REVISED GEOTECHNICAL DEISGN REPORT

Twin Bridge #5315 Over West Branch Souadabscook Stream

Hampden, Maine

Submitted to:

WSP USA, Inc.
428 Dow Highway
Elliot, ME 03903

Submitted by:

Golder Associates Inc.
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1772224/20368356

November 5, 2020

GOLDER

November 5, 2020

Project No.: 1772224/20368356

Adam Stockin, PE
Supervising Structural Engineer
WSP USA
428 Dow Highway
Elliot, ME 03903

**RE: REVISED GEOTECHNICAL DESIGN REPORT
TWIN BRIDGE #5315 OVER WEST BRANCH SOUADABSCOOK STREAM
HAMPDEN, MAINE
MAINE DOT WIN 18959.00**

Dear Mr. Stockin:

Golder Associates Inc. (Golder) is pleased to submit this Revised Geotechnical Design Report to WSP for the proposed replacement of Twin Bridge #5315 over West Branch Souadabscook Stream in Hampden, Maine. This report presents the subsurface information and field testing data obtained during the field investigations, the results of geotechnical laboratory tests, and geotechnical recommendations for the proposed bridge replacement. Golder submitted a Preliminary Geotechnical Design Report in March 2019 following the preliminary field investigations. This revised report has been updated to incorporate the findings of Hager-Richter Geoscience Inc.'s (Hager-Richter or H-R) geophysical investigation performed on September 25, 2020, conducted to supplement the preliminary geotechnical investigation results.

Our work was conducted in accordance with the scope, schedule and budget described in our final design proposal dated April 23, 2019. The terms and conditions governing the work are stated in the Professional Services On-Call/Task Order Subcontract dated February 23, 2017. Please contact us if you have any questions concerning our report or require additional information.

Sincerely,

GOLDER ASSOCIATES INC.

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Table of Contents

1.0 INTRODUCTION

This report summarizes the results of Golder Associates Inc.'s (Golder's) geotechnical investigation for replacement of Twin Bridge culvert carrying Maine Route 69 over West Branch of the Souadabscook Stream in Hampden, Maine (see Figure 1). The purpose of this report is to present soils and bedrock information at the bridge obtained from subsurface investigations and laboratory tests; discuss geotechnical considerations for the bridge foundations; and present recommended geotechnical parameters for design and construction. Our work was completed in accordance with our Professional Services On-Call/Task Order Subcontract dated February 23, 2017.

1.1 Background

The culvert replacement site is about 1 mile southeast of the Newburgh town line where State Route 69 crosses over the West Branch of the Souadabscook Stream. The existing culvert was built in 1951 and consists of a 25-foot (ft) span steel structural plate pipe arch supported on an approximately five-foot high and five-foot wide continuous concrete footing at each abutment. The bearing layer for the existing abutments is unknown but is likely the native soils. The roadway is 26 ft wide and is flanked by steel guardrail for the approaches. The existing culvert replaced a bridge constructed of timber stringers and planking with flat ledge stone abutments. The approximate locations of the flat ledge stone abutments for the original bridge is shown on Figure 2. Details of the demolition of the previous structure are not known.

The soundings made in 1950 indicate approximately 6 to 7 ft of coarse sand and clay overburden exists below the streambed elevation. Soundings conducted for the existing structure encountered refusal conditions around El. 35, which indicates about 15 ft of overburden exists below the existing road grade. Review of the Maine Geological Survey (MGS) bedrock and surficial geology maps for the area indicate the surficial geology consists of Presumpscot Formation clays and silts, and bedrock exposures are mapped in the stream bed and on the stream banks.

WSP plans to replace the existing culvert with a 53' – 3 ¾" span bridge consisting of CT girders on cantilever abutments supported on spread footings bearing on seal concrete over bedrock. In addition, we understand that WSP does not anticipate raising the grade of the road or widening the approaches. During construction, traffic will be diverted to a temporary bridge located east of the Twin Bridge replacement. This report does not include subsurface investigations or recommendations in support of the temporary bridge.

2.0 GEOLOGIC SETTING

2.1 Regional Surficial Geology

The proposed culvert replacement site is located in southern-central Maine within the Seaboard Lowlands Physiographic Province. A layer of till generally overlies bedrock and consists of a heterogeneous, non-sorted to poorly sorted mixture of clay, silt, sand, pebbles, cobbles and boulders. The till generally overlies bedrock, and is in turn overlain by Pleistocene glaciomarine silts, clays and fine sands of the Presumpscot Formation, which is overlain by glacial-marine fans and deltas, eskers, freshwater wetland deposits, and stream alluvium deposits. The Presumpscot Formation is commonly mapped in the area within the lower elevation stream valleys such as Souadabscook Stream.

2.2 Regional Bedrock Geology

Regional bedrock geologic mapping indicates the bedrock consists of the Silurian-Ordovician aged Vassalboro Group (undifferentiated).[1] These rocks consist of greenish-gray, fine-grained, calcareous metasiltstone or quartz-rich meta-wacke interbedded with subordinate amounts of medium to dark gray phyllite, and light-gray, green-weathering, fine grained, quartz-chlorite-plagioclase-muscovite-calcite schist containing accessory ilmenite, and intercalated beds of sulfide-bearing schist.[1,2] At the site location stream bed the beds generally strike northeast-southwest, and are vertical to dipping steeply to the northwest. Cleavage is also mapped striking parallel to bedding and dipping steeply to the northwest. A joint set is also mapped nearly perpendicular to bedding strike, dipping steeply to the northeast.

3.0 SUBSURFACE EXPLORATION PROGRAM

3.1 Geotechnical Investigation

On May 15 and 16, 2017 New England Boring Contractors (NEB) of Hermon, Maine drilled three borings (BB-HAMP-101, BB-HAMP-102 and BB-HAMP-103) at the locations shown on Figure 2 using a Mobile B53 rig. Test borings were drilled along the road located on either side of the existing culvert to depths of approximately 19 to 29 ft below ground surface (bgs). A Golder engineer was onsite during drilling to monitor the work, select sample/field test intervals, log the conditions encountered and collect soil and rock core samples.

Standard Penetration Test (SPT) sampling was conducted in accordance with ASTM D1586 in two of the borings at intervals varying from continuous to 5 ft. Standard 1-3/8-inch ID split spoons were driven 24 inches by a 140-pound hammer dropped 30 inches using a rope and cathead. Golder recorded the number of hammer blows required to advance the sampler at 6-inch increments, assessed lithology, and logged sample recovery lengths. Three- inch inner diameter (ID) split spoons were occasionally used to attempt to increase soil recovery as directed by Golder's field engineer. Drill behavior during casing and drill rod advancement and cuttings observed during drilling were also recorded. After reaching bedrock refusal in BB-HAMP-101 and -102, NEB obtained 10 feet of NQ sized bedrock core from the borings. The third boring, BB-HAMP-103, was advanced with a solid stem auger to refusal on suspected bedrock. No samples were collected. Lithologic descriptions are based on drill cuttings and drilling behavior.

