock Survey Farmington Falls Bridge over Sandy River Farmington, ME prepared for S. W. Cole Engineering, Inc. by NDT Corporation	Line 6 Seismic Profile 5+75 Between Pier 2 and Pier 3	
	December 2020	Figure 9

Farmington Falls Bridge over Sandy River Farmington ME

Line 7 - Project Sta. 4+75 (between South Abutment and Pier 1)

Date of Survey: 11/6/20



Geophysical Top of Rock Survey Farmington Falls Bridge over Sandy River Farmington, ME prepared for S. W. Cole Engineering, Inc. by NDT Corporation	Line 7 Seismic Profile 4+75 Between South Abutment and Pier 1	
	December 2020	Figure 10

APPENDIX 1

SEISMIC REFRACTION

APPENDIX: SEISMIC REFRACTION

OVERVIEW

Seismic exploration methods utilize the natural energy transmitting properties of the soils and rocks and are based on the principle that the velocity at which seismic waves travel through the earth is a function of the physical properties (elastic moduli and Poisson's ratio) of the materials. Energy is generated at the ends and at the center of the seismic spread. The geophone/hydrophone is in direct contact with the earth/water and converts the earth's motion resulting from the energy generation into electric signals with a voltage proportional to the particle velocity of the ground motion. The field operator can amplify and filter the seismic signals to minimize background noise. Data are recorded on magnetic disk and can be printed in the field. Interpretations are based on the time required for a seismic wave to travel form a source to a series of geophones/hydrophones located at specific intervals along the ground surface. The resultant seismic velocities are used for:

* Material identification.
* Stratigraphic correlation.
* Depth determinations.
* Calculation of elastic moduli values and Poison's ratio.

A variety of seismic wave types, differing in resultant particle motion, are generated by a near surface seismic energy source. The two types of seismic waves for seismic exploration are the compressional (P) wave and the shear (S) wave. Particle motion resulting from a (P-wave) is an oscillation, consisting of alternating compression and dilatation, orientated parallel to the direction of propagation. An S-wave causes particle motion transverse to the direction of propagation. The P-wave travels with a higher velocity of the two waves and is of greater importance for seismic surveying. The following discussions are concerned principally with P-waves.

Possible seismic wave paths include a direct wave path, a reflected wave path or a refracted wave path. These wave paths are illustrated in FIGURE A1. The different paths result in different travel times, so that the recorded seismic waveform will theoretically show three distinct wave arrivals. The direct and refracted wave paths are important to seismic refraction exploration while the reflected wave path is important for seismic reflection studies.

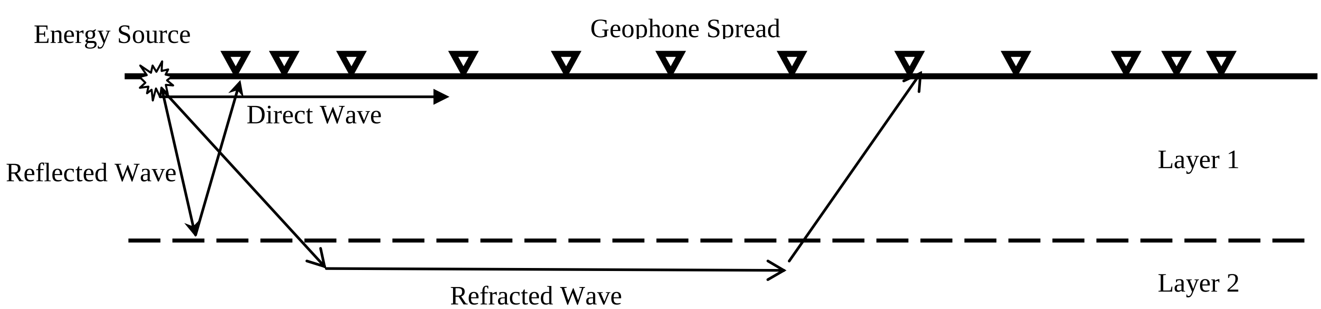


FIGURE A1:
SEISMIC WAVE PATHS FOR DIRECT WAVE, REFLECTED WAVE AND REFRACTED WAVE ILLUSTRATING EFFECTS OF A BOUNDARY BETWEEN MATERIALS WITH DIFFERENT ELASTIC PROPERTIES

Seismic waves incident on the interface between materials of different elastic properties at what is termed the critical angle are refracted and travel along the top of the lower layer. The critical angle is a function of the seismic velocities of the two materials. These same waves are then refracted back to the surface at the same angle. The recorded arrival times of these refracted waves, because they depend on the properties and geometry of the subsurface, can be analyzed to produce a vertical profile of the subsurface. Information such as the number, thickness and depths of stratigraphic layers, as well as clues to the composition of these units can be ascertained.

The first arrivals at the geophones/hydrophones located near the energy source are direct waves that travel through the near surface. At greater distances, the first arrival is a refracted wave. Lower layers typically are higher velocity materials, therefore the refracted wave will overtake both the direct wave and the reflected wave, because of the time gained travelling through the higher velocity material compensates for the longer wave path. Depth computations are based on the ratio of the layer velocities and the distance from the energy source to the point where refracted wave arrivals over take direct arrivals.

Although not the usual case, a constraint on refraction theory is that material velocities ideally should increase with depth. If a velocity inversion exists, i.e. where a higher velocity layer overlies a low velocity layer, depths and seismic velocities can be calculated but the uncertainty in calculations is increased unless borehole data are available.

APPLICATIONS

Seismic refraction technique is an accurate and effective method for determining the thickness of subsurface geologic layers. Applications for engineering design, assessment, and remediation as well as ground water and hydrogeologic studies include:

* Continuous profiling of subsurface layers including the bedrock surface
* Water-table depth determinations
* Mapping and general identification of significant stratigraphic layers
* Detection of sinkholes and cavities
* Detection of bedrock fracture zones
* Detection of filled-in areas
* Elastic moduli and Poisson's ratio values for subsurface layers

Seismic refraction investigations are particularly useful because seismic velocities can be used for material identification. FIGURE A2 presents a guide to material identification based on P-wave seismic velocities. In rocks and compacted overburden material, the seismic waves travel from grain to grain so that the measured seismic velocity value is a direct function of the solid material. In porous or fractured rock and most overburden materials the seismic waves travel partly or wholly though the fluid between the grains.

