

PRELIMINARY DESIGN REPORT

16-1407

May 2, 2017

Explorations and Geotechnical Engineering Services

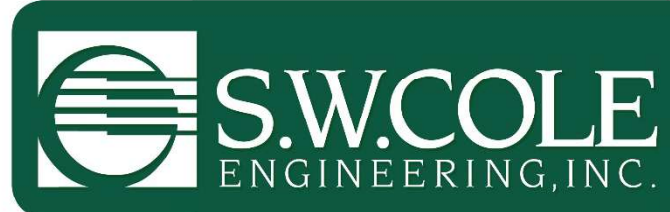
Benjamin River Bridge #3216 Replacement
Route 175 over Benjamin River
Brooklin-Sedgwick, Maine
WIN 021677.00

PREPARED FOR:

Kleinfelder
Attention: Jim Wentworth, PE
151 Capitol Street, Suite 2
Augusta, ME 04330

PREPARED BY:

S. W. Cole Engineering, Inc.
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- *Geotechnical Engineering*
- *Construction Materials Testing and Special Inspections*
- *GeoEnvironmental Services*
- *Test Boring Explorations*

www.swcole.com

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Subject: Preliminary Design Report
Explorations and Geotechnical Engineering Services
Benjamin River Bridge #3216 Replacement
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Dear Jim:

In accordance with our Proposal dated December 7, 2016, we have made the requested subsurface explorations 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 services provided by S. W. Cole Engineering, Inc. (S.W.COLE) and contents of this report are subject to the limitations in Appendix A.

1.0 INTRODUCTION

1.1 Site Conditions

The site is Benjamin River Bridge (MaineDOT Bridge #3216) located on Route 175 (Reach Road) at the crossing of Benjamin River separating Brooklin and Sedgwick, Maine. The site location is shown on the "Site Location Map" attached in Appendix B. Based on the provided historic bridge plans, we understand the existing bridge was constructed in 1935, the superstructure was replaced in 1967 and widened in 1979. The existing structure is ± 25 feet long (end-to-end) and ± 30.6 feet wide (out-to-out). The existing bridge is a single-span structure and consists of steel girders with concrete



cast-in-place structural deck. The existing abutments and wing walls consist of stacked granite blocks.

A Highway Bridge Inspection Report dated January 14, 2016 indicates the structure is in overall “poor” condition with the deck, superstructure, and substructure rated as 4. The structure displays signs of advanced deterioration and is scour critical. The existing structure has a Sufficiency Rating of 65.7 and is listed as “Structurally Deficient.”

1.2 Proposed Construction

Based on discussions with you, we understand the project is scoped as an on-alignment bridge replacement with zero skew. We understand the replacement structure will be a 40 to 50-foot single-span, integral abutment structure founded on a single row of H-piles. The proposed abutments will be constructed behind the existing stacked block abutments requiring removal of the upper portions of the existing abutments. We understand the remaining granite blocks will stay in place to provide scour protection. We understand 1.75:1 (H:V) riprap slopes will be placed in front of the new integral abutments for additional scour protection. We understand the riprap will key into the upper portions of the existing granite block abutments.

We understand the new bridge will be designed to match the existing horizontal alignment and the vertical profile will be within 2 feet of the existing structure.

2.0 EXPLORATIONS AND TESTING

2.1 Explorations

Three test borings (BB-BSBR-101, BB-BSBR-102 and BB-BSBR-102A) were made at the site on February 8, 2017 and February 20 through 22, 2017 by S. W. Cole Explorations, LLC, a subsidiary of S.W.COLE. The exploration locations were selected in consultation with Kleinfelder and established in the field by S.W.COLE using taped measurements from existing site features. The as-drilled exploration locations are shown on the “Boring Location Plan” attached in Appendix B. The ground surface elevations shown on the logs were estimated based on the ground surface contours shown on the “Boring Location Plan.” 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 NQ2 rock core drilling techniques. The soils were sampled at 5-foot intervals using a split-spoon sampler and Standard Penetration Testing (SPT) techniques using a calibrated automatic hammer. The uncorrected SPT blow counts and uncorrected and corrected SPT N-values are shown on the logs.

Soils samples recovered from the test borings were returned to our laboratory for further visual classification and testing. Laboratory testing was performed on disturbed SPT samples obtained during the explorations in accordance with applicable American Association of State Highway and Transportation Officials (AASHTO) testing procedures. Laboratory testing included six (6) natural water content tests and eight (8) grain size analyses. A summary and results of the laboratory testing are provided in Appendix D.

3.0 SUBSURFACE CONDITIONS

3.1 Surficial and Bedrock Geology

According to the Maine Geological Survey's (MGS's) mapping of the Blue Hill Quadrangle, Maine¹, surficial geologic units within the site vicinity consist of glacial-marine deposits (sand, silt, and clay) and glacial till. The subsurface conditions encountered were generally consistent with the mapped surficial geology. The explorations generally encountered a surface deposit of fill soils from previous site development overlying glacial-marine sands overlying glacial till.

According to MGS Bedrock Geology Map of Maine², bedrock in the site vicinity is mapped as Ordovician to Precambrian age interbedded grayish-green phyllite and white to gray, siliceous, feldspathic siltstone of the Ellsworth Formation. The bedrock where sampled was generally composed of dark greenish-grey meta-sandstone/schist and is generally consistent with the mapped bedrock geology.

¹ Thompson, W. B. and Borns, H. W., Jr., 1977, Reconnaissance surficial geology of the Blue Hill [15-minute] quadrangle, Maine, Maine Geological Survey, Open-File Map 77-36.

² Osberg, P. H., Hussey, A. M., II, and Boone, G. M., 1985, Bedrock geologic map of Maine, Maine Geological Survey.

3.2 Subsurface Conditions

The test borings encountered a soils profile generally consisting of a surface layer of pavement overlying fill overlying glacial-marine sands overlying bedrock with depth. An “Interpretive Subsurface Profile” is attached in Appendix B. Refer to the exploration logs in Appendix C for more detailed descriptions of the subsurface findings at the exploration locations. The principal strata encountered in the explorations are summarized below.

Fill: Below an approximate 5 inch layer of pavement, the borings encountered fill with occasional cobbles and small (14 to 16-inch diameter) boulders extending to depths of about 14 to 16 feet below ground surface (bgs) corresponding to Elevation (El.) -5.5 to -7.7 feet. The fill generally consisted of:

- Brown, moist, SAND, some to little gravel, trace silt;
- Brown, wet, silty SAND, trace gravel with black shell fragments; and
- Brown, wet, sandy SILT.

Organics (wood and wood fiber) were encountered at a depth of about 10.2 to 10.5 feet bgs (El. -1.7 to -2.0 feet) in BB-BSBR-101 and from about 11.3 to 16 feet bgs (El. -3.0 to -7.7 feet) in BB-BSBR-102.

The fill was generally medium dense to dense with SPT N_{60} values ranging from 11 to 34 blows per foot (bpf).

Below the layer of organics in test boring BB-BSBR-102, a layer of possible fill was encountered from about 16 to 25 feet bgs (El. -7.7 to -16.7 feet). The soil in this layer was generally described as brown, wet, Gravelly SAND little silt. The Fill was generally very dense with SPT N_{60} values ranging from 51 to 146 bpf.

Marine Sand: Below the fill, the borings encountered marine sand to a depth of about 39 to 41 feet bgs (El. -30.7 to -32.5 feet). The marine sand material generally consisted of:

- Grey, wet, SAND, some to little silt, little to trace gravel;
- Brown, wet, SAND, trace silt, trace gravel with occasional cobbles;
- Brown, wet, gravelly SAND, little silt with occasional cobbles; and
- Grey to brown, wet, SAND, some gravel, some silt.



The marine sand was generally very loose to very dense with SPT N_{60} values ranging from 3 to 156 bpf.

Bedrock: Bedrock was encountered and sampled in borings BB-BSBR-101 and BB-BSBR-102A. The top of bedrock varied from about 39 to 41 feet bgs (El. -30.7 to -32.5). More competent bedrock was encountered at about 40 and 41.7 feet bgs (El. -31.7 to -33.2). The bedrock consisted of dark greenish-grey, moderately hard, very slightly weathered, meta-sandstone/schist of the Ellsworth Formation. Rock quality designation (RQD) values for the bedrock generally ranged from 0 to 93 percent correlating to a Rock Mass Quality (RMQ) of very poor to excellent. 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 moist from the ground surface. Measured water levels made immediately after drilling ranged from 10.1 to 10.7 feet below ground surface. It should be noted that Benjamin River is tidally influenced and water was introduced during drilling therefore, water levels indicated may not represent stabilized ground water conditions. 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, changes in site use and the water level of Benjamin River.

4.0 PRELIMINARY GEOTECHNICAL EVALUATION

S.W. COLE conducted geotechnical engineering evaluations in accordance with 2014 AASHTO LRFD Bridge Design Specifications, 7th Edition with 2016 interim revisions (AASHTO LRFD) and the MaineDOT Bridge Design Guide, 2003 Edition with revisions through August 2014 (MaineDOT BDG) and offers the following:

4.1 Foundation Options and Discussion

Based on the MaineDOT Highway Bridge Inspection Report dated January 14, 2016, we understand the existing bridge is in “poor” condition and scour critical. Due to the potential for scour, spread footing foundations would have to extend below design scour depths and therefore are not considered feasible for support of the proposed bridge.



Therefore, the new structure will be founded on deep foundations (driven piles or drilled shafts).

We understand driven piles are the preferred foundation option. The following sections provide preliminary geotechnical design considerations and recommendation for an H-pile supported integral bridge abutments.

4.2 Integral Abutment H-Piles

Abutments No. 1 (West) and No. 2 (East) will be integral abutments founded on a single row of steel H-piles. The piles shall be end bearing on or within bedrock and driven to the required resistance. Based on the MaineDOT BDG Section 5.4.2.1, piles may include HP 10x42, 12x53, 14x73 or 14x89. Additional pile sections may be considered depending on the factored design axial loads. S.W.COLE can provide additional input on pile size once pile loading has been developed for the proposed structure.

H-piles shall be 50 ksi, Grade A572 steel with cast driving points conforming to MaineDOT Standard Specification 711.10 to help reduce damage to the piles during driving and improve penetration. We recommend pile points consist of Associated Pile and Fitting, Rock Injector, HP-80500 Pile Points.