We understand that MaineDOT has not completed a survey of the as-drilled locations and ground surface elevations for the borings. Table 1 includes a summary of the boring information including station and offset based on hand taped swing-tie measurements in the field as well as as-drilled elevations (based on base map information provided by WSP) and boring depth.

Details of the sampling methods used, field data obtained, and soil, bedrock and groundwater conditions encountered are presented on the boring logs included in Appendix A. A hammer efficiency factor of 0.60 was used for the rope and cathead method in determining the N_{60} corrections[3] shown on the logs. Soils were field classified in general accordance with ASTM D 2488 and the MaineDOT Key to Soil and Rock Descriptions and

[1] West, D.P. Jr., 2016. Bedrock Geology of the Snow Mountain Quadrangle, Maine. Maine Geological Survey Open-File Map 16-26, scale 1:24,000.

[2] Wones, D.R., 1991. Bedrock Geologic Map of the Bucksport Quadrangle, Waldo, Hancock and Penobscot Counties, Maine. U.S. Geological Survey, Geological Quadrangle Map GQ-1692, scale 1:62,500.

[3] Standard Penetration Test N-values (blows per foot) corrected for hammer energy to determine an equivalent N value for a hammer with 60% efficiency are designated as N_{60} values. N values discussed in the report generally refer to corrected N_{60} values.

Terms. A description of the boring log symbols and terms used for the soil and rock descriptions precedes the boring logs presented in Appendix A. Photographs of the rock core are included in Appendix B. Table 2 summarizes rock core descriptions, rock mass rating, and quality assessments.

3.2 Geophysical Survey

On September 25, 2020 Hager-Richter Geoscience Inc. (H-R) of Salem, New Hampshire performed a ground penetrating radar (GPR) survey of the project area to identify the existence and/or extents of buried obstructions and historic abutment structures. The survey area covered an area extending from approximately 40 feet south of the current culvert crossing to roughly 180 feet north of the crossing and roughly 30ft wide. A Golder geologist helped coordinate and observed the geophysical surveys, and H-R's report is provided as Appendix E[4].

4.0 LABORATORY TESTING PROGRAM

Geotechnical laboratory tests were performed on representative soil and rock samples collected during the subsurface investigation to assist in soil classification and anticipated rock strength. Samples were collected at borings BB-HAMP-101 and BB-HAMP-102 locations during the 2017 Golder Investigation. No samples were taken from BB-HAMP-103.

Testing on the selected 2017 Golder Investigation samples was conducted by GeoTesting Express in Acton, Massachusetts. Laboratory work was performed in accordance with applicable AASHTO and American Society for Testing Materials (ASTM) testing procedures. The combined testing performed for the investigation is summarized below.

Soil/Rock Laboratory Test	Testing Procedure	Number of Tests Completed
Grain Size Analysis, sieve only	AASHTO T88, ASTM D422	4
Grain Size Analysis including Hydrometer	AASHTO T88, ASTM D422	1
Natural Moisture Content	AASHTO T265, ASTM D2216	3
Uniaxial Compressive Strength of Rock	ASTM D7012C	1

Selected soil and rock testing results are included on the boring logs in Appendix A and summarized on Table 2. Complete laboratory testing results are provided in Appendix C.

5.0 GENERALIZED SUBSURFACE CONDITIONS

Soils encountered at the borings were found to generally include asphalt and fill materials placed during construction of the roadway and bridge abutments, and naturally occurring alluvial and glacial till sediments. The underlying bedrock surface is interpreted to range from about 17.5 to 19.3 ft below the road grade as shown on the interpreted subsurface profile on Figure 3. During the September 2020 geophysical field investigation, H-R noted possible shallow bedrock (~5 ft below ground surface[5]) located approximately 70 to 130 ft north of the

[4] Hager-Richter Geoscience, Inc., (2020). "Geophysical Survey Twin Bridge # 5315 West Branch Souadabscook Stream ME Route 69 Hampden, Maine."

[5] Michael Howley (Hager-Richter Geophysics, Inc.), personal communication, October 15, 2020.

existing culvert along State Route 69. Detailed descriptions of the soil and bedrock conditions encountered at the borings are provided on the Boring Logs in Appendix A. The following sections summarize the major stratigraphic units.

Asphalt Pavement

Asphalt pavement 8 inches thick was encountered in each of the three borings.

Fill

Sand and gravel subgrade fill materials ranging in thickness from 6.3 to 7.3 ft were encountered directly beneath the asphalt pavement at the boring locations. The fill materials generally consist of brown, medium dense to very dense, dry, fine to coarse sand with some gravel; and silty fine to medium sand with some gravel. Corrected SPT N-values[6] in the fill materials ranged from 14 to 52, with an average of 32, indicating a dense consistency; however, the high 50+ N-value is likely influenced by gravel within the fill and is therefore likely artificially high. A grain size analyses of this layer indicated an AASHTO classifications of A-1-a (0) and SM under the Unified Soil Classification System (Unified). The measured water content of the tested sample was 6.4 percent.

Alluvium

Coarse sand and gravel material was encountered below the fill layer. The alluvium ranged in thickness from 7.0 to 8.0 ft. This layer consists of brown, wet, loose, fine to coarse sand with some gravel and some silt; gravelly medium to coarse sand with some to trace silt; and medium to coarse sandy gravel with little to trace silt. N-values ranged from 6 to 13, with an average of 9, indicating a generally loose consistency. Laboratory testing in this layer indicate an AASHTO classification of A-1-a(1) and Unified classifications of SP and GW.

Glacial Till

Glacial till was encountered beneath alluvial sediments ranging in thickness from 2.5 to 3.5 ft. This layer consists of brown-gray, dense, silty sand with some gravel and some clay, and fine to coarse sandy gravel with little silt. N-values ranged from 38 to 50, with an average of 44, indicating a dense consistency. Grain size analyses of this layer indicated AASHTO classifications of A-4 and A-1-b (0); and GM and SM Unified classifications. The measured water content of the tested samples ranged from about 7.7 to 8.4 percent.

Bedrock

Bedrock was encountered in two borings as determined from rock core (BB-HAMP-101 and -102) and suspected from drill rig/solid stem auger refusal at BB-HAMP-103. Additionally, the H-R geophysical survey indicated the presence of shallow bedrock roughly 70 to 130 ft north of the existing culvert crossing. Bedrock surface depths are interpreted to be between about 17.5 and19.3 ft bgs near the existing culvert crossing and are potentially as shallow as 5 ft[5] north of the culvert based on the geophysical findings. Rock quality designation (RQD) of the core samples indicated bedrock that is very poor (0%) near the north abutment and poor to fair (33-68%) at the south abutment.