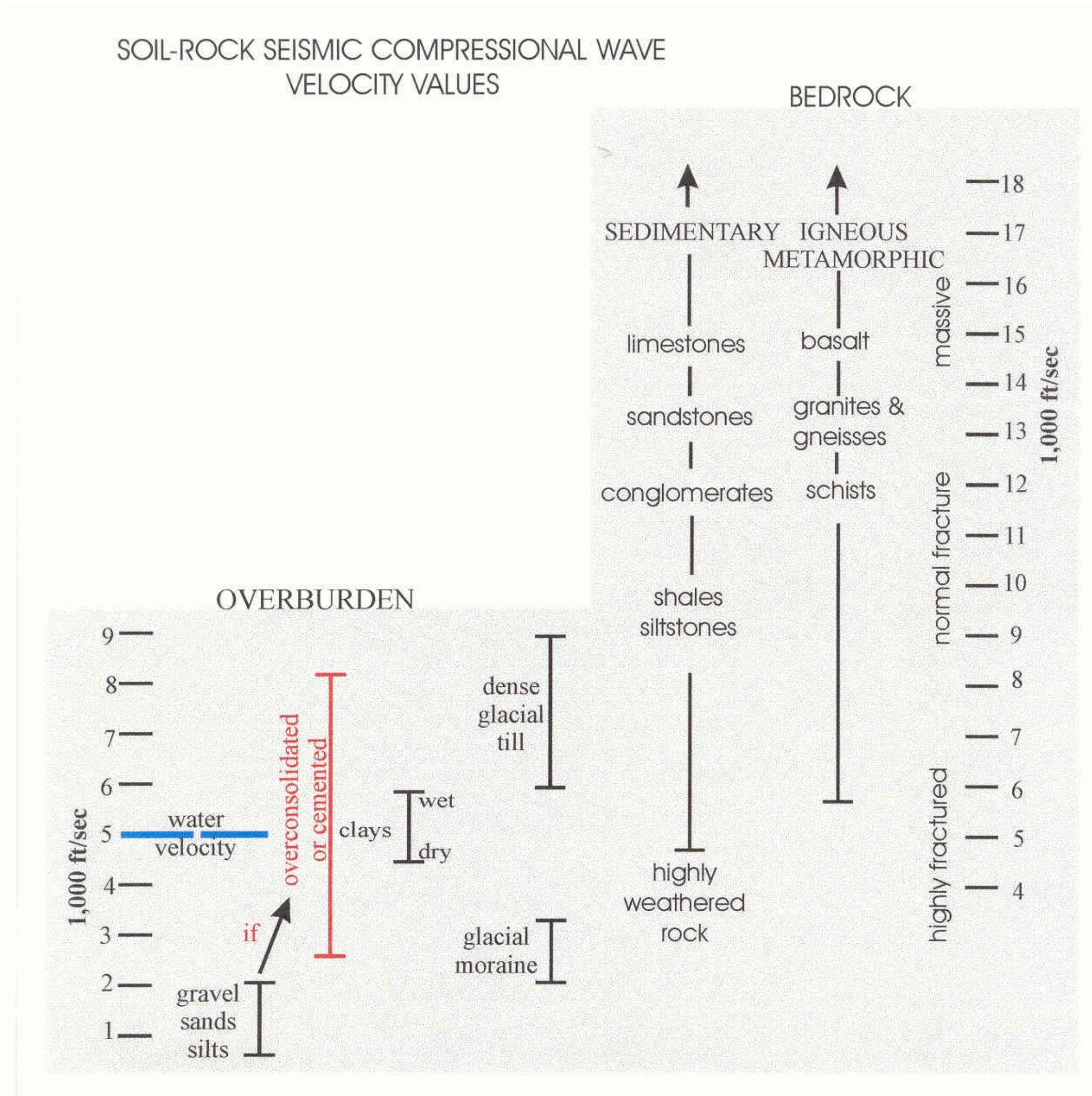


FIGURE A2:
GUIDE TO MATERIAL IDENTIFICATION BY P-WAVE VELOCITY

Seismic compressional wave velocities in unconsolidated deposits are significantly affected by water saturation. The seismic velocity values of unsaturated overburden materials such gravels, sands and silts generally fall in the range of 1,000 to 2,000 ft/sec. When these materials are water saturated, that is when the space between individual grains are 100% filled with water, the seismic velocities range from 4,800 to 5,100 ft/sec, equivalent to the compressional P-wave velocity of sound in water. This is because the seismic wave assumes the velocity of the faster medium, that of water. Even a small decrease in the saturation level will substantially lower the measured P-wave velocity of the material. Because of this velocity contrast between saturated and unsaturated materials, the water table acts as a strong refractor.

Seismic investigations over unconsolidated deposits are used to map stratigraphic discontinuities and to unravel the gross stratigraphy of the subsurface. These can be vertically as in the case of a dense till layer beneath a layer of saturated material or horizontally as in the case of the boundaries of a fill material. Often these boundaries represent significant hydrologic boundaries, such as those between aquifers and aquicludes.

A common use of seismic refraction is the determination of the thickness of a saturated layer in unconsolidated sediments and the depth to relatively impermeable bedrock or dense glacial till. Continuous subsurface profiles and even contour maps of the top of a particular horizon or layer of interest can be developed from a suite of seismic refraction data.

Bedrock velocities FIGURE A2 vary over a broad range depending on variables, which include:

* Rock type
* Density
* Degree of jointing/fracturing
* Degree of weathering

Fracturing and weathering generally reduce seismic velocity values in bedrock. Low velocity zones in seismic data must be evaluated carefully to determine if they are due to overburden conditions or fractured/weathered or perhaps even faulted bedrock.

EQUIPMENT:

The basic equipment necessary to conduct a seismic refraction investigation consists of:

* Energy source
* Seismometers (Geophones/Hydrophones)
* Seismic cables
* Seismograph

Energy sources used for seismic surveys are categorized as either non-explosive or explosive. The energy for a non-explosive seismic signal can be provided by one of the following:

* Sledge Hammer (very shallow penetration)
* Weight Drop
* Seisgun
* Airgun
* Sparker
* Vibrators (for reflection surveys)

Explosive sources can be categorized as:

- Dynamite
- Primers
- Blasting Agents

Choice of energy source is dependent on site conditions, depth of investigation, and seismic technique chosen as well as local restrictions. Explosive sources may be prohibited in urban areas where non-explosive sources can be routinely used. Deeper investigations usually require a larger energy source: therefore, explosives may be required for sufficient penetration.

Geophones/Hydrophones are sensitive vibration detectors, which convert ground motion to an electric voltage for recording the seismic wave arrivals. Seismic cables, which link the geophones/hydrophones and seismograph are generally fabricated with pre-measured locations for the geophones/hydrophones and shot point definitions.

The seismograph can be single channel or multi-channel, although, multi-channel seismographs (12 to 24 channels) are preferred and necessary for all but the simplest of very shallow surveys. The seismograph, amplifies (increases the voltage output of the geophones), conditions/filters the data, and produces analog and digital archives of the data. The analog archive is in the form of a thermal print of the data, which can be printed directly after acquisition in the field. The digital archive is stored on magnetic disk and can be used for subsequent computer processing and enable more extensive and detailed interpretation of seismic data.

ACQUISITION CONSIDERATIONS:

Several concerns arise before data collection, which must be addressed before of any seismic survey:

- Geophone spacing and Spread length
- Energy Source (discussed above)
- On-site utilities and cultural features (buildings, high tension lines, buried utilities, etc.)
- Vibration generating activities
- Geology
- Topography

To acquire seismic refraction data, a specific number of geophones are spaced at regular intervals along a straight line on the ground surface; this line is commonly referred to as a seismic spread. The length of spread determines the depth of penetration; a longer spread is required for a greater depth of penetration. Spread length should be approximately three to five times the required depth of penetration. Required resolution will control the number of geophones in each spread and the distance between each geophone. Closer spacings and more geophones usually result in more detail and greater resolution.