Based on the depths bedrock was encountered at the borings we estimated the following pile lengths:

Location	Assumed Abutment Subgrade Elevation (feet)	Approximate Top of Competent Bedrock Elevation (feet)	Estimated Pile Length (feet)
Abutment No. 1	4.0	-33.2	37
Abutment No. 2	4.0	-31.7	36

The estimated pile lengths do not take into account locations where bedrock may be deeper or shallower than that encountered in the test borings, damaged pile, the additional five (5) feet of pile required for dynamic testing instrumentation (per ASTM D4945), additional pile length needed to accommodate leads and driving equipment, or additional pile length needed for embedment in the abutment or pile cap.

A five-foot section of 4.0-inch steel casing was abandoned in BB-BSBR-102 after breaking on what is assumed to be cobbles or boulders. The proposed piles should be located to avoid conflict with the abandoned steel casing.

4.2.1 Strength Limit State Design

Design of pile foundations bearing on or within bedrock at the strength limit state shall consider;

- Compressive axial geotechnical resistance of individual piles bearing on bedrock;
- Drivability resistance of individual piles driven to bedrock;
- Structural resistance of individual piles in axial compression, and;
- Structural resistance of individual piles in combined axial loading and flexure.

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

Per LRFD Article 6.5.4.2, at the strength limit state, the axial resistance factor $\phi_c = 0.50$, for severe driving conditions shall be applied to the structural compressive resistance of the pile. The H-piles will be subjected to lateral loading, therefore the piles shall be evaluated for resistance against combined axial compression and flexure in accordance with LRFD Articles 6.9.2.2 and 6.15.2.

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

Abutment H-piles should be analyzed for determination of unbraced lengths and fixity using LPILE® 2016 (LPILE) software. The calculated unbraced lengths should be used to analyze the piles in combined axial compression and flexure resistance provided in LRFD Articles 6.9.2.2 and 6.15.2.

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

as determined from L-Pile once structural loads are available. The nominal axial structural resistance should be evaluated based on combined axial compression and flexure.

Geotechnical Resistance. The nominal axial geotechnical resistance in the strength limit state should be calculated using the guidance in LRFD Article 10.7.3.2.3 which states the nominal bearing resistance of piles driven to point bearing on hard rock shall not exceed the structural pile resistances obtained from LRFD Article 6.9.4.1 with a resistance factor ϕ_c , of 0.50, for severe driving conditions. For non-displacement piles driven to end bearing on or within bedrock, it is our experience that this is generally not the controlling resistance.

Drivability Analyses. Drivability analyses shall be performed to determine the pile resistance that might be achieved considering available diesel hammers. The maximum driving stresses in the pile, assuming the use of 50 ksi steel, shall be less than 45 ksi. The drivability resistances shall be calculated using the resistance factor, ϕ_{dyn} , of 0.65, for a single pile in axial compression when a dynamic test is performed as specified in LRFD Table 10.5.5.2.3-1.

4.2.2 Service and Extreme Limit State Design

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

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

4.2.3 Lateral Pile Resistance/Behavior

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

Lateral pile analyses shall be performed by the geotechnical engineer to evaluate pile behavior at both abutments using LPILE software with pile head deflections, moments, and axial loads supplied by the structural engineer.

4.2.4 Driven Pile Resistance and Pile Quality Control

The contract documents shall require the contractor to perform a wave equation analysis for the proposed pile-hammer system and conduct dynamic pile load tests with signal matching. The first pile driven at each abutment should be dynamically tested to confirm nominal pile resistance and verify the stopping criteria developed by the contractor in the wave equation analysis. Minimum 24-hour restrrike tests will be required and should be noted on the plans. Additional dynamic tests may be required as part of the pile field quality control program if:

- Pile behavior vary radically between adjacent piles;
- Pile behavior indicates pile refusal on a boulder or in a cobble layer above bedrock;
- Pile tip is not firmly embedded in bedrock, or
- Pile is out of tolerance.

Piles should be driven to an acceptable penetration resistance based on the results of a wave equation analysis provided by the contractor and as approved by the design team. Pile load testing should be completed by PDA testing with signal matching including one pile at each abutment. Driving stresses in the pile determined in the drivability analysis and confirmed by PDA testing shall be less than 45 ksi, in accordance with LRFD Article 10.7.8. The pile hammer should be selected such that the required pile resistance when the penetration resistance for the final 3 to 6 inches is between 3 to 15 blows per inch (bpi). If an abrupt increase in driving resistance is encountered, the driving may be terminated when the penetration is less than 0.5-inch in 10 consecutive blows. Termination criteria shall be confirmed and evaluated for the selected pile hammer.

4.3 Integral Abutment Design

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

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

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

The designer may assume Soil Type 4 (MaineDOT Bridge Design Guide (BDG) Section 3.6.1) for abutment backfill material soil properties. The backfill properties are as follows:

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

Integral abutment sections shall be designed to withstand a lateral earth load equal to the passive pressure state. Calculation of passive earth pressures should assume a Coulomb passive earth pressure coefficient, K_p , of 6.73. Developing full passive pressure assumes that the ratio of lateral abutment movement to abutment height (y/H) exceeds 0.005. If the calculated displacements are significantly less than that required

to develop full passive pressure the designer may consider using the Rankine passive earth pressure coefficient of 3.25.

Additional lateral earth pressure due to live load surcharge is required per Section 3.6.8 of the MaineDOT BDG for abutments if an approach slab is not specified. When a structural approach slab is specified reduction, not elimination, of the surcharge load is permitted per LRFD Article 3.11.6.5. The live load surcharge may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil (h_{eq}) based on LRFD Table 3.11.6.4-1.

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

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

Slopes in front of the pile supported integral abutments should be constructed with riprap and erosion control geotextile. The riprap slopes should not exceed 1.75:1(H:V) in accordance with MaineDOT Standard Detail 610(03). The 1.75H:1V riprap slopes shall “toe-in” behind the existing reinforced concrete mat foundation.

4.4 Settlement

We understand the current design development will match the existing horizontal alignment and the vertical profile will be within 2 feet of the existing structure. Therefore a minimal increase in overburden pressure is anticipate. Post-construction settlement of new embankment fills and abutment backfills should be elastic and occur during construction with proper compaction.

Post-construction settlement of the pile-supported bridge abutments will be due to axial compression and should not exceed a ½ inch including elastic shortening of the piles.

4.5 Frost Considerations

According to MaineDOT BDG Section 5.2.1, pile supported integral abutments 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 Map³, the design freezing index for the Brooklin-Sedgwick, Maine area is approximately 1,150 freezing degree-days. An assumed water content of 10% was used for coarse grained soils. 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 6 feet. Considering this, we recommend foundations on soil should have at least 6 feet of soil cover to provide frost protection.

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

4.6 Seismic Design Considerations

Seismic site class was evaluated in accordance with AASHTO Section 3.10.3.1 Method B (average N-value method). Based on the subsurface information, the average N-value fell between 15 and 50 bpf corresponding to an AASHTO Site Class D as defined in AASHTO Table 3.10.3.1-1.

The USGS online Seismic Design Maps Tool 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. The results for the project site are summarized below:

³ Maine Department of Transportation, Bridge Design Guide (BDG), August 2003, with Revisions through 2014, Figure 5-1.

RECOMMENDED SEISMIC DESIGN PARAMETERS ⁴	
Site Class	D
PGA	0.055 g
S _s	0.122 g
S ₁	0.039 g
F _{pga}	1.6
F _a	1.6
F _v	2.4
A _s	0.088 g
S _{DS}	0.195 g
S _{D1}	0.094 g
Seismic Zone (based on S _{D1})	Zone 1

NOTE: Site Coordinates: N44.303922, W68.611789

4.7 Recommendations for Scour Evaluation

Based on the laboratory grain size analyses test results, we estimated the average grain diameter corresponding to 50 percent passing by weight (D₅₀) for use in scour evaluations ranges from 0.19 to 0.21 mm. Results of the grain size analyses tests are included in Appendix D and the estimated D₅₀ for samples near the channel elevation are summarized in the following table:

Boring No.	Sample No.	Depth (feet)	Sample Elevation (feet)	Estimated D ₅₀
BB-BSBR-101	4D	15	-6.5	0.19 mm
BB-BSBR-101	5D	20	-11.5	0.21 mm

Design at the strength limit state should consider loss of lateral and vertical support due to scour. Design at the extreme limit state should check that the nominal foundation resistance due to the check flood (Q₅₀₀) 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 the pile-supported abutments, the bridge approach slopes and the abutment slopes will be armored with the existing granite blocks and riprap where the granite blocks are removed. In accordance with MaineDOT BDG Section 2.3.11.3, the top of the riprap shall be located at or above, the Q₅₀ elevation.

⁴ U.S. Geological Survey, Seismic Design Map, , accessed August 29, 2016
<http://earthquake.usgs.gov/designmaps/us/application.php>



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Riprap shall conform to MaineDOT Standard Specification 703.26 "Plain and Hand Laid Riprap." The riprap section shall be underlain by 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.

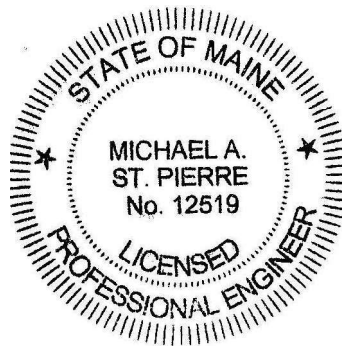
Sincerely,

S. W. Cole Engineering, Inc.

A handwritten signature in black ink, appearing to read 'Michael A. St. Pierre'.

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

MAS/ejb-rec



APPENDIX A LIMITATIONS

This report has been prepared for the exclusive use of the Kleinfelder for specific application to the Benjamin River Bridge #3216 Replacement carrying Route 175 over Benjamin River (MaineDOT WIN 021677.00) in Brooklin-Sedgwick, 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.