Core samples collected near the north abutment in BB-HAMP-101 showed bedrock consisting of grayish-black, fine grained phyllite. This rock is medium strong (R3), fresh to highly weathered (W1-W4), with extremely close to closely spaced discontinuities. Rock Mass Rating (RMR) of the rock core runs ranged from 37 to 44 with an average of (40.5). Laboratory testing was not completed on rock samples from this boring. Core samples

[6] All subsequent references to SPT N-values in this report pertain to the energy corrected N_{60} values shown on the boring logs in Appendix A.

collected near the south abutment in BB-HAMP-102 showed bedrock consisting of medium dark gray, fine grained phyllite. This rock is medium strong (R3), fresh (W1), with extremely close to moderately spaced discontinuities. Rock Mass Rating (RMR) of the rock core runs ranged from 52 to 54. Laboratory testing indicated an unconfined compressive strength (UCS) of 11,473 psi. A detailed summary of the rock core is presented in Table 2.

Groundwater

Groundwater level measurements were attempted upon completion of the boreholes and removal of the casing but the boreholes were dry. Groundwater levels shown on the subsurface profile (Figure 3) were interpreted based on observed changes in saturation of the SPT samples. Groundwater levels are interpreted to lie 7 to 8 ft below the roadway which is in-line with the edge of waterway drainage elevation indicated on the Base Map provided by WSP for the West Branch Souadabscook Stream.

Subsurface Obstructions

As shown in Appendix E, the H-R report indicated the presence of subsurface obstructions in several areas north of the existing culvert crossing. The obstructions, as shown on Figures 2 and 3, consist of buried former abutment structures, a possible buried retaining wall as well as some unknown buried objects. Based on subsequent telecommunication with H-R, we understand that the buried abutment structures are located approximately 2 ft below the road surface. The geophysical survey was limited to the road grade and did not extend down the embankment sideslopes. The potential for subsurface obstructions beyond the extents for the geophysical survey is unknown.

6.0 EVALUATIONS AND RECOMMENDATIONS

We understand that WSP recommends replacing the existing culvert with a on 53' – 3 ¾" span bridge consisting of CT girders on cantilever abutment and wingwalls supported on reinforced concrete spread footings bearing on bedrock or seal concrete over bedrock. Based on the subsurface conditions encountered during the field exploration program and discussions with WSP, we conclude the proposed bridge abutment and wingwalls can be satisfactorily supported on shallow spread footings bearing on bedrock.

6.1 Frost Protection

We anticipate that the abutment and retaining walls will be supported on concrete spread footings founded directly on bedrock. As such, heave due to frost is not a design concern and no requirements for minimum depth of embedment are necessary.

However, Golder evaluated an average frost depth for the range of materials present below streambed elevation. According to Figure 5-1 in the MaineDOT Bridge Design Guide, the site has an air design-freezing index of 1700 F-degree days. Assuming a water content of 10% for soils above the water table and a water content of 30% for soils below the water table, Golder estimates an average frost depth of approximately 5.6 feet for coarse to fine grained soil at the site. Refer to the frost depth calculations in Appendix D.

6.2 Seismic Site Class

The borings drilled during this investigation were terminated between 19.3 and 28.5 ft bgs and all encountered bedrock between 17.5 to 19.3 ft bgs. The seismic site class was calculated using the average N values for each subsurface layer and the depths of each layer. The N value for the bedrock is assumed to be 100 blows/ft. Based on the site class definitions presented in AASHTO Table 3.10.3.1.1, the existing subsurface profile (overburden soils and bedrock) results in a Site Class C. Refer to the seismic site class calculations in Appendix D.

6.3 Spread Footings on Bedrock

As discussed above, the bedrock in the area of the proposed bridge replacement is interpreted to be fairly level at a depth of approximately 18 feet below ground surface, or approximately five to six feet below streambed elevation.

Bearing Resistance

Based on the rock type, structure, unconfined compressive strength, and interpreted Rock Mass Rating (RMR), we recommend a strength limit state factored bearing resistance of 12 kips per square foot (ksf) be used for spread footings bearing on a level rock surface. A presumptive bearing resistance of 20 ksf may be assumed for the bedrock at the factored service limit state, based on AASHTO LRFD Bridge Design Specifications Table C10.6.2.6.1-1. In no instance shall the factored bearing stress exceed the nominal resistance of the footing concrete (0.3f'c). No footing shall be less than 2 ft wide regardless of the applied bearing pressure or bearing material.

Sliding Resistance

Assuming the cast-in-place concrete abutment and wingwall sub footings bear on bedrock, a sliding coefficient (tan δ) of 0.6 is recommended per AASHTO LRFD Table 3.11.5.3-1. A resistance factor of 0.8 is recommended for sliding per MaineDOT Bridge Design Guide Table 5-3.

Rock Erodibility

Due to the very poor to fair RQD values obtained in the rock core, Golder performed a preliminary evaluation of the bedrock erodibility at the footing bearing elevation assuming scour eroded the streambed soils down to the bedrock surface. At the time these analyses were performed (June 2017, following the field investigation) the results of WSP's hydraulics and hydrology analysis for the project were not available. Therefore, our analysis included simplifying assumptions as stated below.

- Overburden – 0 feet, conservatively assume river is flowing directly over bedrock. Ignore ~6 feet of overburden over bedrock at streambed elevation
- Stream Depth – 4 feet, based on 2015 MaineDOT Inspection Report
- Stream Bankfull Flow – 32.6 feet wide, StreamStats[7] Data
- 100 Year Peak Flood – 1,680 ft^3/s, StreamStats Data
- Stream Velocity – 12.9ft/s, calculated based on rectangular channel shape
- Steady State Flow – energy slope = culvert slope

Our preliminary analysis indicate that bedrock erodibility at the footing subgrade level is not a concern based in the stated assumptions. Refer to the erodibility calculations included in Appendix D. Subsequent to Golder's analyses detailed above, WSP performed a hydraulics and scour analysis, which also concluded that the proposed foundations founded on bedrock are stable and not susceptible to scour.

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[7] StreamStats data output for 44.71211 latitude, -68.95101 longitude. (2017). Version Alpha 4.1.3. United States Geological Survey. <https://streamstats.usgs.gov/>

Settlement

As stated above, we assume that all bridge abutments and wingwalls will be founded on cast-in-place spread footings founded directly on bedrock. As such, any settlement of the bridge abutments or wingwalls will be due to elastic compression of the bedrock mass and is estimated to be less than 0.5 inch.

Footing Subgrade Preparation

The nature, slope, and degree of fracturing in the bedrock bearing surface will not be evident until the foundation excavations are made. The bedrock surface should be cleared of all loose, fractured, and decomposed bedrock and soil. The final bedrock surface should be washed with high pressure water and air and approved for cleanliness and condition by the Resident prior to the placement of the footing concrete.

6.4 Backfill

Backfill adjacent to the abutment breastwall and wingwalls should be consist of Granular Borrow Underwater Backfill (Backfill, MaineDOT Section 703.19). Backfill should extend for a horizontal distance of at least ten (10) feet from the back face of the abutment wall. The backfill should be placed in 6- to 8-inch loose lifts and compacted to a minimum of 95 percent of the maximum dry density as determined by AASHTO T-180. Mechanical tampers or an approved alternative should be used to compact the backfill directly adjacent to the walls. The abutment and wingwall system should be equipped with a drainage system in conformance with MaineDOT Bridge Design Guide Section 5.4.1.9, which directs drainage through 4-inch diameter weep holes constructed in the wall face.