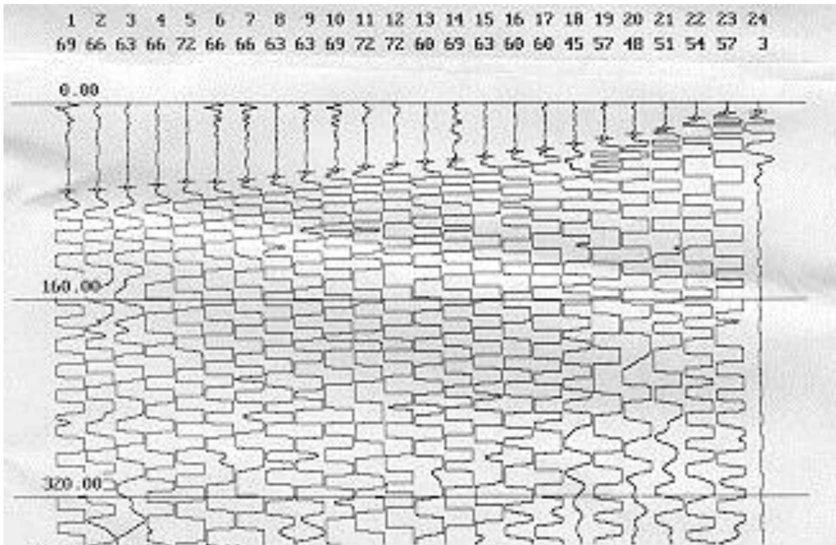
Cultural effects such as vibration generating activities, on-site utilities, and building affect where data can be acquired, and where lines/spreads are located. High volume traffic areas may require nighttime acquisition. If the survey is to be conducted near a building where vibration-sensitive manufacturing is conducted, data acquisition may be constrained to particular time intervals and appropriate energy sources must be used. Over head and buried utilities must be located an avoided, for both safety and induced electrical noise concerns. Since the seismic method measures ground vibration, it is inherently sensitive to noise from a variety of sources such as traffic, wind, rain etc. Signal Enhancement, such as record stacking, accomplished by adding a number of seismic signals from a repeated source, causes the seismic signal to "grow" out of the noise level, permitting operation in noisier environments and at greater source to phone spacings.

Knowledge of site geology can be used to determine the energy source. Some geologic materials, such as loose, unsaturated alluvium, do not transmit seismic energy as well and a powerful energy source may be required. Geologic conditions also dictate whether or not drilled shotholes are required. Site geology can also dictate the positioning of seismic lines/spreads. Where a bedrock depression of a feature is suspected, seismic lines should be orientated perpendicular to the suspected trend of the feature. Seismic cross profiles may be necessary to confirm depths to a particular refracting horizon.

The topography of a site dictates whether or not surveyed elevations are required. If possible, refraction profile lines should be positioned along level topography. For highly variable topography, a continuous elevation profile may be required to ensure sufficiently accurate cross-sections and to permit the use of time corrections in the interpretation of the refraction data.

DATA PRESENTATION AND INTERPETATION:

Interpretation of seismic refraction data involves solving a number of mathematical equations with the refraction data as it is presented on a travel-time versus distance chart. Seismic refraction data FIGURE A3 can be processed by plotting the "First Arrival" travel times at each geophone location. The preferred format of data presentation is a graph (Travel Time Plot) illustrated in FIGURE A4, in which travel time in milliseconds is plotted against source-receiver distance. From such a chart, the velocities of each layer can be obtained directly from the increase slope of each straight-line segment. Using the velocities the critical angle of refraction for each boundary can be calculated using Snell's Law. Then, utilizing these velocities, and angles and the recorded distances to crossover points (where line segments cross); the depths and thickness of each layer can be calculated using simple geometric relationships.



Shot point is 0.00

Channel	1	Phone location	195.00	Arrival time	71.3	msec
Channel	2	Phone location	190.00	Arrival time	70.3	msec
Channel	3	Phone location	185.00	Arrival time	68.5	msec
Channel	4	Phone location	175.00	Arrival time	66.3	msec
Channel	5	Phone location	165.00	Arrival time	63.8	msec
Channel	6	Phone location	155.00	Arrival time	63.8	msec
Channel	7	Phone location	145.00	Arrival time	62.0	msec
Channel	8	Phone location	135.00	Arrival time	59.0	msec
Channel	9	Phone location	125.00	Arrival time	56.8	msec
Channel	10	Phone location	115.00	Arrival time	54.8	msec
Channel	11	Phone location	110.00	Arrival time	51.5	msec
Channel	12	Phone location	105.00	Arrival time	49.3	msec
Channel	13	Phone location	95.00	Arrival time	47.3	msec
Channel	14	Phone location	90.00	Arrival time	46.0	msec
Channel	15	Phone location	85.00	Arrival time	43.0	msec
Channel	16	Phone location	75.00	Arrival time	39.0	msec
Channel	17	Phone location	65.00	Arrival time	37.0	msec
Channel	18	Phone location	55.00	Arrival time	33.0	msec
Channel	19	Phone location	45.00	Arrival time	28.0	msec
Channel	20	Phone location	35.00	Arrival time	25.0	msec
Channel	21	Phone location	25.00	Arrival time	20.0	msec
Channel	22	Phone location	15.00	Arrival time	12.0	msec
Channel	23	Phone location	10.00	Arrival time	8.0	msec
Channel	24	Phone location	5.00	Arrival time	7.0	msec

FIGURE A3:
TYPICAL 24 CHANNEL ANALOG SEISMIC REFRACTION RECORD, WITH FIRST ARRIVAL TIMES

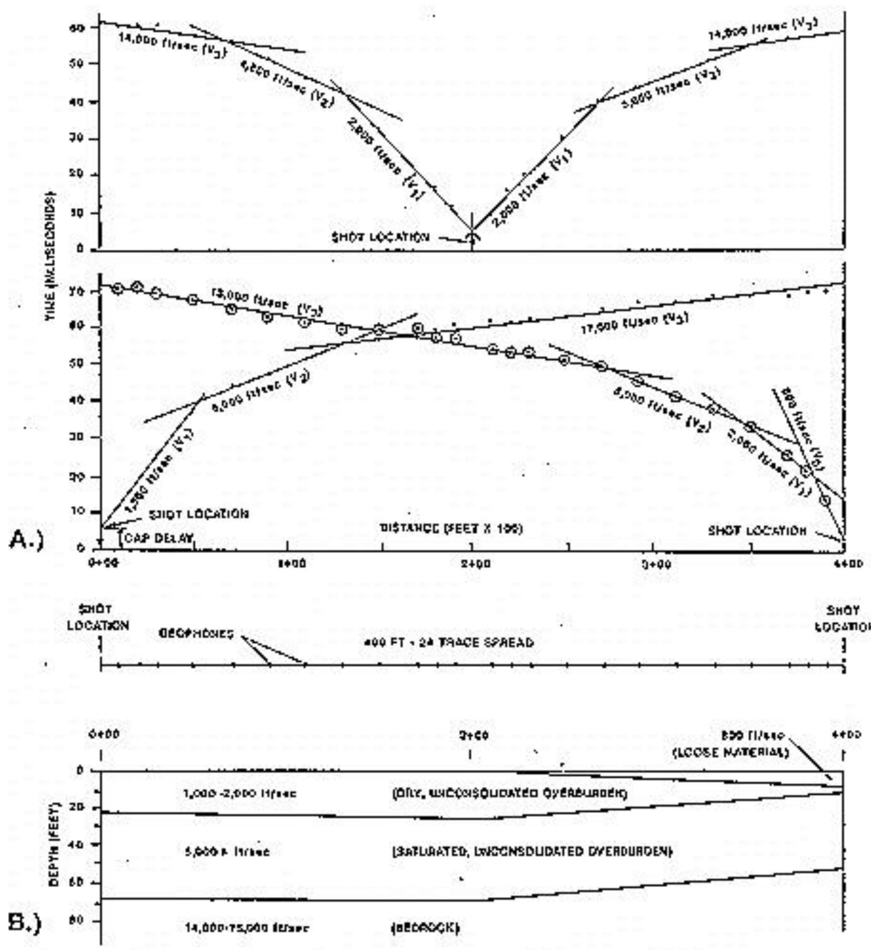


FIGURE A4:
A: TRAVEL-TIME PLOTS; UPPER PLOT IS A CENTER SHOT, LOWER PLOT IS TWO END SHOTS
B: RESULTING PROFILE OF SUBSURFACE MATERIALS SHOWING INTERFACE BETWEEN DIFFERENT SEISMIC VELOCITY LAYERS

The results of any seismic survey, refraction or reflection are usually presented in profile form showing elevations of seismic horizons/layers. Data acquired on a grid basis can be contoured and used to construct isopach maps. Seismic velocities and therefore, generalized material identifications should be presented on refraction profiles along with any test borings used for correlation to establish confidence in the overall subsurface data, both seismic and borings.