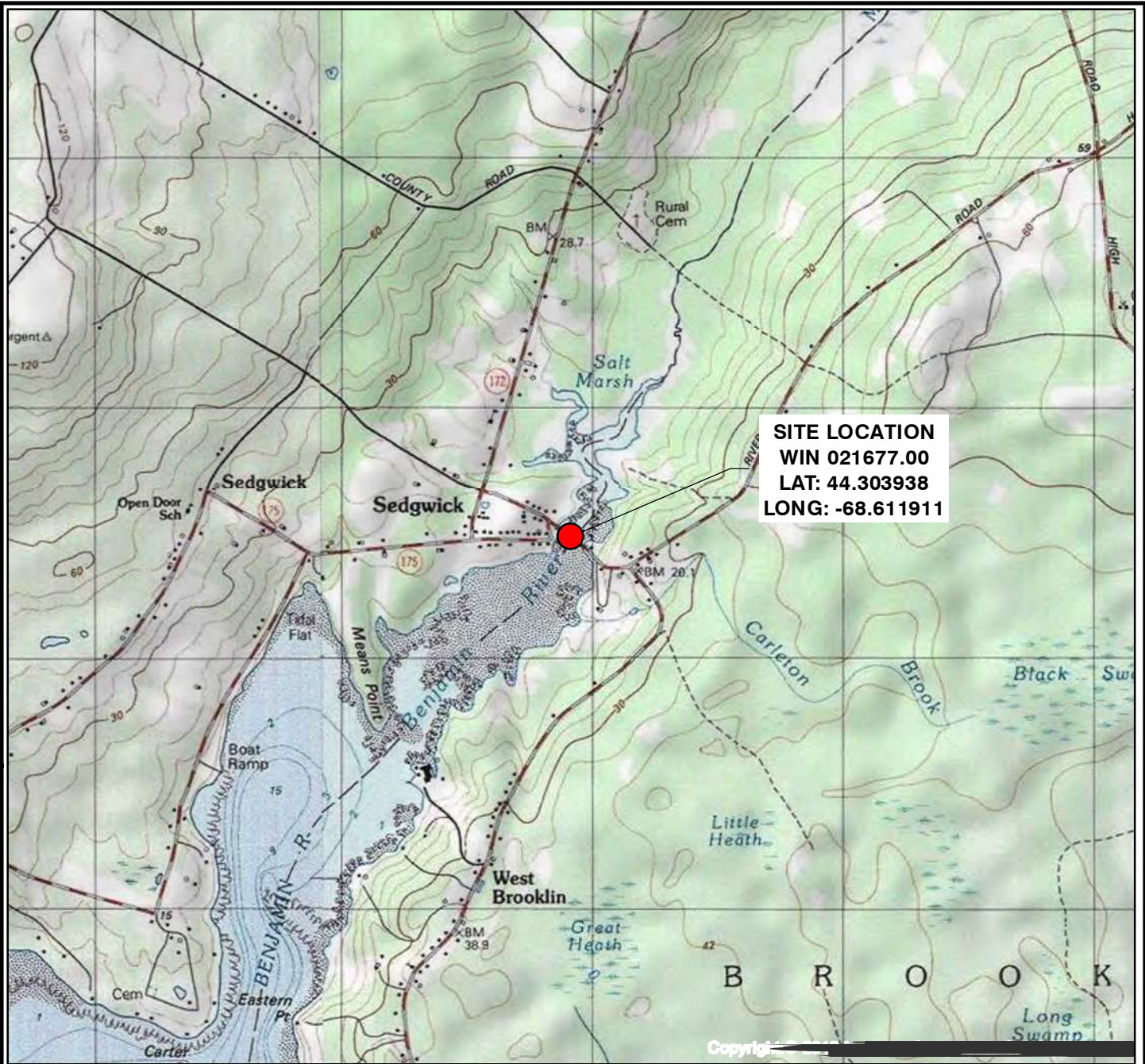
The analyses performed during this investigation and recommendations presented in this report are based in part upon the data obtained from subsurface explorations made at the site. Variations in subsurface conditions may occur between explorations and may not become evident until construction. If variations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

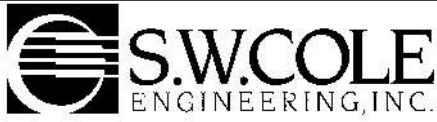
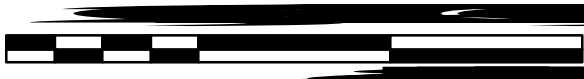
Recommendations contained in this report are based substantially upon information provided by others regarding the proposed project. In the event that any changes are made in the design, nature, or location of the proposed project, S.W.COLE should review such changes as they relate to analyses associated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



APPENDIX B
Sheets



SITE LOCATION
WIN 021677.00
LAT: 44.303938
LONG: -68.611911



KLEINFELDER

BENJAMIN RIVER BRIDGE #3216 REPLACEMENT
ROUTE 175 OVER BENJAMIN RIVER
BROOKLIN-SEDGWICK MAINE
WIN 021677.00

NOTE:

R:\2016\16-1407\GIS\MXDs\16-1407_SLM.mxd, 3/21/2017 10:26:08 AM, 1:24,000.



APPENDIX C
Boring Logs & Key to Soil and Rock Descriptions and Terms

UNIFIED SOIL CLASSIFICATION SYSTEM				MODIFIED BURMISTER SYSTEM																											
MAJOR DIVISIONS		GROUP SYMBOLS	TYPICAL NAMES	Descriptive Term	Portion of Total (%)																										
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	GW Well-graded gravels, gravel-sand mixtures, little or no fines.	trace	0 - 10																										
		(little or no fines)	GP Poorly-graded gravels, gravel sand mixtures, little or no fines.	little	11 - 20																										
	SANDS (more than half of coarse fraction is smaller than No. 4 sieve size)	GRAVEL WITH FINES (Appreciable amount of fines)	GM Silty gravels, gravel-sand-silt mixtures.	some	21 - 35																										
		CLEAN SANDS	SW Well-graded sands, gravelly sands, little or no fines	adjective (e.g. sandy, clayey)	36 - 50																										
		(little or no fines)	SP Poorly-graded sands, gravelly sand, little or no fines.	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). <table border="1"> <thead> <tr> <th>Density of Cohesionless Soils</th> <th>Standard Penetration Resistance N-Value (blows per foot)</th> </tr> </thead> <tbody> <tr><td>Very loose</td><td>0 - 4</td></tr> <tr><td>Loose</td><td>5 - 10</td></tr> <tr><td>Medium Dense</td><td>11 - 30</td></tr> <tr><td>Dense</td><td>31 - 50</td></tr> <tr><td>Very Dense</td><td>> 50</td></tr> </tbody> </table>		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														
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SANDS WITH FINES (Appreciable amount of fines)	SM Silty sands, sand-silt mixtures	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. <table border="1"> <thead> <tr> <th>Consistency of Cohesive soils</th> <th>SPT N-Value (blows per foot)</th> <th>Approximate Undrained Shear Strength (psf)</th> <th>Field Guidelines</th> </tr> </thead> <tbody> <tr><td>Very Soft</td><td>WOH, WOR, WOP, <2</td><td>0 - 250</td><td>Fist easily penetrates</td></tr> <tr><td>Soft</td><td>2 - 4</td><td>250 - 500</td><td>Thumb easily penetrates</td></tr> <tr><td>Medium Stiff</td><td>5 - 8</td><td>500 - 1000</td><td>Thumb penetrates with moderate effort</td></tr> <tr><td>Stiff</td><td>9 - 15</td><td>1000 - 2000</td><td>Indented by thumb with great effort</td></tr> <tr><td>Very Stiff</td><td>16 - 30</td><td>2000 - 4000</td><td>Indented by thumbnail</td></tr> <tr><td>Hard</td><td>>30</td><td>over 4000</td><td>Indented by thumbnail with difficulty</td></tr> </tbody> </table>		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
Consistency of Cohesive soils	SPT N-Value (blows per foot)			Approximate Undrained Shear Strength (psf)	Field Guidelines																										
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Hard	>30	over 4000	Indented by thumbnail with difficulty																												
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.	Rock Quality Designation (RQD): RQD (%) = $\frac{\text{sum of the lengths of intact pieces of core} * > 4 \text{ inches}}{\text{length of core advance}}$ *Minimum NQ rock core (1.88 in. OD of core) Correlation of RQD to Rock Mass Quality <table border="1"> <thead> <tr> <th>Rock Mass Quality</th> <th>RQD (%)</th> </tr> </thead> <tbody> <tr><td>Very Poor</td><td>≤25</td></tr> <tr><td>Poor</td><td>26 - 50</td></tr> <tr><td>Fair</td><td>51 - 75</td></tr> <tr><td>Good</td><td>76 - 90</td></tr> <tr><td>Excellent</td><td>91 - 100</td></tr> </tbody> </table>			Rock Mass Quality	RQD (%)	Very Poor	≤25	Poor	26 - 50	Fair	51 - 75	Good	76 - 90	Excellent	91 - 100														
		Rock Mass Quality				RQD (%)																									
		Very Poor				≤25																									
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Fair	51 - 75																														
Good	76 - 90																														
Excellent	91 - 100																														
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.	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 mass quality (very poor, poor, etc.) ref: ASTM D6032 and AASHTO Standard Specification for Highway Bridges, 17th Ed. Table 4.4.8.1.2A Recovery (inch/inch and percentage) Rock Core Rate (X.X ft - Y.Y ft (min:sec))																													
	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																													
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																											

Driller: S.W. Cole Explorations, LLC	Elevation (ft.): 8.48 ft	Auger ID/OD: 5" Solid Stem
Operator: S. Hollabaugh	Datum: NAVD88	Sampler: Standard Split-Spoon
Logged By: N. Strout	Rig Type: Diedrich D-50	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 2/8/2017-2/20/2017	Drilling Method: Cased-Wash	Core Barrel: NQ-2"
Boring Location: STA. 14+05.00, 5.30 ft RT	Casing ID/OD: HW 4"/4.5" NW 3"/3.5"	Water Level*: 10.1' (after drilling)