6.5 Scour and Riprap

Scour countermeasures and slope erosion control issues need to be addressed within the stream channel and on the end slopes adjacent to the upstream and downstream ends of the culvert. The design criteria for erosion protection measures will need to be based on the results of the hydraulics and hydrology evaluations should be included in final design.

6.6 Construction Considerations

Construction activities will require soil excavation and construction of cofferdams and earth support systems to support approach fills during construction and to control water flow during construction. Design of all temporary earth support systems should be in compliance with OSHA requirements. The contractor shall control groundwater and surface water infiltration so that construction can be completed in the dry. According to the 1950 historical drawings and as shown on Figure 2, the original abutments and piers were constructed of dry stacked ledge stone. Demolition details of the original structure are unknown; however, as shown in Figures 2 and 3, the H-R geophysical survey located portions of the abutment structures and other buried obstructions north of the existing culvert crossing. Additionally, the geophysical survey was not performed over the sides of the roadway embankment so it is not known whether the wingwalls of the historical abutments have been removed. If encountered, these wingwalls may present difficulties for cofferdam and excavation support system installations.

7.0 CLOSING

This Revised Geotechnical Design Report was prepared for the exclusive use of WSP for specific application to the proposed Twin Bridge #5315 Replacement over West Branch Souadabscook Stream in Hampden, Maine in accordance with generally accepted soil and foundation engineering practices practiced in this geographical area and under similar time and financial constraints. Golder makes no other warranty, either express or implied. In the event that any changes in the nature, design or location of the proposed project are planned, Golder should be notified to review the appropriateness of our conclusions and recommendations, and to modify the recommendations as appropriate to reflect the changes in design. In addition, Golder should review the final plans and specifications to evaluate compliance with these recommendations.

Our analyses, and recommendations are based, in part, on information obtained from the referenced subsurface explorations completed at the discrete locations described in the report. Readers of this report should make their own interpretation of the results of the subsurface investigations and laboratory testing. Variations in the nature and extent of subsurface conditions between explorations should be expected. Golder should be notified if conditions encountered during construction vary from those described in this report so that we may re-evaluate, and if necessary, revise the recommendations made in this report.

The professional services provided by Golder for this project include only the geotechnical aspects of the subsurface conditions at this site. The presence or implications of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this report and have not been investigated or addressed.

https://golderassociates.sharepoint.com/sites/123637/project files/6 deliverables/20.10.29 hampden final pdr.docx

TABLES

Table 1: Subsurface Exploration Locations
Preliminary Geotechnical Design Report
Twin Bridge #5325 Souadabscook Stream
Hampden, Maine
MaineDOT WIN 18959.00

Test Boring Designation[1,2]	As-Drilled Locations[3]		Existing Ground Surface Elevation[3] (feet)	Boring Depth[4] (feet)
	Station	Offset		
BB-HAMP-101	100 + 86.1	7.2 ft L	192.8	28.5
BB-HAMP-102	101+24.3	10.2 ft R	192.2	27.5
BB-HAMP-103	101+23.8	8.6 ft L	193.4	19.3

Notes:

1. Test boring locations are shown in Figure 1 - "Boring Location Plan".
2. Borings BB-HAMP-101, BB-HAMP-102, and BB-HAMP-103 were performed by New England Boring Contractors in May 2017.
3. As-drilled locations and elevations are approximate and based on the basemap and alignment provided by WSP.
4. Depth below ground surface.
5. Boring logs presented in Appendix A.

Prepared By: LLM
Checked By: CJS
Reviewed By: MCM

Table 2: Summary of Rock Core Quality
Preliminary Geotechnical Design Report
Twin Bridge #5325 Souadabscook Stream
Hampden, Maine
MaineDOT WIN: 18959.00

Test Boring Designation	Core Size [in]	Existing Ground Surface Elevation[1] [ft]	Run No.	Run Midpoint Depth Below Bedrock Surface [ft]	Run Depth Below Ground Surface [ft] Start	Run Depth Below Ground Surface [ft] End	Run Depth Below Ground Surface [ft] Midpoint	Run Length [ft]	TCR[2] Length [ft]	TCR[2] %	RQD[3] Length [ft]	RQD[3] %	RQD[3] Designation	Physical Rock Parameters: Weathering[4]	Physical Rock Parameters: Estimated Field Strength[4]	Physical Rock Parameters: Rock Mass Rating [RMR][5]	Lithologic, Rock Mass and Discontinuity Description
BB-HAMP-101	NQ [1.875"]	192.8	R1	2.2	18.5	22.9	20.7	4.4	4.4	100%	0.0	0%	Very Poor	Slightly Weathered (W2)	Medium Strong Rock (R3)	44	18.5 ft - 22.9 ft: Geyish black (N2), fine grained, fresh to slightly weathered (W1-W2), medium strong rock (R3), PHYLLITE, discontinuity spacing is extremely close to closely spaced (<20 mm to 80 mm), dipping 55 to 85 degrees relative to horizontal axis, discontinuity surfaces are planar to curved to stepped, texture is smooth to very rough, trace pyrite [Vassalboro Formation (Silurian – Ordovician)] .
			R2	7.2	22.9	28.5	25.7	5.6	4.8	85%	0.0	0%	Very Poor	Moderatly Weathered (W3)	Medium Strong Rock (R3)	37	22.9 ft - 28.5 ft: Greyish black (N2), fine grained, fresh to highly weathered (W1-W4) with highly weathered zone from 26.8 feet to 28.5 feet below ground surface, medium weak rock (R3), weathered zone is very weak rock (R1), PHYLLITE, spacing is extremely close to closely spaced (<20 mm to 100 mm), dipping 85 to 90 degrees relative to horizontal axis, discontinuity surfaces are planar to stepped, texture is smooth to very rough, trace pyrite and iron staining in joints [Vassalboro Formation (Silurian – Ordovician)].
BB-HAMP-102	NQ [1.875"]	192.2	R1	2.5	17.5	22.5	20.0	5.0	4.2	83%	1.7	33%	Poor	Fresh (W1)	Medium Strong Rock (R3)	52	17.5 ft - 22.5 ft: Medium dark grey (N4), fine grained, fresh (W1), medium strong (R3), PHYLLITE, discontinuity spacing is extremely close to moderately spaced (<20 mm to 460 mm), dipping 55 to 85 degrees relative to horizontal axis, discontinuity surfaces are planar to curved to stepped, texture is smooth to rough [Vassalboro Formation (Silurian – Ordovician)].
			R2	7.5	22.5	27.5	25.0	5.0	4.7	93%	3.4	68%	Fair	Fresh (W1)	Medium Strong Rock (R3)	54	22.5 ft - 27.5 ft: Medium dark grey (N4), fine grained, fresh (W1), medium strong (R3), PHYLLITE, discontinuity spacing is very close to moderately spaced, dipping 55 to 85 degrees relative to horizontal axis, discontinuity surfaces are planar to curved, texture is smooth to rough, white seam observed at about 23.5 feet below ground surface [Vassalboro Formation (Silurian – Ordovician)].