Where profiles indicate dipping boundaries, calculation of dips, true depths and true velocities involve more complicated equations. Further more, corrections for differing elevations and varying thicknesses of weathered zones must often be made. Fracturing and weathering generally reduce seismic velocity values in bedrock. Consequently, travel-time plots with late arrivals must be evaluated carefully to determine if the late arrival times (slower velocities) are due to overburden conditions or fractured/weathered bedrock.

Very thin layers or low velocity zones often complicate the travel-time chart as well. Although not the usual case, one constraint on refraction theory is that material velocities ideally should increase with depth. If a velocity inversion exists, i.e. where a higher velocity layer overlies a low velocity layer, depths and seismic velocities can be calculated but the uncertainty in calculations is increased unless borehole velocity data are available.

ADVANTAGES AND LIMITATIONS:

The seismic refraction technique, when properly employed, is the most accurate of the geophysical methods for determining subsurface layering and materials. It is extremely effective in that as much as 2,000 linear feet or more of profiling can be acquired in a field day. The resulting profiles can be used to minimize drilling and place drilling at locations where borehole information will be maximized resulting in cost-effective exploration. A standard drilling program runs the risk of missing key locations due to drillhole spacing. This risk is substantially reduced when refraction is used.

In summary, the advantages and limitations of the seismic techniques are:

Advantages:

* Material identification
* Subsurface data over broader areas at less cost than drilling
* Relatively accurate depth determination
* Correlation between drillholes
* Preliminary results available almost immediately
* Rapid data processing

Limitations:

* As depth of interest and geophone spacing increases, resolution decreases
* Thin layers may be undetected
* Velocity inversions may add uncertainty to calculations
* Susceptible to noise interference in urban areas, which require use of grounded cables and equipment, signal enhancement and alternative energy sources.

S.W.COLE ENGINEERING, INC.

APPENDIX F
Calculations

Bedrock Bearing Resistance

Evaluation of Nominal and Factored Bearing Resistance on Rock

Service Limit State

From 2017 AASHTO LRFD Table C10.6.2.6.1-1, Presumptive Bearing Resistance for Spread Footing Foundations at the Service Limit State Modified after U.S. Department of the Navy (1982)

Bearing Material: weathered or broken bedrock of any kind except shale

Consistency in Place: moderately hard to hard

Bearing Resistance Range: 16 to 24 ksf

Recommended Bearing Resistance: 20 ksf

Nominal Bearing Resistance $q_{n\_service} := 20\ \boldsymbol{ksf}$

Resistance Factor Service Limit $\phi_{b\_service} := 1.0$

Factored Bearing Resistance $q_{f\_service} := \phi_{b\_service} \cdot q_{n\_service} = 20\ \boldsymbol{ksf}$

Recommend Service Limit Nominal and Factored Bearing Resistance = 20 ksf

From 2017 LRFD Article C10.6.2.6.1, when using presumptive bearing resistance values the service limit bearing resistances are limited to 1 inch of settlement

Strength and Extreme Limit States

Reference(s): Wyllie (2009) Foundations on Rock, 2nd Ed.
Hoek and Brown (1988) The Hoek-Brown Failure Criterion - A 1988 Update
AASHTO LRFD Bridge Design Specifications, 8th Ed. 2017
AASHTO LRFD Bridge Design Specifications, 6th Ed. 2012

Establish Bedrock Properties

BB-FCSR-101, R2-R3: phyllitic SCHIST, moderately hard, RQD = 23, 17%

BB-FCSR-102, R1: phyllitic SCHIST, moderately hard, RQD = 34%
UCT qp = XXXX psi

BB-FCSR-103, R1-R2: phyllitic SCHIST, moderately hard, RQD = 8, 35%
UCT qp = XXXX psi

Determine Rock Mass Rating (RMR)
Values based on 2012 LRFD Table 10.4.6.4-1 Geomechanics Classification of Rock Masses

1. Strength of Intact Rock Material
   Compressive Strengths (from laboratory testing):

$$q_u := \begin{bmatrix} 7409 \\ 6400 \end{bmatrix} \ psi$$

Use $q_{u\_design} := 6900 \ psi = 994 \ ksf$

For Uniaxial Compressive Strength = 520-1,080 ksf

$RR_1 := 4$

2. Drill Core Quality RQD
   RQD ranged from 8 to 35% (Very Poor to Poor)
   RQD near bearing surface ranged from 8 to 34% (Very Poor to Poor)
   weighted average of 22%

   For RQD < 25%

$RR_2 := 3$

3. Spacing of Joints
   Jointing near the bearing surface generally characterized as "very close to moderately close." Assume bedrock joints close joint with spacing of 2 to 12 inches

   For joint spacing of 2 to 12 in

$RR_3 := 10$

4. Condition of Joints
   Jointing generally characterized as healed with quartz and calcite infilling

   For joints with slightly rough surfaces, separation of less than 0.05 inch and soft joint wall rock

$RR_4 := 12$

5. Groundwater Conditions
   Bedrock generally underwater.
   Assume "water under moderate pressure"

$RR_5 := 4$

Sum Relative Ratings 1 through 5 to develop Raw RMR

$$RMR_{raw} := RR_1 + RR_2 + RR_3 + RR_4 + RR_5 = 33$$

6. Strike and Dip Orientations

Jointing generally characterized as moderately and steeply dipping.

From 2012 LRFD Table 10.4.6.4-2 for Strike and Dip Orientations

For Foundations, assume rating of "Fair"

$RR_6 := -7$

Adjust RMR to account for strike and dip

$RMR_{adjusted} := RMR_{raw} + RR_6 = 26$

Determine Rock Mass Class from Adjusted RMR

From 2012 LRDF Table 10.4.6.4-3 Geomechanics Rock Mass Classes

Adjusted RMR of 26 is indicative of Poor Rock - Class IV

Determine Rock Type

From 2012 LRDF Table 10.4.6.4-4

Rock Type E - Coarse grained polyminerallic igneous & metamorphic crystalline rocks - *amphibolite, gabbro, gneiss, granite, norite, quartz-diorite*

Determine Rock Property Constants *s* and *m*

From Hoek and Brown (1988) Table 1, Calculate m and s

To evaluate the disturbed rock mass constants (m and s), the m and s values for "intact rock samples" are used.

For Rock Type E, Intact Rock Mass constants m (mi) and s (si):

$m_i := 25$

$s_i := 1$

For Disturbed rock mass use Hoek and Brown (1988)

Eqn 18 m/mi = exp((RMR-100)/14)
Eqn 19 s = exp((RMR-100)/6)

$m := m_i \cdot \exp\left(\frac{RMR_{adjusted} - 100}{14}\right)$ $m = 0.127$

$s := \exp\left(\frac{RMR_{adjusted} - 100}{6}\right)$ $s = 4.403 \cdot 10^{-6}$

Determine Correction Factor for Foundation Shape
From Wyllie (2009) Table 5.4 (Pg 138)

Evaluate abutments and pier foundation shape factors **ASSUMED VALUES**

North Abutment

$L_f := 42\ ft$ $B_f := 18\ ft$ $\frac{L_f}{B_f} = 2.3$ $C_{f1\_A1} := 1.12$

South Abutment

$L_f := 42\ ft$ $B_f := 18\ ft$ $\frac{L_f}{B_f} = 2.3$ $C_{f1\_A2} := 1.12$

Pier

$L_f := 30\ ft$ $B_f := 15\ ft$ $\frac{L_f}{B_f} = 2$ $C_{f1\_P} := 1.15$

Use $C_{f1\_design} := 1.12$

$$q_{nominal} := C_{f1\_design} \cdot \sqrt{s} \cdot q_{u\_design} \cdot \left(1 + \sqrt{m \cdot \left(s^{-0.5}\right) \cdot 1}\right) = 20.5\ ksf$$