Hammer Efficiency Factor: 0.9288 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_U = Peak/Remolded Field Vane Undrained Shear Strength (psf) T_V = Pocket Torvane Shear Strength (psf)
 D = Split Spoon Sample SSA = Solid Stem Auger S_{U(lab)} = Lab Vane Undrained Shear Strength (psf) WC = Water Content, percent
 MD = Unsuccessful Split Spoon Sample Attempt HSA = Hollow Stem Auger q_p = Unconfined Compressive Strength (ksf)
 U = Thin Wall Tube Sample RC = Roller Cone N-uncorrected = Raw Field SPT N-value LL = Liquid Limit
 MU = Unsuccessful Thin Wall Tube Sample Attempt WOH = Weight of 140lb. Hammer Hammer Efficiency Factor = Rig Specific Annual Calibration Value PL = Plasticity Index
 V = Field Vane Shear Test, PP = Pocket Penetrometer WOR/C = Weight of Rods or Casing N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency G = Grain Size Analysis
 MV = Unsuccessful Field Vane Shear Test Attempt WO1P = Weight of One Person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				
0								8.06	5" of Pavement.		
	1D	24/18	1.00 - 3.00	25/15/16/16	--				Brown, moist, dense, SAND, little gravel, trace silt (Fill).	#20325B A-3, SW-SM WC=8.1%	
									Drill action indicates cobbles from ±3.5 to 4.5 ft bgs.		
5	2D	24/15	5.00 - 7.00	3/6/16/63	22	34	OPEN		2D(A) (5-6') Brown, wet, dense, Silty SAND, trace gravel with black shell fragments/ash? (Fill). 2D(B) (6-7') Grey, wet, SAND, some gravel, trace silt (Fill).	#20326B A-3, SW-SM WC=16.5%	
									14" diameter boulder at ±7 ft bgs.		
									16" diameter boulder at ±8.5 ft bgs.		
10	3D	24/5	10.00 - 12.00	1/1/6/8	7	11	46		Black, wet, loose to medium dense Sandy SILT (Probable Fill). 3" layer of organics (wood fiber) at ±10.2 ft bgs.		
									Probable cobbles from ±12 to 14 ft bgs.		
	MD	5/0	12.00 - 12.42	100-5"	--		140		-----		
							113		-----		
							89		-----		
15	4D	24/10	15.00 - 17.00	5/1/1/3	2	3	37		Grey, wet, very loose, fine to medium SAND, some silt, trace gravel (Marine Sand).	#20327B A-3, SM-SC WC=22.8%	
							39		-----		
							49		-----		
							82		-----		
							53		-----		
20	5D	24/13	20.00 - 22.00	8/10/24/59	34	53	52		Brown, wet, very dense, SAND, little silt, trace gravel (Marine Sand).	#20328B A-2-4, SM	
							93		-----		
							OPEN		11" diameter cobble at ±21.9 ft bgs.		
25									Place NW casing.		

Remarks:
 Auto-Hammer #562
 Casing driven using 140# Auto-Hammer.
 Soils frozen to ±2'
 Begin 2/8/2017, resume 2/20/2017 due to weather.
 Water level on 2/20/2017 at low tide.

Driller: S.W. Cole Explorations, LLC	Elevation (ft.): 8.48 ft	Auger ID/OD: 5" Solid Stem
Operator: S. Hollabaugh	Datum: NAVD88	Sampler: Standard Split-Spoon
Logged By: N. Strout	Rig Type: Diedrich D-50	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 2/8/2017-2/20/2017	Drilling Method: Cased-Wash	Core Barrel: NQ-2"
Boring Location: STA. 14+05.00, 5.30 ft RT	Casing ID/OD: HW 4"/4.5" NW 3"/3.5"	Water Level*: 10.1' (after drilling)
Hammer Efficiency Factor: 0.9288	Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>	

Definitions: R = Rock Core Sample S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf) T_v = Pocket Torvane Shear Strength (psf)
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 MV = Unsuccessful Field Vane Shear Test Attempt WO1P = Weight of One Person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
25	MD	24/0	25.00 - 27.00	12/11/11/12	22	34	20			Brown, wet, medium dense, SAND, trace silt, trace gravel (Marine Sand).		
							18					
	6D	24/17	27.00 - 29.00	10/10/9/10	19	29	34					
							23					
30							32			Similar to above except dense.		
	7D	24/8	30.00 - 32.00	5/6/20/9	26	40	15					
							27					
							48					
35							54			Brown, wet, dense, SAND, little silt, trace gravel (Marine Sand).	#20329B A-3, SW-SM	
	8D	23/6	35.00 - 36.92	11/11/11/100-5"	22	34	38					
							96					
							SPUN					
40							63			9D(A) 40.0-41.0 ft: Grey, wet, very dense, SAND, some gravel, some silt, (Marine Sand). 9D(B) 41.0-41.7 ft: Highly Weathered Bedrock. Top of Bedrock at ±Elev. -33.2 ft. Advanced by rollercone through Bedrock from 41.7 to 45.0 ft bgs.		
	9D	30/13	40.00 - 42.50	18/26/75/100-2"	101	156						
							OPEN					
45										R1: Bedrock: Dark greenish-grey, phyllitic, fine to coarse-grained, META-SANDSTONE/SCHIST, moderately hard very slightly weathered, laminated to thinly bedded, moderate to steeply dipping (50 to 70 degrees), tight, cross-cutting joints, local laminations of calcareous sediment with remobilized calcareous mineralization filling foliation plane partings, (Ellsworth Formation). Rock Mass Quality = Poor. R1: Core Times (min:sec) 45.0-46.0 ft (2:55) 46.0-47.0 ft (3:46)		
	R1	40/38	45.00 - 48.33	RQD = 38%			NQ-2					
	R2	20/10	48.30 - 49.97	RQD = 50%								
50												

Remarks:
 Auto-Hammer #562
 Casing driven using 140# Auto-Hammer.
 Soils frozen to ±2'
 Begin 2/8/2017, resume 2/20/2017 due to weather.

Maine Department of Transportation		Project: Benjamin River Bridge #3216 carries Route 175 over Benjamin River	Boring No.: BB-BSBR-102
Soil/Rock Exploration Log US CUSTOMARY UNITS		Location: Brooklin-Sedgwick, Maine	WIN: 21677.00
Driller: S.W. Cole Explorations, LLC	Elevation (ft.): 8.30 ft	Auger ID/OD: 5" Solid Stem	
Operator: S. Hollabaugh	Datum: NAVD88	Sampler: Standard Split-Spoon	
Logged By: N. Strout	Rig Type: Diedrich D-50	Hammer Wt./Fall: 140#/30"	
Date Start/Finish: 2/20/2017-2/21/2017	Drilling Method: Cased-Wash	Core Barrel:	
Boring Location: STA. 13+49.00, 5.25 ft LT	Casing ID/OD: HW 4"/4.5" NW 3"/3.5"	Water Level*:	
Hammer Efficiency Factor: 0.9288	Hammer Type: Automatic <input checked="" type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>		
<small> 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 N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N₆₀ = (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 </small>			

Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
25	7D	24/9	25.00 - 27.00	12/17/21/13	38	59	106	-16.70		Grey, wet, very dense, Gravelly SAND, little silt (Marine Sand).	#20332B A-1-b, SM WC=7.8%	
							255			Cobble from 27 to 27.7 ft and 28.5 to 29.1 feet bgs.		
							50/0" OPEN ↓					
30	8D	24/5	30.00 - 32.00	54/30/21/18	51	79	SPUN	-23.70		Similar to above except brown.		
										Spun NW casing shoe burnt out advancing to 35 ft bgs. Removed NW casing, rollercone and redrive HW casing. HW casing broke at ±27 ft bgs. Abandon boring.		
										Bottom of Exploration at 32.00 feet below ground surface.		
35												
40												
45												
50												

Remarks:
 Auto-Hammer #562
 Casing driven using 140# Auto-Hammer.
 Soils frozen to ±3'

Driller: S.W. Cole Explorations, LLC	Elevation (ft.): 8.30 ft	Auger ID/OD: 5" Solid Stem
Operator: S. Hollabaugh	Datum: NAVD88	Sampler: Standard Split-Spoon
Logged By: N. Strout	Rig Type: Diedrich D-50	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 2/20/2017-2/21/2017	Drilling Method: Cased-Wash	Core Barrel: NQ-2"
Boring Location: STA. 13+47.25, 5.25 ft LT	Casing ID/OD: HW 4"/4.5" NW 3"/3.5"	Water Level*: 10.7' (after drilling)
Hammer Efficiency Factor: 0.9288	Hammer Type: <input checked="" type="checkbox"/> Automatic <input type="checkbox"/> Hydraulic <input type="checkbox"/> Rope & Cathead <input type="checkbox"/>	

Definitions: R = Rock Core Sample S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf) T_v = Pocket Torvane Shear Strength (psf)
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 MV = Unsuccessful Field Vane Shear Test Attempt WO1P = Weight of One Person N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected C = Consolidation Test

Depth (ft.)	Sample Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				
25								-16.70			
30							SPUN				
35	1D	24/8	35.00 - 37.00	12/10/22/40	32	50				Brown, wet, very dense, SAND, some gravel, some silt.	
40								-30.70		Probable Highly Weathered Bedrock.	
40	MD	0/0	40.00 - 40.00	50-0"	--			-31.70		Top of Bedrock at ±Elev. -31.7 ft.	
45	R1	44/44	41.00 - 44.67	RQD = 93%						Advance by rollercone through Bedrock from 40 to 41 ft bgs. R1: Bedrock: Dark greenish-grey, phyllitic, fine to coarse-grained, META-SANDSTONE/SCHIST, moderately hard very slightly weathered, laminated to thinly bedded, moderate to steeply dipping (50 to 70 degrees), tight, cross-cutting joints, local laminations of calcareous sediment with remobilized quartz and calcareous mineralization filling foliation plane partings, (Ellsworth Formation). Rock Mass Quality = Excellent.	
45	R2	60/58	44.70 - 49.70	RQD = 50%						R1: Core Times (min:sec) 41.0-42.0 ft (2:41) 42.0-43.0 ft (2:49) 43.0-44.0 ft (2:22) 44.0-44.7 ft (2:12) 100% Recovery R2: Bedrock: Similar to above. Rock Mass Quality = Poor.	
50	R3	16/16	49.70 - 51.03	RQD = 81%						R2: Core Times (min:sec) 44.7-45.0 ft (0:48) 45.0-46.0 ft (3:52) 46.0-47.0 ft (3:33) 47.0-48.0 ft (3:15)	

Remarks:
Auto-Hammer #562

Driller: S.W. Cole Explorations, LLC	Elevation (ft.): 8.30 ft	Auger ID/OD: 5" Solid Stem
Operator: S. Hollabaugh	Datum: NAVD88	Sampler: Standard Split-Spoon
Logged By: N. Strout	Rig Type: Diedrich D-50	Hammer Wt./Fall: 140#/30"
Date Start/Finish: 2/20/2017-2/21/2017	Drilling Method: Cased-Wash	Core Barrel: NQ-2"
Boring Location: STA. 13+47.25, 5.25 ft LT	Casing ID/OD: HW 4"/4.5" NW 3"/3.5"	Water Level*: 10.7' (after drilling)