Notes:
1. As-drilled surface elevations were not provided to Golder at the writing this report. Elevations are approximate based on the basemap provided by WSP.
2. TCR = total core recovery. Total core recovery is the length of core recovered divided by the length of the run.
3. RQD = rock quality designation. RQD is the total length of intact, full diameter core pieces recovered with a length greater than or equal to twice the core diameter (i.e., length of 4 inches) measured along the core axis. The percent RQD is the total length of RQD measured versus the run length. Note that vertical discontinuities are not included in determination of RQD.
4. Weathering and Estimated Field Strength based on Tables II.4 and II.3 (respectively) in Willey, 2004 (based on ISRM, 1981).
5. Rock Mass Rating System (RMR; Bieniawski, 1989) assigns numerical ratings to six parameters, including the strength of the intact rock, the RQD, the discontinuity spacing, discontinuity conditions, groundwater conditions, and orientation of discontinuities. These ratings are summed to give the RMR value. As proposed foundations or tie-back designs are not complete, the rating adjustment for joint orientation was assigned a value of 0.
6. ft = feet, in = inches

Prepared by: LLM
Checked by: CJS
Reviewed by: MCM

Table 3: Summary of Laboratory Testing Results
Preliminary Geotechnical Design Report
Twin Bridge #5325 Souadabscook Stream
Hampden, Maine
MaineDOT WIN 18959.00

Test Boring Designation[1]	Existing Ground Surface Elevation [2] (ft)	Sample Number	Sample Depth Below Ground Surface (ft)	Approximate Sample Elevation[2] (ft)	Water Content (%):	Sieve Minus No. 200 (%):	AASHTO Soil Classification[3,4]	USCS Soil Classification[3,4]	Rock Uniaxial Compressive Strength[4] (psi)
BB-HAMP-101	192.8	2D	3.0 - 5.0	189.8 - 187.8	6.4	12.2	A-1-a (0)	SM	- -
		5D	12.0 - 14.0	180.8 - 178.8	11.5	3.0	A-1-a (1)	SP	- -
		6D	15.0 - 17.0	177.8 - 175.8	8.4	18.0	A-1-b (0)	GM	- -
BB-HAMP-102	192.2	5D	11.0 - 13.0	181.2 - 179.2	9.4	2.5	A-1-a (1)	GW	- -
		6D	15.0 - 17.0	177.2 - 175.2	7.7	39.9	A-4 (0)	SM	- -
		R1	17.7 - 19.0	174.5 - 173.2	- -	- -	- -	- -	11,473

Notes:

1. Test boring locations are shown in Figure 1 - "Boring Location Plan"
2. As-drilled locations and elevations are approximate and based on alignment and basemap data provided by WSP.
3. AASHTO and USCS symbols assigned based on interpretation of laboratory test results.
4. Laboratory testing was performed by GeoTesting Express, Inc. Complete laboratory test results are provided in Appendix C.

Prepared By: LLM
Checked By: CJS
Reviewed By: MCM

FIGURES

REFERENCE

1.) BASEMAP TAKEN FROM U.S.G.S. 7.5 MINUTE QUADRANGLE OF BANGOR, ME DATED 1960.