Recommend Strength & Extreme Limit Nominal Bearing Resistance = 20.5 ksf

Factored Bearing Resistance - Strength I

From AASHTO LRFD Table 10.5.5.2.2-1, Resistance Factor for Geotechnical Resistance of Shallow Foundations at the Strength Limit State

$\varphi_b := 0.45$

$q_{f\_strength} := \varphi_b \cdot q_{nominal} = 9.2\ ksf$

Recommend Strength Limit Factored Bearing Resistance = 9.2 ksf

Factored Bearing Resistance - Extreme I

From AASHTO LRFD Table 10.5.5.2.2-1, Resistance Factor for Geotechnical Resistance of Shallow Foundations at the Extreme Limit State

$\varphi_b := 0.8$

$q_{f\_extreme} := \varphi_b \cdot q_{nominal} = 16.4\ ksf$

Recommend Extreme Limit Factored Bearing Resistance = 16.4 ksf

Evaluation of Nominal and Factored Bearing Resistance on Rock

Service Limit State

From 2017 AASHTO LRFD Table C10.6.2.6.1-1, Presumptive Bearing Resistance for Spread Footing Foundations at the Service Limit State Modified after U.S. Department of the Navy (1982)

Bearing Material:	weathered or broken bedrock of any kind except shale
Consistency in Place:	moderately hard to hard
Bearing Resistance Range:	16 to 24 ksf
Recommended Bearing Resistance:	20 ksf
Nominal Bearing Resistance	$q_{n\_service} := 20\ ksf$
Resistance Factor Service Limit	$\phi_{b\_service} := 1.0$
Factored Bearing Resistance	$q_{f\_service} := \phi_{b\_service} \cdot q_{n\_service} = 20\ ksf$

Recommend Service Limit Nominal and Factored Bearing Resistance = 20 ksf

From 2017 LRFD Article C10.6.2.6.1, when using presumptive bearing resistance values the service limit bearing resistances are limited to 1 inch of settlement

Strength and Extreme Limit States

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BB-FCSR-102, R1: phyllitic SCHIST, moderately hard, RQD = 34%
UCT qp = 17,030 psi

BB-FCSR-103, R1-R2: phyllitic SCHIST, moderately hard, RQD = 8, 35%
UCT qp = 9,200 psi

Calculated by: MAS / March 2020
Checked by: TJB / March 2020

Determine Rock Mass Rating (RMR)
Values based on 2012 LRFD Table 10.4.6.4-1 Geomechanics Classification of Rock Masses

1. Strength of Intact Rock Material
Compressive Strengths (from laboratory testing):

$$q_u := \begin{bmatrix} 17030 \\ 9200 \end{bmatrix} \; psi$$

Use $q_{u\_design} := 9200 \; psi = 1325 \; ksf$

For Uniaxial Compressive Strength = 1,080-2,160 ksf

$RR_1 := 7$

2. Drill Core Quality RQD
RQD ranged from 8 to 35% (Very Poor to Poor)
RQD near bearing surface ranged from 8 to 34% (Very Poor to Poor)
weighted average of 22%

For RQD < 25%

$RR_2 := 3$

3. Spacing of Joints
Jointing near the bearing surface generally characterized as "very close to moderately close." Assume bedrock joints close joint with spacing of 2 to 12 inches

For joint spacing of 2 to 12 in

$RR_3 := 10$

4. Condition of Joints
Jointing generally characterized as healed with quartz and calcite infilling

For joints with slightly rough surfaces, separation of less than 0.05 inch and soft joint wall rock

$RR_4 := 12$

5. Groundwater Conditions
Bedrock generally underwater.
Assume "water under moderate pressure"

$RR_5 := 4$

Sum Relative Ratings 1 through 5 to develop Raw RMR

$$RMR_{raw} := RR_1 + RR_2 + RR_3 + RR_4 + RR_5 = 36$$

6. Strike and Dip Orientations

Jointing generally characterized as moderately and steeply dipping.

From 2012 LRFD Table 10.4.6.4-2 for Strike and Dip Orientations

For Foundations, assume rating of "Fair"

$RR_6 := -7$

Adjust RMR to account for strike and dip

$RMR_{adjusted} := RMR_{raw} + RR_6 = 29$

Determine Rock Mass Class from Adjusted RMR
From 2012 LRDF Table 10.4.6.4-3 Geomechanics Rock Mass Classes

Adjusted RMR of 29 is indicative of Poor Rock - Class IV

Determine Rock Type
From 2012 LRDF Table 10.4.6.4-4

Rock Type E - Coarse grained polyminerallic igneous & metamorphic crystalline rocks - *amphibolite, gabbro, gneiss, granite, norite, quartz-diorite*

Determine Rock Property Constants *s* and *m*
From Hoek and Brown (1988) Table 1, Calculate m and s

To evaluate the disturbed rock mass constants (m and s), the m and s values for "intact rock samples" are used.

For Rock Type E, Intact Rock Mass constants m (mi) and s (si):

$m_i := 25$

$s_i := 1$

For Disturbed rock mass use Hoek and Brown (1988)

Eqn 18 m/mi = exp((RMR-100)/14)
Eqn 19 s = exp((RMR-100)/6)

$m := m_i \cdot \exp\left(\frac{RMR_{adjusted} - 100}{14}\right)$ $m = 0.157$

$s := \exp\left(\frac{RMR_{adjusted} - 100}{6}\right)$ $s = 7.259 \cdot 10^{-6}$

Determine Correction Factor for Foundation Shape
From Wyllie (2009) Table 5.4 (Pg 138)

Evaluate abutments and pier foundation shape factors **ASSUMED VALUES**

North Abutment

$L_f := 42\ ft$ $B_f := 18\ ft$ $\frac{L_f}{B_f} = 2.3$ $C_{f1\_A1} := 1.12$

South Abutment

$L_f := 42\ ft$ $B_f := 18\ ft$ $\frac{L_f}{B_f} = 2.3$ $C_{f1\_A2} := 1.12$

Pier

$L_f := 30\ ft$ $B_f := 15\ ft$ $\frac{L_f}{B_f} = 2$ $C_{f1\_P} := 1.15$

Use $C_{f1\_design} := 1.12$

$$q_{nominal} := C_{f1\_design} \cdot \sqrt{s} \cdot q_{u\_design} \cdot \left(1 + \sqrt{m \cdot \left(s^{-0.5}\right) \cdot 1}\right) = 34.5\ ksf$$

Recommend Strength & Extreme Limit Nominal Bearing Resistance = 20.5 ksf

Factored Bearing Resistance - Strength I

From AASHTO LRFD Table 10.5.5.2.2-1, Resistance Factor for Geotechnical Resistance of Shallow Foundations at the Strength Limit State

$\varphi_b := 0.45$

$q_{f\_strength} := \varphi_b \cdot q_{nominal} = 15.5\ ksf$

Recommend Strength Limit Factored Bearing Resistance = 9.2 ksf

Factored Bearing Resistance - Extreme I

From AASHTO LRFD Table 10.5.5.2.2-1, Resistance Factor for Geotechnical Resistance of Shallow Foundations at the Extreme Limit State