Hammer Efficiency Factor: 0.9288 **Hammer Type:** Automatic Hydraulic Rope & Cathead

Definitions: R = Rock Core Sample S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf) T_v = Pocket Torvane Shear Strength (psf)
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Depth (ft.)	Sample Information								Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows					
50									-42.70	[Hatched Box]	48.0-49.0 ft (3:02) 49.0-49.7 ft (3:04) 97% Recovery R3:Bedrock: Similar to above with clastic zones. Rock Mass Quality = Good. R3:Core Times (min:sec) 49.7-50.0 ft (0:55) 50.0-51.0 ft (3:13) 100% Recovery Bottom of Exploration at 51.00 feet below ground surface.	
55												
60												
65												
70												
75												

Remarks:
Auto-Hammer #562



APPENDIX D
Laboratory Test Results



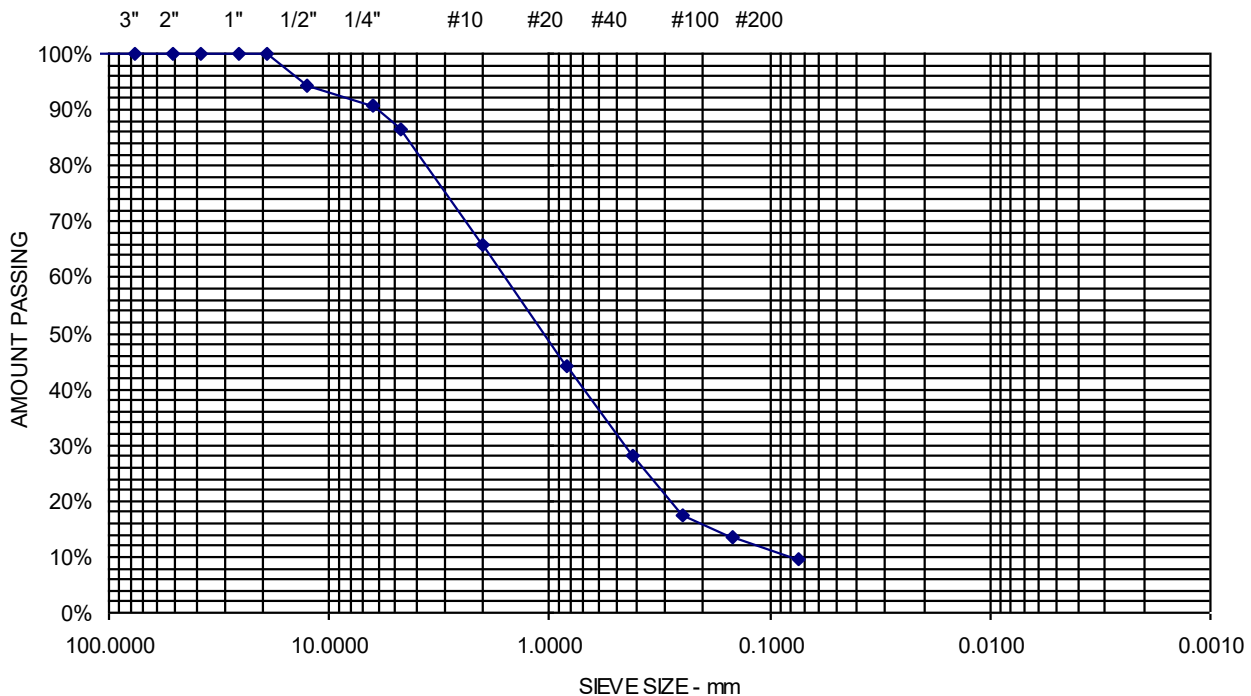
Report of Gradation

AASHTO T11 & T27

Project Name BROOKLIN-SEDGWICK ME - MAINEDOT WIN 21677 BENJAMIN RIVER BRIDGE #3216 REPLACMEENT - GEOTECHNICAL
 Client KLEINFELDER, INC.
 Exploration **BB-BSBR-101**
 Material Source **1D**

Project Number 16-1407
 Lab ID 20325B
 Date Received 3/10/2017
 Date Completed 3/13/2017
 Tested By JASON ORCUTT

<u>STANDARD DESIGNATION (mm/μm)</u>	<u>SIEVE SIZE</u>	<u>AMOUNT PASSING (%)</u>	
150	6"	100	
125	5"	100	
100	4"	100	
75	3"	100	
50	2"	100	
38.1	1-1/2"	100	
25.0	1"	100	
19.0	3/4"	100	
12.5	1/2"	94	
6.3	1/4"	91	
4.75	No. 4	86	13.6% Gravel
2.00	No. 10	66	
850	No. 20	44	
425	No. 40	28	76.6% Sand
250	No. 60	18	
150	No. 100	13	
75	No. 200	9.8	9.8% Fines





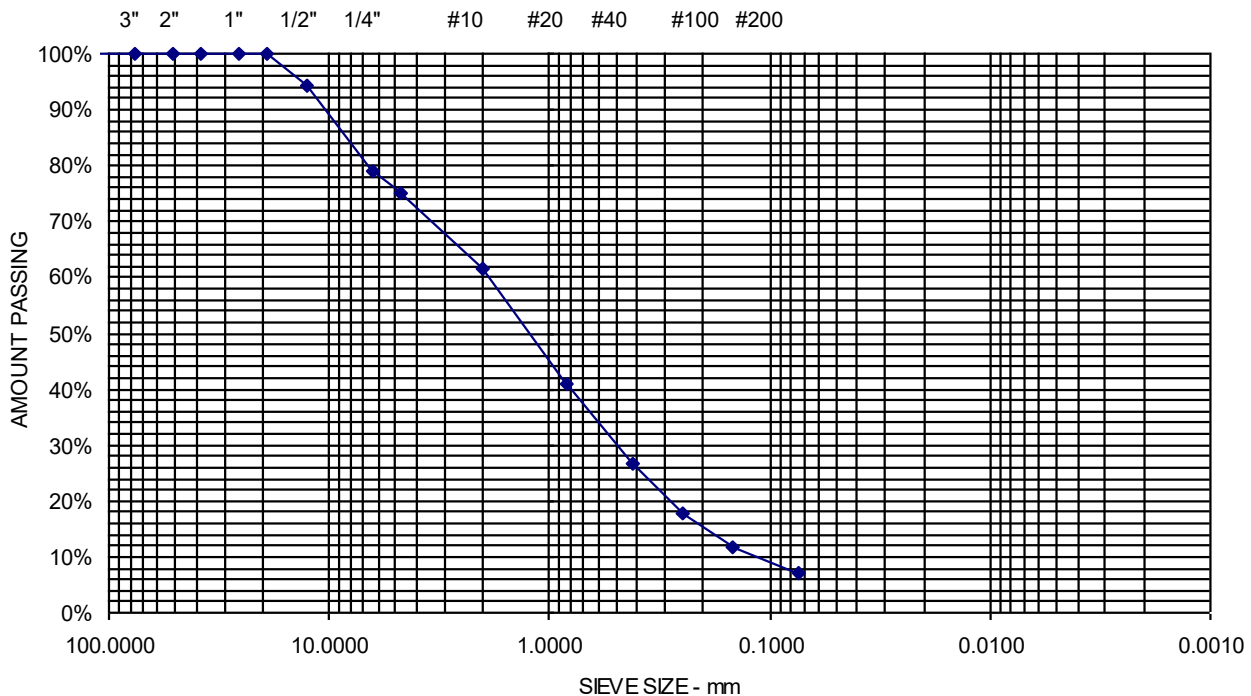
Report of Gradation

AASHTO T11 & T27

Project Name BROOKLIN-SEDGWICK ME - MAINEDOT WIN 21677 BENJAMIN RIVER BRIDGE #3216 REPLACMEENT - GEOTECHNICAL
 Client KLEINFELDER, INC.
 Exploration **BB-BSBR-101**
 Material Source **2D(B)**

Project Number 16-1407
 Lab ID 20326B
 Date Received 3/10/2017
 Date Completed 3/13/2017
 Tested By CHRISTOPHER RAYMOND

<u>STANDARD DESIGNATION (mm/μm)</u>	<u>SIEVE SIZE</u>	<u>AMOUNT PASSING (%)</u>	
150	6"	100	
125	5"	100	
100	4"	100	
75	3"	100	
50	2"	100	
38.1	1-1/2"	100	
25.0	1"	100	
19.0	3/4"	100	
12.5	1/2"	94	
6.3	1/4"	79	
4.75	No. 4	75	25% Gravel
2.00	No. 10	61	
850	No. 20	41	
425	No. 40	27	68.1% Sand
250	No. 60	18	
150	No. 100	12	
75	No. 200	7.0	7% Fines





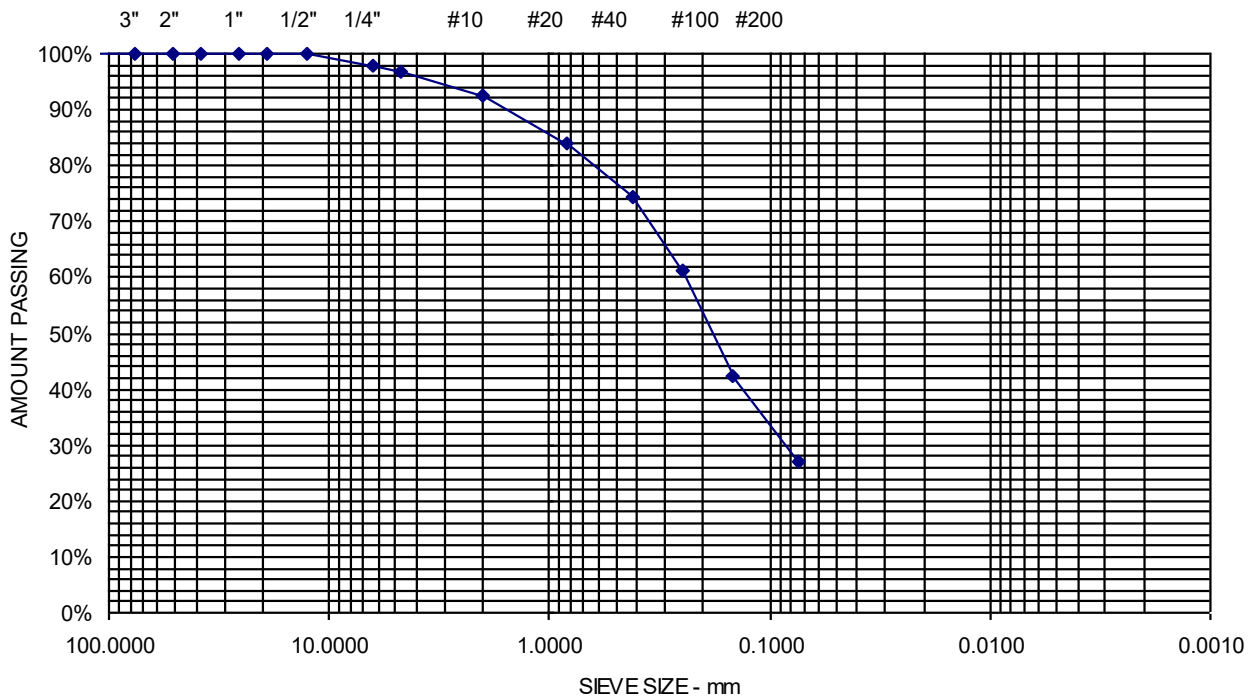
Report of Gradation

AASHTO T11 & T27

Project Name BROOKLIN-SEDGWICK ME - MAINEDOT WIN 21677 BENJAMIN RIVER BRIDGE #3216 REPLACMEENT - GEOTECHNICAL
 Client KLEINFELDER, INC.
 Exploration **BB-BSBR-101**
 Material Source **4D**