0 1500 3000

1" = 3000' FEET

CLIENT

WSP
428 DOW HIGHWAY
ELLIOT, MAINE 03903

PROJECT

TWIN BRIDGE #5315 OVER WEST BRANCH SOUABABSCOOK STREAM
HAMPDEN, MAINE
MAINEDOT WIN 18959.00

CONSULTANT

GOLDER

YYYY-MM-DD	2017-06-15
DESIGNED	LLM
PREPARED	RWC
REVIEWED	MCM
APPROVED	MSP

TITLE

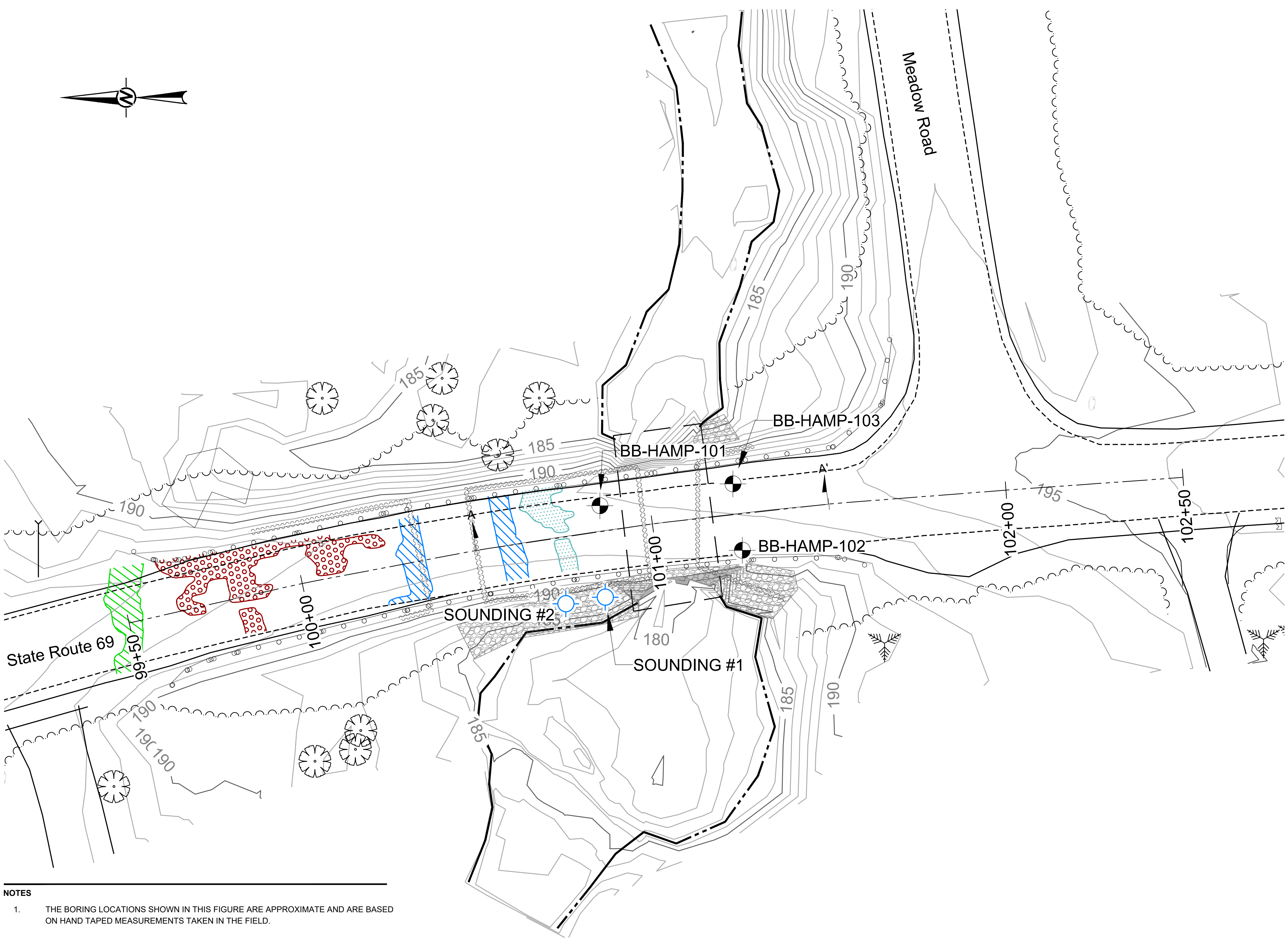
SITE LOCATION MAP

PROJECT NO.	SUBTITLE	REV.	FIGURE
1772224	D	0	1

Last Edited By: rclark Date: 2019-03-12 Time:4:33:33 PM | Printed By: rclark Date: 2019-03-13 Time:11:01:43 AM
Path: \\manchester\cadd\Drawings\2017\1772224 - PB Culvert - Hampden ME\PRODUCTION\ | File Name: 1772224D001.dwg

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI A

0 1 in



LEGEND

- 5 FT INDEX CONTOUR (FT MSL)
- 1 FT INDEX CONTOUR (FT MSL)
- EDGE OF DRAINAGE WATERWAY
- GUARDRAIL BEAM
- EDGE OF PAVEMENT
- EDGE OF ROADWAY SHOULDER
- TREELINE
- POSSIBLE FORMER RETAINING WALL (REF. 4)
- POSSIBLE BEDROCK (REF. 4)
- POSSIBLE FORMER ARCH BRIDGE ABUTMENTS (REF. 4)
- UNKNOWN BURIED OBJECTS (REF. 4)
- DECIDUOUS TREES
- BB-HAMP-101 CASED WASHED BORINGS
- EXISTING RIP RAP ARMOR
- A SUBSURFACE PROFILE (FIGURE 3) LOCATION AND DIRECTION
- EXISTING CULVERT
- APPROXIMATE LOCATION OF ORIGINAL LEDGE STONE ABUTMENTS & PIERS (AS BUILT)
- APPROXIMATE LOCATION OF 1950 SOUNDINGS

0 15 30
1" = 30' FEET

NOTES

1. THE BORING LOCATIONS SHOWN IN THIS FIGURE ARE APPROXIMATE AND ARE BASED ON HAND TAPED MEASUREMENTS TAKEN IN THE FIELD.
2. SEE BORING LOGS FOR DETAILED LITHOLOGIC DESCRIPTIONS.
3. THE LOCATIONS OF THE SOUNDINGS AND ORIGINAL ABUTMENTS & PIERS ARE APPROXIMATE AND ARE TAKEN FROM THE 1950 STATE HIGHWAY COMMISSION BRIDGE DIVISION DRAWINGS FOR TWIN BRIDGES OVER WEST BRANCH SOUADABSCOOK STREAM IN HAMPDEN. DEMOLITION DETAILS OF THE ORIGINAL STRUCTURE ARE UNKNOWN.
4. APPROXIMATE LOCATIONS OF POSSIBLE FORMER RETAINING WALL, FORMER ARCH BRIDGE ABUTMENT, BEDROCK, AND UNKNOWN BURIED OBJECTS ARE BASED ON HAGER-RICHTER'S INTERPRETATION OF GEOPHYSICAL SURVEY PERFORMED ON SEPTEMBER 25, 2020. ACTUAL LOCATIONS MAY VARY. REFER TO THE COMPLETE HAGER-RICHTER GEOPHYSICAL REPORT DATED OCTOBER 14, 2020 FOR MORE INFORMATION.

REFERENCES

1. BASEMAP ELEMENTS FROM WSP DRAWING TITLED "3D Topo_120616" DATED ON DECEMBER 05, 2016. RECEIVED BY GOLDER ASSOCIATES ON APRIL 27, 2017.
2. BORINGS WERE LOCATED AND OBSERVED BY GOLDER AND DRILLED BY NEW ENGLAND BORING CONTRACTORS OF HERMON, MAINE ON MAY 15-16, 2017.
3. BRIDGE ABUTMENT AND WINGWALL EXTENTS FROM 1950 BRIDGE DRAWINGS TITLED "TWIN BRIDGES OVER THE WEST BRANCH OF SOUADABSCOOK STREAM" BY THE STATE HIGHWAY COMMISSION DATED FEBRUARY 1950.
4. GEOPHYSICAL SURVEY INFORMATION FROM FILE "ACAD-Figure2-Figure2.dgn" PROVIDED TO GOLDER BY HAGER-RICHTER ON OCTOBER 14, 2020.