$\varphi_b := 0.8$

$q_{f\_extreme} := \varphi_b \cdot q_{nominal} = 27.6\ ksf$

Recommend Extreme Limit Factored Bearing Resistance = 16.4 ksf

Table 10.4.6.4-1—Geomechanics Classification of Rock Masses

<table>
<tr><th colspan="3">Parameter</th><th colspan="7">Ranges of Values</th></tr>
<tr><td rowspan="3">1</td><td rowspan="2">Strength of intact rock material</td><td>Point load strength index</td><td>>175 ksf</td><td>85–175 ksf</td><td>45–85 ksf</td><td>20–45 ksf</td><td colspan="3">For this low range, uniaxial compressive test is preferred</td></tr>
<tr><td>Uniaxial compressive strength</td><td>>4320 ksf</td><td>2160–4320 ksf</td><td>1080–2160 ksf</td><td>520–1080 ksf</td><td>215–520 ksf</td><td>70–215 ksf</td><td>20–70 ksf</td></tr>
<tr><td colspan="2">Relative Rating</td><td>15</td><td>12</td><td>7</td><td>4</td><td>2</td><td>1</td><td>0</td></tr>
<tr><td rowspan="2">2</td><td colspan="2">Drill core quality RQD</td><td colspan="2">90% to 100%</td><td>75% to 90%</td><td>50% to 75%</td><td colspan="2">25% to 50%</td><td><25%</td></tr>
<tr><td colspan="2">Relative Rating</td><td colspan="2">20</td><td>17</td><td>13</td><td colspan="2">8</td><td>3</td></tr>
<tr><td rowspan="2">3</td><td colspan="2">Spacing of joints</td><td colspan="2">>10 ft</td><td>3–10 ft</td><td>1–3 ft</td><td colspan="2">2 in.–1 ft</td><td><2 in.</td></tr>
<tr><td colspan="2">Relative Rating</td><td colspan="2">30</td><td>25</td><td>20</td><td colspan="2">10</td><td>5</td></tr>
<tr><td rowspan="2">4</td><td colspan="2">Condition of joints</td><td colspan="2">• Very rough surfaces
• Not continuous
• No separation
• Hard joint wall rock</td><td>• Slightly rough surfaces
• Separation <0.05 in.
• Hard joint wall rock</td><td>• Slightly rough surfaces
• Separation <0.05 in.
• Soft joint wall rock</td><td colspan="2">• Slicken-sided surfaces
or
• Gouge <0.2 in. thick
or
• Joints open 0.05–0.2 in.
• Continuous joints</td><td>• Soft gouge >0.2 in. thick
or
• Joints open >0.2 in.
• Continuous joints</td></tr>
<tr><td colspan="2">Relative Rating</td><td colspan="2">25</td><td>20</td><td>12</td><td colspan="2">6</td><td>0</td></tr>
<tr><td rowspan="4">5</td><td rowspan="3">Groundwater conditions
(use one of the three evaluation criteria as appropriate to the method of exploration)</td><td>Inflow per 30 ft tunnel length</td><td colspan="2">None</td><td colspan="2"><400 gal./hr.</td><td colspan="2">400–2000 gal./hr.</td><td>>2000 gal./hr.</td></tr>
<tr><td>Ratio = joint water pressure/ major principal stress</td><td colspan="2">0</td><td colspan="2">0.0–0.2</td><td colspan="2">0.2–0.5</td><td>>0.5</td></tr>
<tr><td>General Conditions</td><td colspan="2">Completely Dry</td><td colspan="2">Moist only (interstitial water)</td><td colspan="2">Water under moderate pressure</td><td>Severe water problems</td></tr>
<tr><td colspan="2">Relative Rating</td><td colspan="2">10</td><td colspan="2">7</td><td colspan="2">4</td><td>0</td></tr>
</table>

Table 10.4.6.4-2—Geomechanics Rating Adjustment for Joint Orientations

Strike and Dip Orientations of Joints		Very Favorable	Favorable	Fair	Unfavorable	Very Unfavorable
Ratings	Tunnels	0	–2	–5	–10	–12
	Foundations	0	–2	–7	–15	–25
	Slopes	0	–5	–25	–50	–60

Table 10.4.6.4-3—Geomechanics Rock Mass Classes Determined from Total Ratings

RMR Rating	100–81	80–61	60–41	40–21	<20
Class No.	I	II	III	IV	V
Description	Very good rock	Good rock	Fair rock	Poor rock	Very poor rock

The shear strength of fractured rock masses should be evaluated using the Hoek and Brown criteria, in which the shear strength is represented as a curved envelope that is a function of the uniaxial compressive strength of the intact rock, q_u, and two dimensionless constants m and s. The values of m and s as defined in Table 10.4.6.4-4 should be used.

The shear strength of the rock mass should be determined as:

$$\tau = \left(\cot\ \phi'_i - \cos\ \phi'_i\right) m\ \frac{q_u}{8} \tag{10.4.6.4-1}$$

in which:

$$\phi'_i = \tan^{-1}\left\{4h\ \cos^2\left[30 + 0.33\ \sin^{-1}\left(h^{\frac{-3}{2}}\right)\right] - 1\right\}^{\frac{-1}{2}}$$

$$h = 1 + \frac{16\left(m\sigma'_n + sq_u\right)}{(3m^2 q_u)}$$

where:

τ = the shear strength of the rock mass (ksf)

ϕ'_i = the instantaneous friction angle of the rock mass (degrees)

q_u = average unconfined compressive strength of rock core (ksf)

σ'_n = effective normal stress (ksf)

m, s = constants from Table 10.4.6.4-4 (dim)

This method was developed by Hoek (1983) and Hoek and Brown (1988, 1997). Note that the instantaneous cohesion at a discrete value of normal stress can be taken as:

$$c_i = \tau - \sigma'_n\ \tan\phi'_i \tag{C10.4.6.4-1}$$

The instantaneous cohesion and instantaneous friction angle define a conventional linear Mohr envelope at the normal stress under consideration. For normal stresses significantly different than that used to compute the instantaneous values, the resulting shear strength will be unconservative. If there is considerable variation in the effective normal stress in the zone of concern, consideration should be given to subdividing the zone into areas where the normal stress is relative constant and assigning separate strength parameters to each zone. Alternatively, the methods of Hoek (1983) may be used to compute average values for the range of normal stresses expected.

Table 10.4.6.4-4—Approximate Relationship between Rock-Mass Quality and Material Constants Used in Defining Nonlinear Strength (Hoek and Brown, 1988)

Rock Quality	Constants	Rock Type A = Carbonate rocks with well developed crystal cleavage—*dolomite, limestone and marble* B = Lithified argrillaceous rocks—*mudstone, siltstone, shale and slate (normal to cleavage)* C = Arenaceous rocks with strong crystals and poorly developed crystal cleavage—*sandstone and quartzite* D = Fine grained polymineralic igneous crystalline rocks—*andesite, dolerite, diabase and rhyolite* E = Coarse grained polymineralic igneous & metamorphic crystalline rocks—*amphibolite, gabbro gneiss, granite, norite, quartz-diorite*				
		A	B	C	D	E
INTACT ROCK SAMPLES Laboratory size specimens free from discontinuities. CSIR rating: *RMR* = 100	*m* *s*	7.00 1.00	10.00 1.00	15.00 1.00	17.00 1.00	25.00 1.00
VERY GOOD QUALITY ROCK MASS Tightly interlocking undisturbed rock with unweathered joints at 3–10 ft CSIR rating: *RMR* = 85	*m* *s*	2.40 0.082	3.43 0.082	5.14 0.082	5.82 0.082	8.567 0.082
GOOD QUALITY ROCK MASS Fresh to slightly weathered rock, slightly disturbed with joints at 3–10 ft CSIR rating: *RMR* = 65	*m* *s*	0.575 0.00293	0.821 0.00293	1.231 0.00293	1.395 0.00293	2.052 0.00293
FAIR QUALITY ROCK MASS Several sets of moderately weathered joints spaced at 1–3 ft CSIR rating: *RMR* = 44	*m* *s*	0.128 0.00009	0.183 0.00009	0.275 0.00009	0.311 0.00009	0.458 0.00009
POOR QUALITY ROCK MASS Numerous weathered joints at 2 to 12 in.; some gouge. Clean compacted waste rock. CSIR rating: *RMR* = 23	*m* *s*	0.029 3×10^{-6}	0.041 3×10^{-6}	0.061 3×10^{-6}	0.069 3×10^{-6}	0.102 3×10^{-6}
VERY POOR QUALITY ROCK MASS Numerous heavily weathered joints spaced <2 in. with gouge. Waste rock with fines. CSIR rating: *RMR* = 3	*m* *s*	0.007 1×10^{-7}	0.010 1×10^{-7}	0.015 1×10^{-7}	0.017 1×10^{-7}	0.025 1×10^{-7}