Project Number 16-1407
 Lab ID 20327B
 Date Received 3/10/2017
 Date Completed 3/13/2017
 Tested By JASON ORCUTT

<u>STANDARD DESIGNATION (mm/μm)</u>	<u>SIEVE SIZE</u>	<u>AMOUNT PASSING (%)</u>	
150	6"	100	
125	5"	100	
100	4"	100	
75	3"	100	
50	2"	100	
38.1	1-1/2"	100	
25.0	1"	100	
19.0	3/4"	100	
12.5	1/2"	100	
6.3	1/4"	98	
4.75	No. 4	97	3.1% Gravel
2.00	No. 10	92	
850	No. 20	84	
425	No. 40	74	69.9% Sand
250	No. 60	61	
150	No. 100	42	
75	No. 200	27.1	27.1% Fines





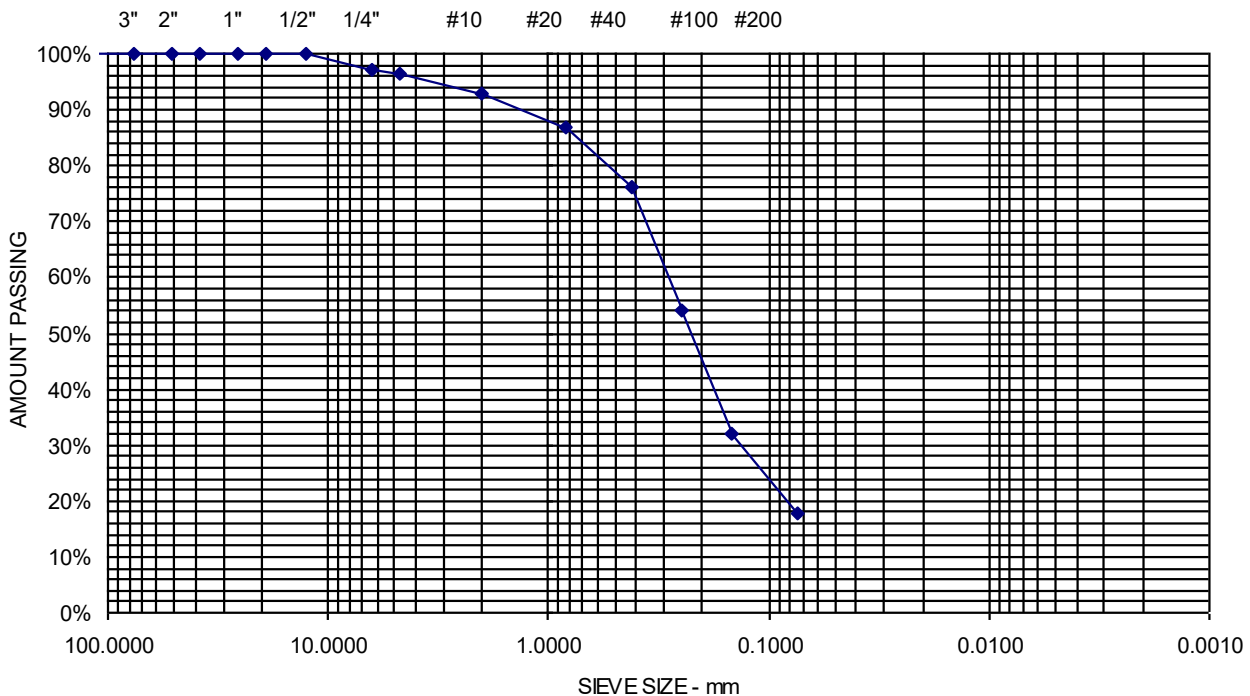
Report of Gradation

AASHTO T11 & T27

Project Name BROOKLIN-SEDGWICK ME - MAINEDOT WIN 21677 BENJAMIN RIVER BRIDGE #3216 REPLACMEENT - GEOTECHNICAL
 Client KLEINFELDER, INC.
 Exploration **BB-BSBR-101**
 Material Source **5D**

Project Number 16-1407
 Lab ID 20328B
 Date Received 3/10/2017
 Date Completed 3/21/2017
 Tested By DEAN MALLET

<u>STANDARD DESIGNATION (mm/μm)</u>	<u>SIEVE SIZE</u>	<u>AMOUNT PASSING (%)</u>	
150	6"	100	
125	5"	100	
100	4"	100	
75	3"	100	
50	2"	100	
38.1	1-1/2"	100	
25.0	1"	100	
19.0	3/4"	100	
12.5	1/2"	100	
6.3	1/4"	97	
4.75	No. 4	96	3.6% Gravel
2.00	No. 10	93	
850	No. 20	87	
425	No. 40	76	78.5% Sand
250	No. 60	54	
150	No. 100	32	
75	No. 200	17.9	17.9% Fines



Comments:



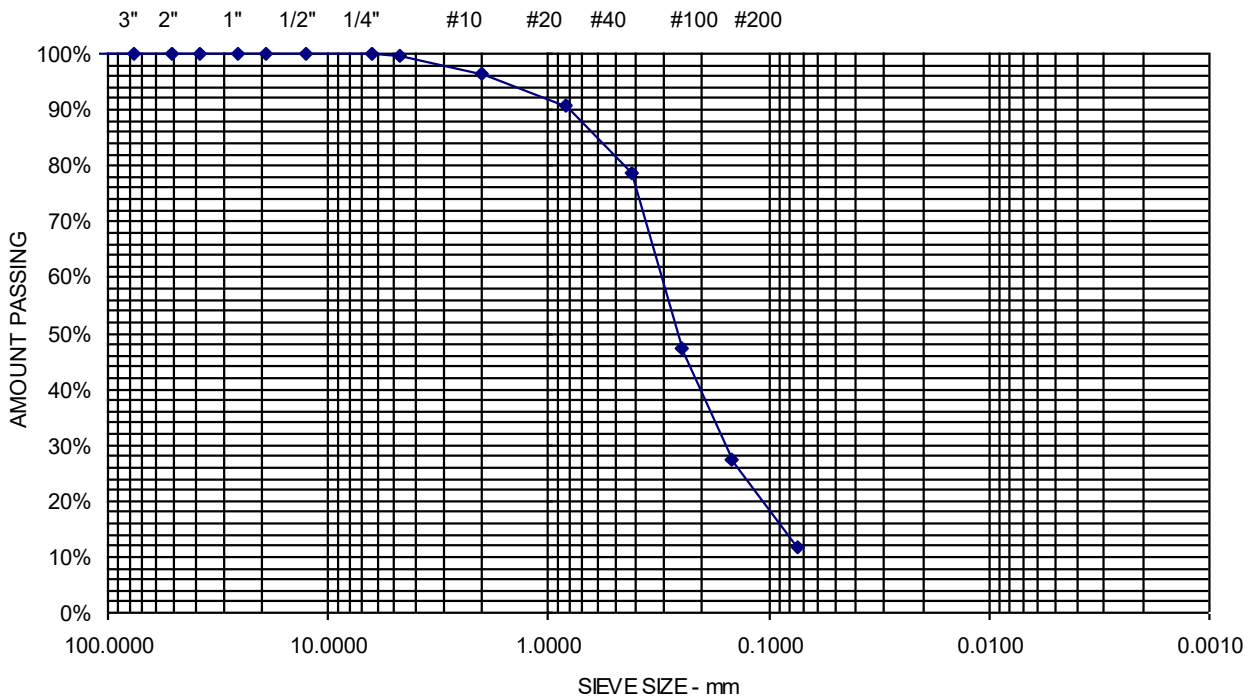
Report of Gradation

AASHTO T11 & T27

Project Name BROOKLIN-SEDGWICK ME - MAINEDOT WIN 21677 BENJAMIN RIVER BRIDGE #3216 REPLACMEENT - GEOTECHNICAL
 Client KLEINFELDER, INC.
 Exploration **BB-BSBR-101**
 Material Source **8D**

Project Number 16-1407
 Lab ID 20329B
 Date Received 3/10/2017
 Date Completed 3/21/2017
 Tested By DEAN MALLET

<u>STANDARD DESIGNATION (mm/μm)</u>	<u>SIEVE SIZE</u>	<u>AMOUNT PASSING (%)</u>	
150	6"	100	
125	5"	100	
100	4"	100	
75	3"	100	
50	2"	100	
38.1	1-1/2"	100	
25.0	1"	100	
19.0	3/4"	100	
12.5	1/2"	100	
6.3	1/4"	100	
4.75	No. 4	100	0.3% Gravel
2.00	No. 10	97	
850	No. 20	91	
425	No. 40	79	88.1% Sand
250	No. 60	47	
150	No. 100	27	
75	No. 200	11.6	11.6% Fines



Comments:



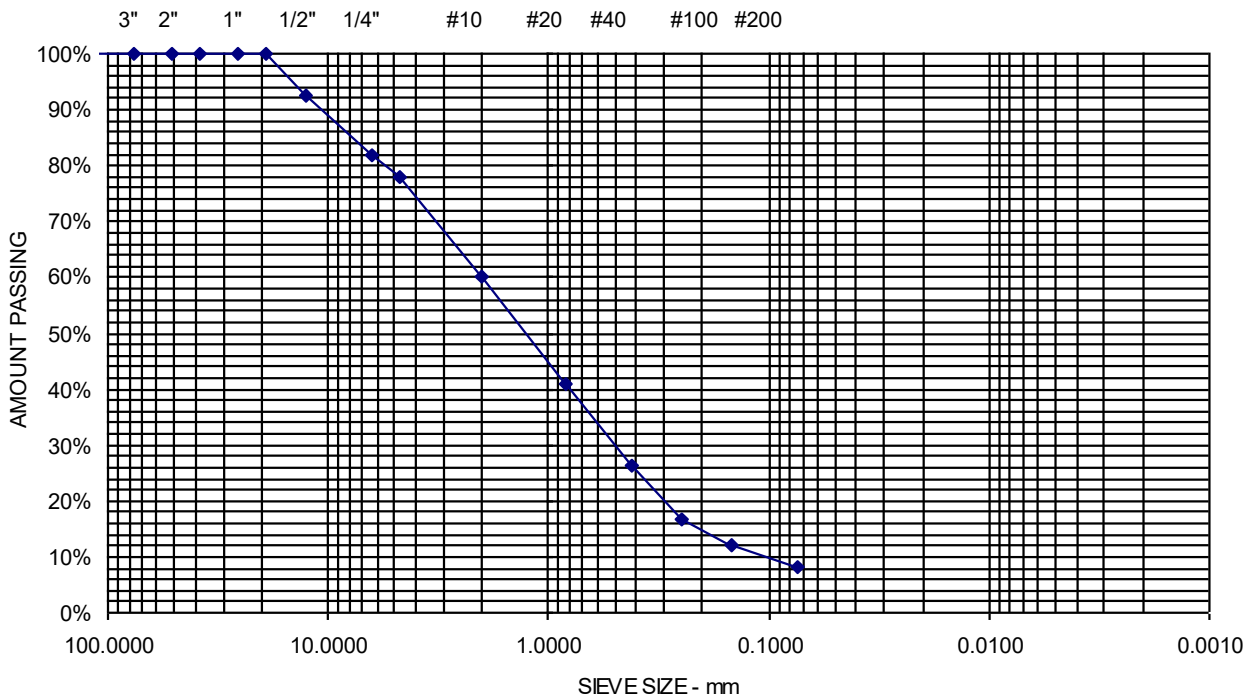
Report of Gradation

AASHTO T11 & T27

Project Name **BROOKLIN-SEDGWICK ME - MAINEDOT WIN 21677 BENJAMIN RIVER BRIDGE #3216 REPLACMEENT - GEOTECHNICAL**
 Client **KLEINFELDER, INC.**
 Exploration **BB-BSBR-102**
 Material Source **1D**

Project Number **16-1407**
 Lab ID **20330B**
 Date Received **3/10/2017**
 Date Completed **3/13/2017**
 Tested By **CHRISTOPHER RAYMOND**

<u>STANDARD DESIGNATION (mm/μm)</u>	<u>SIEVE SIZE</u>	<u>AMOUNT PASSING (%)</u>	
150	6"	100	
125	5"	100	
100	4"	100	
75	3"	100	
50	2"	100	
38.1	1-1/2"	100	
25.0	1"	100	
19.0	3/4"	100	
12.5	1/2"	93	
6.3	1/4"	82	
4.75	No. 4	78	22.1% Gravel
2.00	No. 10	60	
850	No. 20	41	
425	No. 40	26	69.8% Sand
250	No. 60	17	
150	No. 100	12	
75	No. 200	8.2	8.2% Fines





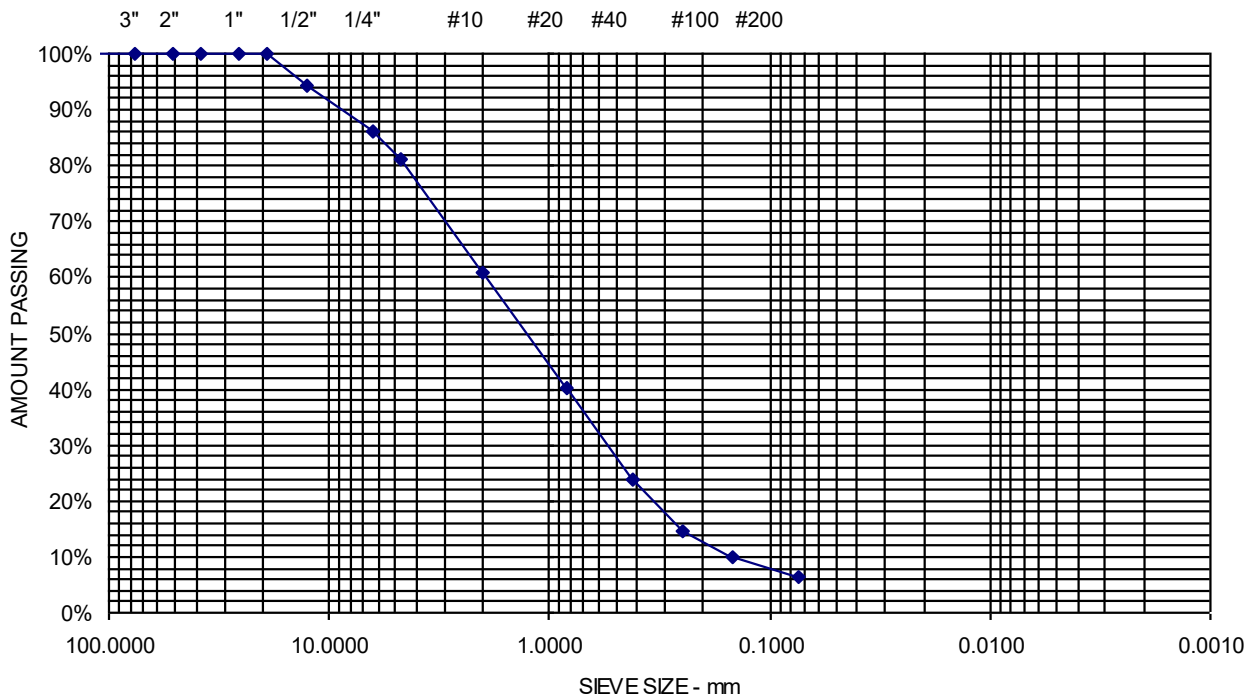
Report of Gradation

AASHTO T11 & T27

Project Name BROOKLIN-SEDGWICK ME - MAINEDOT WIN 21677 BENJAMIN RIVER BRIDGE #3216 REPLACMEENT - GEOTECHNICAL
 Client KLEINFELDER, INC.
 Exploration **BB-BSBR-102**
 Material Source **2D**

Project Number 16-1407
 Lab ID 20331B
 Date Received 3/10/2017
 Date Completed 3/13/2017
 Tested By JASON ORCUTT

<u>STANDARD DESIGNATION (mm/μm)</u>	<u>SIEVE SIZE</u>	<u>AMOUNT PASSING (%)</u>	
150	6"	100	
125	5"	100	
100	4"	100	
75	3"	100	
50	2"	100	
38.1	1-1/2"	100	
25.0	1"	100	
19.0	3/4"	100	
12.5	1/2"	94	
6.3	1/4"	86	
4.75	No. 4	81	18.9% Gravel
2.00	No. 10	61	
850	No. 20	40	
425	No. 40	24	74.5% Sand
250	No. 60	14	
150	No. 100	10	
75	No. 200	6.5	6.5% Fines





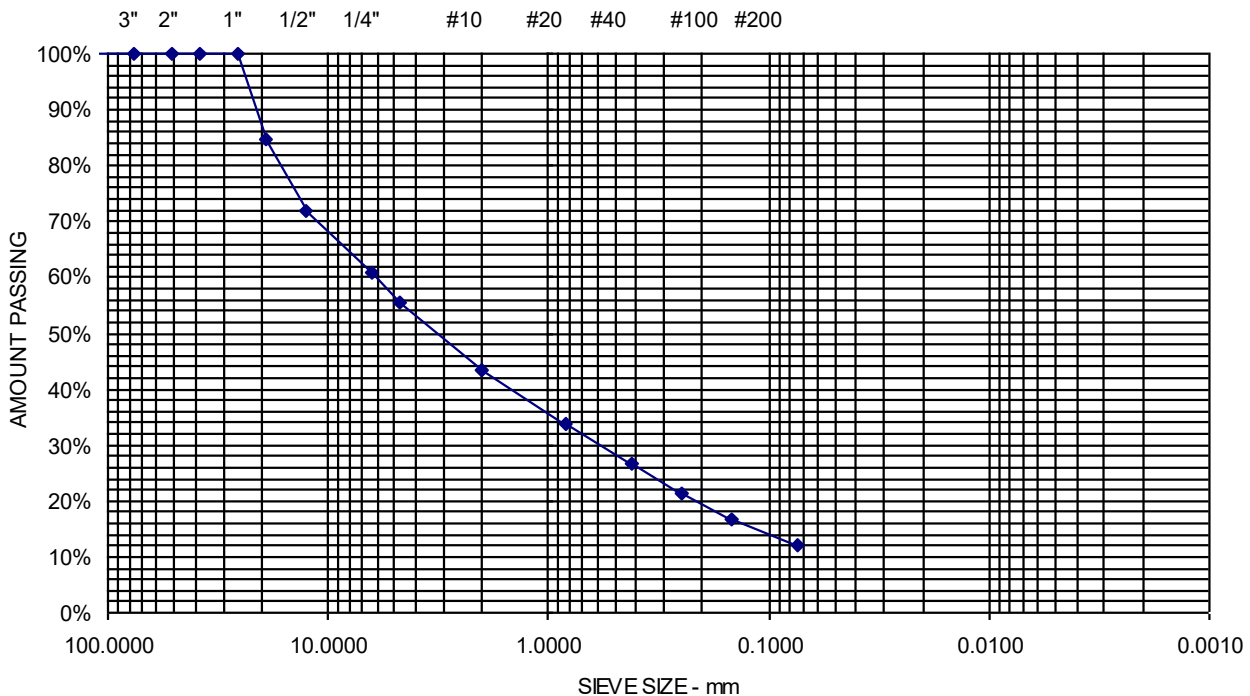
Report of Gradation

AASHTO T11 & T27

Project Name BROOKLIN-SEDGWICK ME - MAINEDOT WIN 21677 BENJAMIN RIVER BRIDGE #3216 REPLACMEENT - GEOTECHNICAL
 Client KLEINFELDER, INC.
 Exploration **BB-BSBR-102**
 Material Source **7D**