CLIENT
WSP
428 DOW HIGHWAY
ELLIOT, MAINE 03903

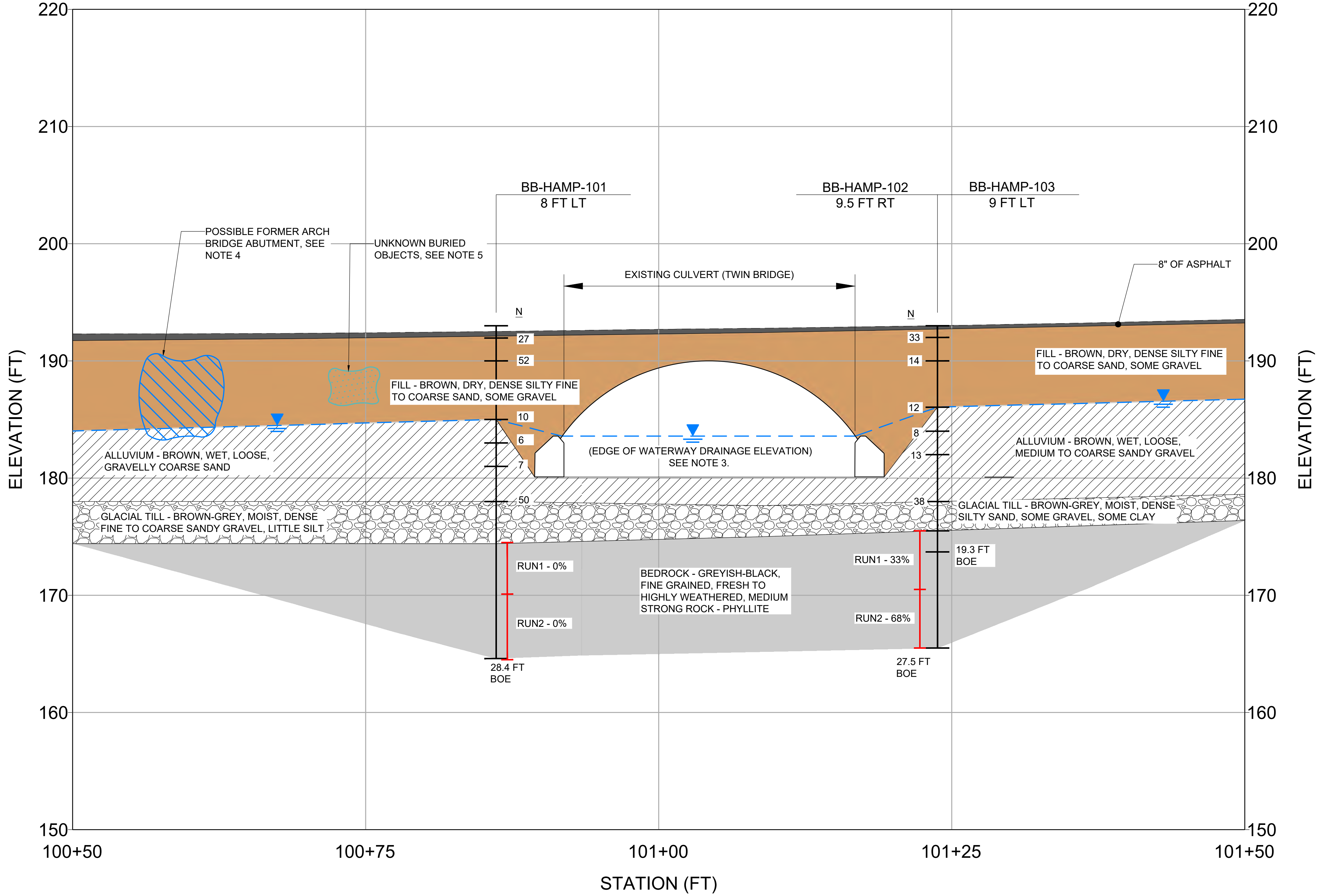
CONSULTANT
GOLDER

YYYY-MM-DD	2017-06-15
DESIGNED	LLM
PREPARED	RWC
REVIEWED	MCM
APPROVED	MSP

PROJECT
TWIN BRIDGE #5315 OVER WEST BRANCH SOUABABSCOOK STREAM
HAMPDEN, MAINE
MAINEDOT WIN 18959.00

TITLE
BORING LOCATION PLAN

PROJECT NO.	SUBTITLE	REV.	FIGURE
177-2224	A	0	2



LEGEND

- BB-HAMP-102 / 9.5FT RT — BORING LOCATION I.D. / OFFSET
- 14 — SPT : N_{60} - VALUE
- GROUNDWATER ELEVATION (FT) DURING DRILLING (SEE NOTE 3)
- RUN1 - 33% — ROCK CORE RUN NUMBER AND RQD
- 27.4 FT BOE — BOTTOM OF EXPLORATION
- ASPHALT
- FILL - BROWN, DRY, DENSE SILTY FINE TO COARSE SAND, SOME GRAVEL
- ALLUVIUM - BROWN, WET, LOOSE, GRAVELLY SAND / SANDY GRAVEL
- GLACIAL TILL - BROWNISH-GRAY, MOIST, DENSE FINE TO COARSE SANDY GRAVEL / SILTY SAND
- BEDROCK - GRAYISH-BLACK, FINE GRAINED, FRESH TO HIGHLY WEATHERED, MEDIUM STRONG ROCK-PHYLLITE
- POSSIBLE FORMER ARCH BRIDGE ABUTMENTS, SEE NOTE 4
- UNKNOWN BURIED OBJECTS, SEE NOTE 5

NOTES

1. THE BORING LOCATIONS SHOWN IN THIS FIGURE ARE APPROXIMATE AND ARE BASED ON HAND TAPED MEASUREMENTS TAKEN IN THE FIELD.
2. SEE BORING LOGS FOR DETAILED LITHOLOGIC DESCRIPTIONS.
3. 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. WATER LEVEL SHOWN IN THE RIVER IS BASED ON THE EDGE OF WATERWAY ELEVATION SHOWN ON FIGURE 2 AND TAKEN FROM WSP BASEMAP.
4. THIS GENERALIZED SUBSURFACE 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 AND BEDROCK TRANSITIONS MAY VARY AND ARE PROBABLY MORE ERRATIC. FOR MORE SPECIFIC INFORMATION REFER TO EXPLORATION LOGS.
5. APPROXIMATE DEPTH AND EXTENT OF POSSIBLE FORMER ARCH BRIDGE ABUTMENTS ARE BASED ON HAGER-RICHTER'S INTERPRETATION OF THEIR GEOPHYSICAL SURVEY PERFORMED ON SEPTEMBER 25, 2020. ACTUAL DEPTH AND EXTENTS OF THE ABUTMENTS MAY VARY. REFER TO COMPLETE HAGER-RICHTER GEOPHYSICAL REPORT DATED OCTOBER 14, 2020 FOR MORE INFORMATION.
6. APPROXIMATE LOCATION OF UNKNOWN BURIED OBJECTS AS INTERPRETED BY HAGER-RICHTER INTERPRETED FROM THEIR GEOPHYSICAL SURVEY PERFORMED ON SEPTEMBER 25, 2020. ACTUAL DEPTH AND EXTENTS MAY VARY. REFER TO COMPLETE HAGER-RICHTER GEOPHYSICAL REPORT DATED OCTOBER 14, 2020 FOR MORE INFORMATION.

0 5 10
1" = 10' FEET

REFERENCES

1. BASEMAP ELEMENTS FROM WSP DRAWING TITLED "3D Topo_120616" DATED ON DECEMBER 05, 2016. RECEIVED BY GOLDER ASSOCIATES ON APRIL 27, 2017.
2. BORINGS WERE LOCATED AND OBSERVED BY GOLDER AND DRILLED BY NEW ENGLAND BORING CONTRACTORS OF HERMON, MAINE ON MAY 15-16, 2017.
3. BRIDGE ABUTMENT AND WINGWALL EXTENTS FROM 1950 BRIDGE DRAWINGS TITLED "TWIN BRIDGES OVER THE WEST BRANCH OF SOUADABSCOOK STREAM."
4. GEOPHYSICAL SURVEY INFORMATION FROM FILE "ACAD-Figure2-Figure2.dgn" PROVIDED TO GOLDER BY HAGER-RICHTER ON OCTOBER 14, 2020.