Where it is necessary to evaluate the strength of a single discontinuity or set of discontinuities, the strength along the discontinuity should be determined as follows:

- For smooth discontinuities, the shear strength is represented by a friction angle of the parent rock material. To evaluate the friction angle of this type of discontinuity surface for design, direct shear tests on samples should be performed. Samples should be formed in the laboratory by cutting samples of intact core.
- For rough discontinuities the nonlinear criterion of Barton (1976) should be applied.

The range of typical friction angles provided in Table C10.4.6.4-1 may be used in evaluating measured values of friction angles for smooth joints.

Earth Pressures

Evaluation of Earth Pressure Coefficients for Substructure Design

Assumed Backfill Values

MaineDOT BDG Section 3.6.1 - Soil Type 4

$\gamma_1 := 125\ pcf$	Unit Weight
$\phi_1 := 32\ deg$	Friction Angle
$c_1 := 0\ psf$	Cohesion

Wall Parameters

$\theta := 90\ deg$	Angle of back face of wall (from horizontal)
$\delta := \frac{2}{3} \cdot \phi_1 = 21.3\ deg$	Interface Friction between Fill and Wall LRFD Table 3.11.5.3-1, δ = 19 to 24 deg
$\beta := 0\ deg$	Continuous Backslope Angle(s) (from horizontal)

Coulomb Active Earth Pressure Coefficient (LRFD Eq. 3.11.5.3-1 and 3.11.5.3-2)

$$\Gamma_a := \left(1 + \sqrt{\frac{\sin(\phi_1 + \delta) \cdot \sin(\phi_1 - \beta)}{\sin(\theta - \delta) \cdot \sin(\theta + \beta)}}\right)^2$$

$$k_a := \frac{\sin(\theta + \phi_1)^2}{\Gamma_a \cdot (\sin(\theta))^2 \cdot (\sin(\theta - \delta))} \qquad k_a = 0.28$$

At-Rest Earth Pressure Coefficient (LRFD Eq. 3.11.5.2-1)

$$k_o := 1 - \sin(\phi_1) \qquad k_o = 0.47$$

Seismic

Determine Seismic Site Classification per AASHTO LRFD Table C3.10.3.1-1 - Method B

Data From Boring BB-FCSR-101

Layer No.	Layer Description	Depth Range (ft)		N_{60} values recorded within layer								Average N_{60} value	Layer Thickness	d_i/N_i
		Top	End									Ni	di	
1	Fill	0	10	20	13							16.5	10	0.61
2	Alluvium	10	29.5	4	3	51						19.3	19.5	1.01
3	Bedrock	29.5	100	100								100.0	70.5	0.71
												Σ =	**100**	**2.32**

N_bar = di/di/Ni = 43.11
Site Class D

Data From Boring BB-FCSR-102

Layer No.	Layer Description	Depth Range (ft)		N_{60} values recorded within layer								Average N_{60} value	Layer Thickness	d_i/N_i
		Top	End									Ni	di	
1	Alluvium	0	0.5	40								40.0	0.5	0.01
2	Glacial Till	0.5	9	100								100.0	8.5	0.09
3	Bedrock	9	100	100								100.0	91	0.91
												Σ =	**100**	**1.01**

N_bar = di/di/Ni = 99.26
Site Class C

Data From Boring BB-FCSR-103

Layer No.	Layer Description	Depth Range (ft)		N_{60} values recorded within layer								Average N_{60} value	Layer Thickness	d_i/N_i
		Top	End									Ni	di	
1	Fill	0	14.5	40	7	39						28.7	14.5	0.51
3	Bedrock	14.5	100	100								100.0	85.5	0.86
												Σ =	**100**	**1.36**

N_bar = di/di/Ni = 73.49
Site Class C

NOTES:
1. Weight of rod (WOR) and weight of hammer (WOH) values taken as N=1
2. N60 values > 100 taken as N=100
3. N60 value for bedrock taken as N=100

Calculated by / Date: MAS / March 2020
Checked by: TJB / March 2020

Farmington Falls Bridge #2273 Replacement

Rt 41 over Sandy River

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
AASHTO Spectrum for 7% PE in 75 years
Latitude = 44.620080
Longitude = -070.074840
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.081	PGA - Site Class B
0.2	0.169	Ss - Site Class B
1.0	0.048	S1 - Site Class B

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Spectral Response Accelerations SDs and SD1
Latitude = 44.620080
Longitude = -070.074840
As = FpgaPGA, SDs = FaSs, and SD1 = FvS1
Site Class D - Fpga = 1.60, Fa = 1.60, Fv = 2.40
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	
0.0	0.130	As - Site Class D
0.2	0.271	SDs - Site Class D
1.0	0.115	SD1 - Site Class D

Rt 41 over Sandy River

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Map Response Spectra for Site Class B
Latitude = 44.620080
Longitude = -070.074840
Ss and S1 = Mapped Spectral Acceleration Values
Site Class B
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	Sd in.	
0.000	0.081	0.000	T = 0.0, Sa = PGA
0.057	0.169	0.005	T = To, Sa = Ss
0.200	0.169	0.066	T = 0.2, Sa = Ss
0.283	0.169	0.133	T = Ts, Sa = Ss
0.300	0.160	0.141	
0.400	0.120	0.187	
0.600	0.080	0.281	
0.800	0.060	0.375	
1.000	0.048	0.468	T = 1.0, Sa = S1
1.200	0.040	0.562	
1.400	0.034	0.656	

1.600	0.030	0.750
1.800	0.027	0.843
2.000	0.024	0.937
2.200	0.022	1.031
2.400	0.020	1.124
2.600	0.018	1.218
2.800	0.017	1.312
3.000	0.016	1.405
3.200	0.015	1.499
3.400	0.014	1.593
3.600	0.013	1.686
3.800	0.013	1.780
4.000	0.012	1.874

Conterminous 48 States
2007 AASHTO Bridge Design Guidelines
Design Response Spectra for Site Class D
Latitude = 44.620080
Longitude = -070.074840
As = FpgaPGA, SDs = FaSs, SD1 = FvS1
Site Class D - Fpga = 1.60, Fa = 1.60, Fv = 2.40
Data are based on a 0.05 deg grid spacing.