Project Number 16-1407
 Lab ID 20332B
 Date Received 3/10/2017
 Date Completed 3/13/2017
 Tested By JASON ORCUTT

<u>STANDARD DESIGNATION (mm/μm)</u>	<u>SIEVE SIZE</u>	<u>AMOUNT PASSING (%)</u>	
150	6"	100	
125	5"	100	
100	4"	100	
75	3"	100	
50	2"	100	
38.1	1-1/2"	100	
25.0	1"	100	
19.0	3/4"	85	
12.5	1/2"	72	
6.3	1/4"	61	
4.75	No. 4	55	44.6% Gravel
2.00	No. 10	43	
850	No. 20	34	
425	No. 40	27	43.3% Sand
250	No. 60	21	
150	No. 100	17	
75	No. 200	12.2	12.2% Fines



Comments: WC=7.8%



APPENDIX E
Calculations



Evaluation of Passive Earth Pressure for IAB

Assumed Backfill Values

MaineDOT BDG Section 3.6.1 - Soil Type 4

$\gamma_1 := 125 \text{ pcf}$ Unit Weight

$\phi_1 := 32 \text{ deg}$ Friction Angle

$c_1 := 0 \text{ psf}$ Cohesion

Integral Abutment Wall Parameters

$\beta := 0 \text{ deg}$ Continous Backslope Angle(s) (from horizontal)

$\theta := 90 \text{ deg}$ Angle of back face of wall (from horizontal)

$\delta := 19.5 \text{ deg}$ For IAB, Interface Friction between Fill and Wall
LRFD Table 3.11.5.3-1, $\delta = 17$ to 22 deg

Coulomb Theory Passive Earth Pressure Coefficient

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

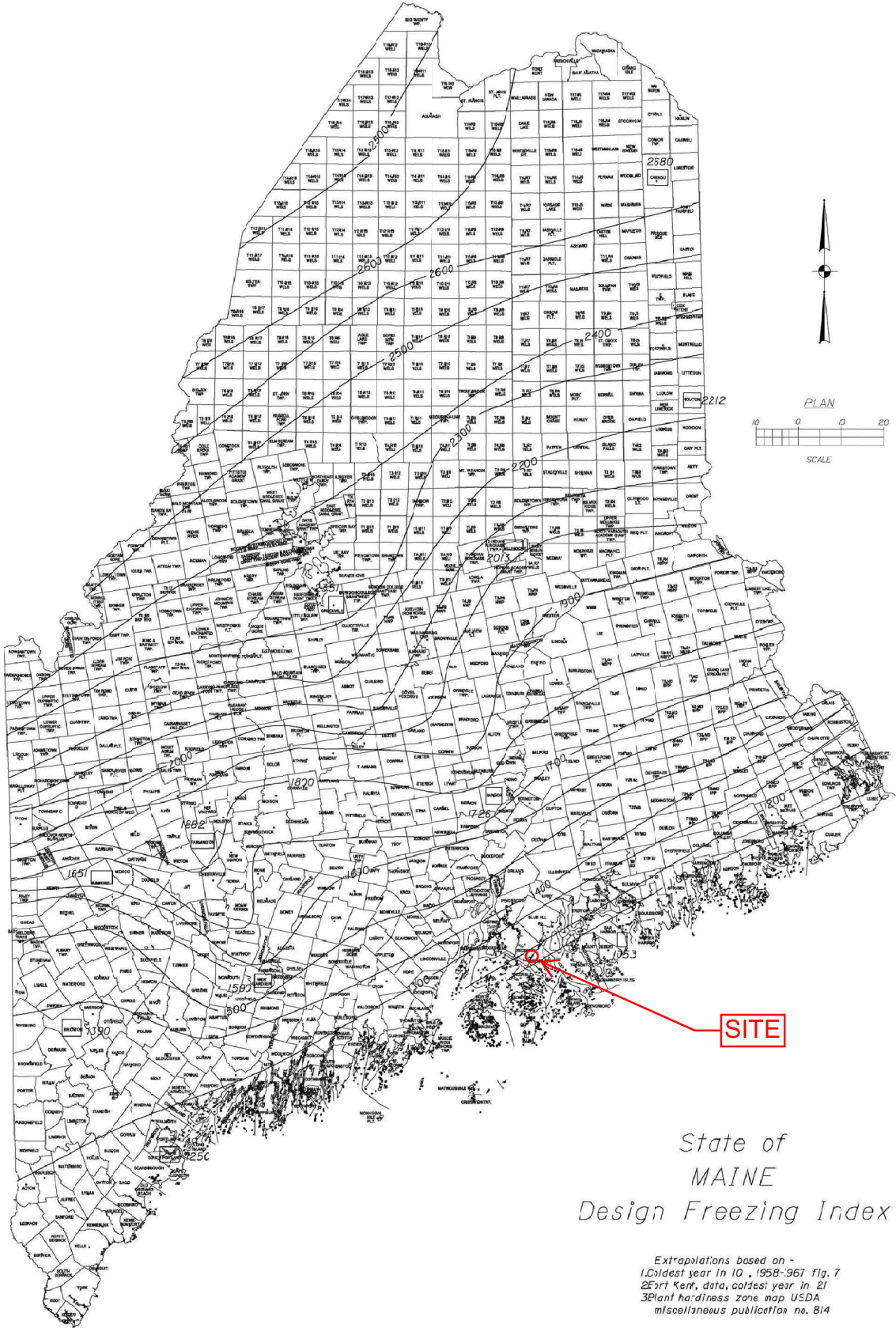
$$k_{p_c} := \frac{\sin(\theta - \phi_1)^2}{\Gamma_{p_c} \cdot (\sin(\theta))^2 \cdot (\sin(\theta + \delta))} \quad k_{p_c} = 6.73 \quad \text{for} \quad \beta = 0 \text{ deg}$$

Rankine Theory Passive Earth Pressure Coefficient

$$k_{p_r} := \frac{\cos(\beta) + \sqrt{\cos(\beta)^2 - \cos(\phi_1)^2}}{\cos(\beta) - \sqrt{\cos(\beta)^2 - \cos(\phi_1)^2}} \quad k_{p_r} = 3.25 \quad \text{for} \quad \beta = 0 \text{ deg}$$

Evaluated by: MAS
Date: April 24, 2017
Checked by: EJB
Date: April 25, 2017

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.



Estimated Frost Penetration Depth

Based on MaineDOT Bridge Design Guide Section 5.2.1

Site Location: Brooklin-Sedgwick, Maine

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

Step 1. From Figure 5-1: Design Freezing Index = ±1150 freezing degree-days

Step 2. Soils moist; assume $w = 10\%$

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

$$DFI := 1150$$

$$DFI_1 := 1100 \quad d_1 := 69.8 \text{ in}$$

$$DFI_2 := 1200 \quad d_2 := 73.1 \text{ in}$$

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

$$d_{frost} = 71.5 \text{ in}$$

$$d_{frost} = 6 \text{ ft}$$

Evaluated by: MAS
Date: April 24, 2017
Checked by: EJB
Date: April 25, 2017

USGS Design Maps Summary Report

User-Specified Input

Report Title BSR3216

Wed March 22, 2017 21:13:29 UTC

Building Code Reference Document 2009 AASHTO Guide Specifications for LRFD Seismic Bridge Design
(which utilizes USGS hazard data available in 2002)

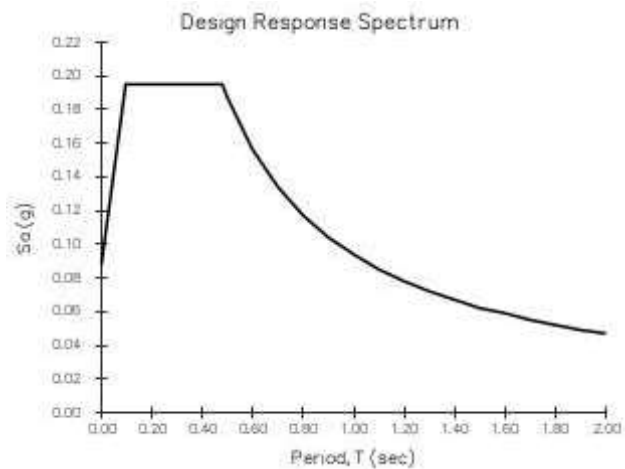
Site Coordinates 44.30392°N, 68.61179°W

Site Soil Classification Site Class D - "Stiff Soil"



USGS-Provided Output

PGA = 0.055 g	A_s = 0.088 g
S_s = 0.122 g	S_{DS} = 0.195 g
S₁ = 0.039 g	S_{D1} = 0.094 g



Although this information is a product of the U.S. Geological Survey, we provide no warranty, expressed or implied, as to the accuracy of the data contained therein. This tool is not a substitute for technical subject-matter knowledge.

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

Data From Boring BB-BSBR-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									N _i	d _i	
1	Fill	0	14	34	11	100						48.3	14	0.29
2	Marine Sand	14	41	3	53	34	29	40	34	100		41.9	27	0.65
4	Bedrock	41	100	100								100.0	59	0.59

- Notes:**
1. N60 value for frozen soil ignored
 2. N60 values > 100 taken as N=100
 3. N60 value for bedrock taken as N=100

$\Sigma = 100 \quad 1.52$

$N_bar = d_i/d_i/N_i = \boxed{65.6}$
 Site Class **C**

Data From Boring BB-BSBR-102 & BB-BSBR-102A

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									N _i	d _i	
1	Fill	0	17	14	20	17	12					15.8	17	1.08
3	Marine Sand	17	39	59	100	51	59	79	50			66.3	22	0.33
4	Bedrock	39	100	100								100.0	61	0.61

- Notes:**
1. N60 value for frozen soil ignored
 2. N60 values > 100 taken as N=100
 3. N60 value for bedrock taken as N=100

$\Sigma = 100 \quad 2.02$

$N_bar = d_i/d_i/N_i = \boxed{49.5}$
 Site Class **D**