Period (sec)	Sa (g)	Sd in.	
0.000	0.130	0.000	T = 0.0, Sa = As
0.085	0.271	0.019	
0.200	0.271	0.106	T = 0.2, Sa = SDs
0.425	0.271	0.478	T = Ts, Sa = SDs
0.500	0.230	0.562	
0.600	0.192	0.675	
0.800	0.144	0.899	
1.000	0.115	1.124	T = 1.0, Sa = SD1
1.200	0.096	1.349	
1.400	0.082	1.574	
1.600	0.072	1.799	
1.800	0.064	2.024	
2.000	0.058	2.249	
2.200	0.052	2.473	
2.400	0.048	2.698	
2.600	0.044	2.923	
2.800	0.041	3.148	
3.000	0.038	3.373	
3.200	0.036	3.598	
3.400	0.034	3.823	
3.600	0.032	4.047	
3.800	0.030	4.272	
4.000	0.029	4.497	

Frost Depth

Estimated Frost Penetration Depth

Based on MaineDOT Bridge Design Guide Section 5.2.1

Site Location: Farmington-Chesterville, Maine

Soil Conditions: SAND, some to little silt, some to little gravel
(Coarse Grained)

Step 1. From Figure 5-1: Design Freezing Index = ±1800 freezing degree-days

Step 2. From laboratory test results:
soil water content: ??%
USE WC = 10%

Step 3. From Table 5-1: interpolate frost penetration for w = 10%

$$DFI := 1800$$

$$DFI_1 := 1700 \quad d_1 := 87.5\ in$$

$$DFI_2 := 1800 \quad d_2 := 90.1\ in$$

$$d_{frost} := d_1 + (d_2 - d_1) \cdot \left(\frac{DFI - DFI_1}{DFI_2 - DFI_1} \right)$$

$$d_{frost} = 90.1\ in$$

$$d_{frost} = 7.5\ ft$$

Calculated by / Date: MAS / March 2020
Checked by: TJB / March 2020

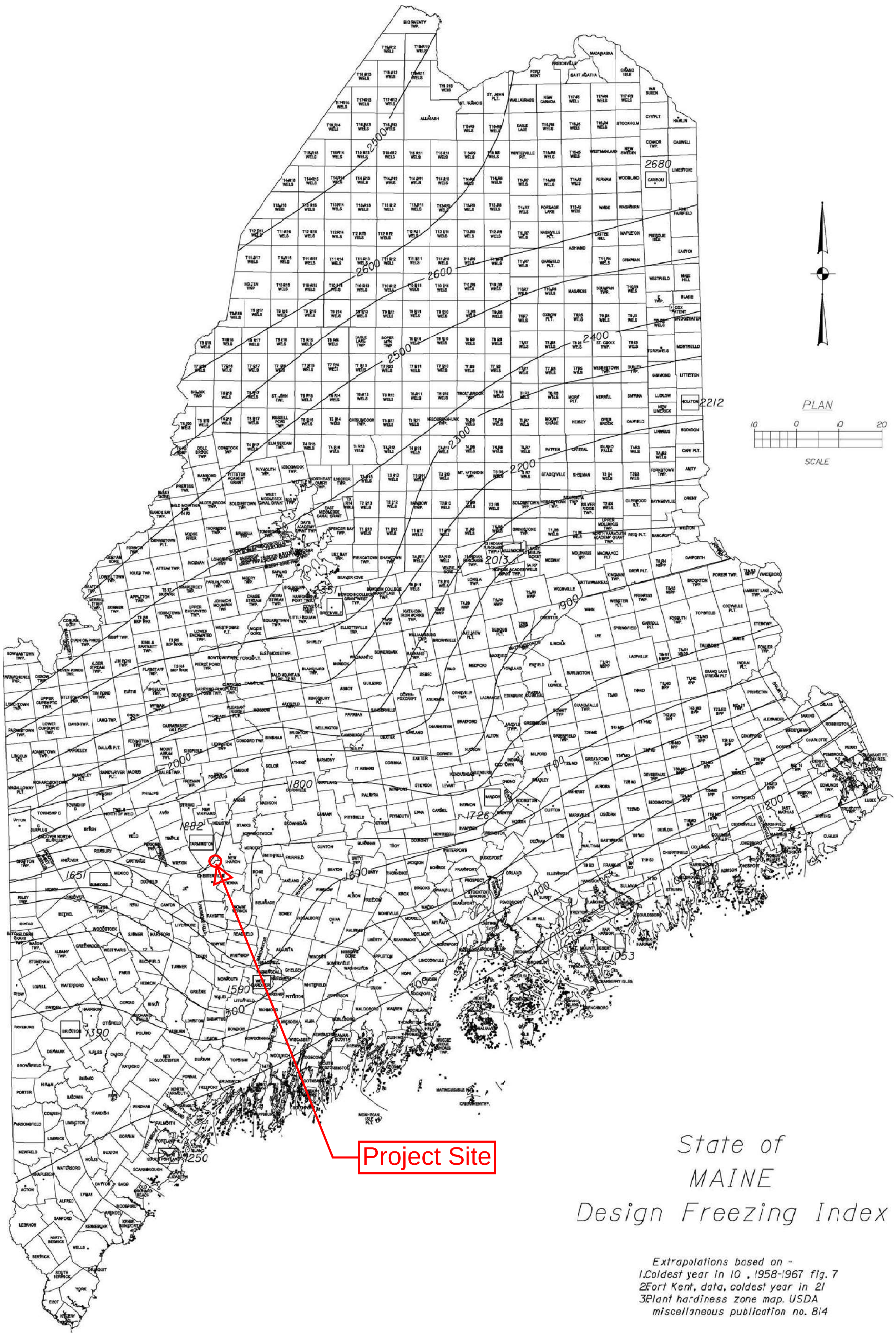

Figure 5-1 Maine Design Freezing Index Map

5.2 General

5.2.1 Frost

Any foundation placed on seasonally frozen soils must be embedded below the depth of frost penetration to provide adequate frost protection and to minimize the potential for freeze/thaw movements. Fine-grained soils with low cohesion tend to be most frost susceptible. Soils containing a high percentage of particles smaller than the No. 200 sieve also tend to promote frost penetration.

In order to estimate the depth of frost penetration at a site, Table 5-1 has been developed using the Modified Berggren equation and Figure 5-1 Maine Design Freezing Index Map. The use of Table 5-1 assumes site specific, uniform soil conditions where the Geotechnical Designer has evaluated subsurface conditions. Coarse-grained soils are defined as soils with sand as the major constituent. Fine-grained soils are those having silt and/or clay as the major constituent. If the make-up of the soil is not easily discerned, consult the Geotechnical Designer for assistance. In the event that specific site soil conditions vary, the depth of frost penetration should be calculated by the Geotechnical Designer.

Table 5-1 Depth of Frost Penetration

Design Freezing Index	Frost Penetration (in)					
	Coarse Grained			Fine Grained		
	w=10%	w=20%	w=30%	w=10%	w=20%	w=30%
1000	66.3	55.0	47.5	47.1	40.7	36.9
1100	69.8	57.8	49.8	49.6	42.7	38.7
1200	73.1	60.4	52.0	51.9	44.7	40.5
1300	76.3	63.0	54.3	54.2	46.6	42.2
1400	79.2	65.5	56.4	56.3	48.5	43.9
1500	82.1	67.9	58.4	58.3	50.2	45.4
1600	84.8	70.2	60.3	60.2	51.9	46.9
1700	87.5	72.4	62.2	62.2	53.5	48.4
1800	90.1	74.5	64.0	64.0	55.1	49.8
1900	92.6	76.6	65.7	65.8	56.7	51.1
2000	95.1	78.7	67.5	67.6	58.2	52.5
2100	97.6	80.7	69.2	69.3	59.7	53.8
2200	100.0	82.6	70.8	71.0	61.1	55.1
2300	102.3	84.5	72.4	72.7	62.5	56.4
2400	104.6	86.4	74.0	74.3	63.9	57.6
2500	106.9	88.2	75.6	75.9	65.2	58.8
2600	109.1	89.9	77.1	77.5	66.5	60.0

Notes: 1. w = water content
2. Where the Freezing Index and/or water content is between the presented values, linear interpretation may be used to determine the frost penetration.