CLIENT
WSP
428 DOW HIGHWAY
ELLIOT, MAINE 03903

CONSULTANT
GOLDER

YYYY-MM-DD	2017-06-15
DESIGNED	LLM
PREPARED	RWC
REVIEWED	MCM
APPROVED	MSP

PROJECT
TWIN BRIDGE #5315 OVER WEST BRANCH SOUABABSCOOK STREAM
HAMPDEN, MAINE
MAINEDOT WIN 18959.00

TITLE
INTERPRETED SUBSURFACE PROFILE A-A'

PROJECT NO.	SUBTITLE	REV.	FIGURE
1772224	A	0	3

APPENDIX A

BORING LOGS

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Twin Bridge #5315 Over West Branch Souababscook Stream
Location: Hampden, Maine

Boring No.: BB-HAMP-101
WIN: 18959.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	192.8	**Auger ID/OD:**	NA/4"
Operator:	Tom Schaefer	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	TRM	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140 lbs./30"
Date Start/Finish:	5/16/17 7:55 AM / 1:30 PM	**Drilling Method:**	Cased Wash	**Core Barrel:**	1-7/8" - NQ
Boring Location:	Sta. 100+86.1, 7.2FT L	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	See Remarks
Hammer Efficiency Factor:	0.6	**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	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	192.1		8" of asphalt. 0.7	
	1D	24/14.4	1.00 - 3.00	15/14/13/21	27	27				1D: Brown, dry, medium dense, fine to coarse SAND, some gravel, little silt; quartz rock fragments (FILL).	
	2D	24/14.4	3.00 - 5.00	5/21/31/25	52	52				2D: Brown, dry, very dense, fine to coarse SAND, some angular gravel, little silt; quartz fragments (FILL).	#412731 A-1-a/SM WC: 6.4%
5							56				
							46				
							29			Cobble at 7' bgs.	
	3D	24/3.6	8.00 - 10.00	6/5/5/9	10	10	6	184.8		8.0 3D: Brown, wet, loose, fine to coarse SAND, some sub-angular gravel, some silt (ALLUVIUM).	
							13				
10	4D	24/6	10.00 - 12.00	5/4/2/3	6	6	7			4D: Brown, wet, loose, gravelly, coarse SAND, trace silt, 2" piece of rounded gravel (ALLUVIUM).	
							14				
	5D	24/7.2	12.00 - 14.00	3/3/4/7	7	7	6			5D: 3" Spoon: Brown, wet, loose, gravelly, medium to coarse SAND, trace silt, 0.5" to 1" gravel; sub-angular to sub-rounded (ALLUVIUM).	#412732 A-1-a/SP WC: 11.5%
							8				
							8				
15	6D	24/18	15.00 - 17.00	21/20/30/(5/1")	50	50	55	177.8		15.0 6D: 3" spoon: Brown-gray, moist, dense, fine to coarse sandy GRAVEL, little silt; white quartz bedrock fragments (GLACIAL TILL).	#412733 A-1-b/GM WC: 8.4%
							74				
							56/10				
	R1	52.8/52.8	18.50 - 22.90	RQD = 0%			NQ	174.3		18.5 Top of Bedrock at 18.5ft bgs. R1: Bedrock: Greyish black (N2), fine grained, fresh to slightly weathered (W1-W2), medium strong rock (R3), PHYLLITE, discontinuity spacing is extremely close to closely spaced (<20 mm to 80 mm), dipping 55 to 85 degrees relative to horizontal axis, discontinuity surfaces are planar to curved to stepped, texture is smooth to very rough, trace pyrite [Vassalboro Formation]. Core Times: (min:sec) 18.5' - 19.5': (1:29) 19.5' - 20.5': (1:20)	
20											
	R2	67.2/57.2	22.90 - 28.50	RQD = 0%							
25											

Remarks:

- Borehole collapsed to 6.2' bgs. after casing was removed. Water was absent from hole. Water level based on observed saturation of soil samples.
- Boring backfilled with gravelly cuttings and 30 lbs of cold patch.
- Elevation based on base map data provided by WSP.

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: Twin Bridge #5315 Over West Branch Souababscook Stream
Location: Hampden, Maine

Boring No.: BB-HAMP-101
WIN: 18959.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	192.8	**Auger ID/OD:**	NA/4"
Operator:	Tom Schaefer	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	TRM	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140 lbs./30"
Date Start/Finish:	5/16/17 7:55 AM / 1:30 PM	**Drilling Method:**	Cased Wash	**Core Barrel:**	1-7/8" - NQ
Boring Location:	Sta. 100+86.1, 7.2FT L	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	See Remarks
Hammer Efficiency Factor:	0.6	**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 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								164.3		20.5' - 21.5': (2:10) 21.5' - 22.5': (2:08) 22.5' - 22.9': (0:41) Recovery 100% R2: Bedrock: Greyish black (N2), fine grained, fresh to highly weathered (W1 W4) with highly weathered zone from 26.8 feet to 28.5 feet below ground surface, medium weak rock (R3), weathered zone is very weak rock (R1), PHYLLITE, spacing is extremely close to closely spaced (<20 mm to 100 mm), dipping 85 to 90 degrees relative to horizontal axis, discontinuity surfaces are planar to stepped, texture is smooth to very rough, trace pyrite and iron staining in joints [Vassalboro Formation]. Core Times: (min:sec) 22.9' - 23.9': (1:53) 23.9' - 24.9': (1:37) 24.9' - 25.9': (1:19) 25.9' - 26.9': (1:59) 26.9' - 27.9': (1:51) 27.9' - 28.5': (1:32) Recovery 85% 28.5 **Bottom of Exploration at 28.5 feet below ground surface.**	
30											
35											
40											
45											
50											

Remarks:

- Borehole collapsed to 6.2' bgs. after casing was removed. Water was absent from hole. Water level based on observed saturation of soil samples.
- Boring backfilled with gravelly cuttings and 30 lbs of cold patch.
- Elevation based on base map data provided by WSP.

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-HAMP-101

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Twin Bridge #5315 Over West Branch Souababscook Stream Location: Hampden, Maine	Boring No.: BB-HAMP-102 WIN: 18959.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	192.2	**Auger ID/OD:**	NA/4"
Operator:	Tom Schaefer	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	TRM	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140 lbs./30"
Date Start/Finish:	5/15/17 8:30 AM/2:45 PM	**Drilling Method:**	Cased Wash	**Core Barrel:**	1-7/8" - NQ
Boring Location:	Sta. 101+24.3, 10.2FT R	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	See Remarks
Hammer Efficiency Factor:	0.6	**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	191.5		8" of asphalt. 0.7	
	1D	24/12	1.00 - 3.00	14/12/21/28	33	33				1D: Brown, dry, dense, fine to coarse SAND, some gravel, little silt (FILL).	
	2D	24/12	3.00 - 5.00	26/7/7/5	14	14				2D: Brown, dry, medium dense, Silty fine to medium SAND, some gravel (FILL).	
5							14				
							24	185.2		7.0	
	3D	24/4.8	7.00 - 9.00	6/6/6/4	12	12	12			3D: Brown, wet, medium dense, Gravelly coarse SAND, some silt, angular (ALLUVIUM).	
							22				
	4D	24/4.8	9.00 - 11.00	6/5/3/3	8	8	17			4D: Brown, wet, loose, medium to coarse Sandy GRAVEL, little silt, angular gravel (ALLUVIUM).	
10							24				
	5D	24/3.6	11.00 - 13.00	9/8/5/5	13	13	38			5D: Brown, wet, medium dense, medium to coarse Sandy GRAVEL, trace silt, rock fragments, angular gravel (ALLUVIUM). 3" spoon used to collect additional sample material.	#412734 A-1-a/GW WC: 9.4%
							44				
							49			3" to 4" gravel/rock in casing at 13' to 14'.	
							28	177.2		15.0	
15	6D	24/16.8	15.00 - 17.00	12/19/19/81	38	38	64			6D: Brown-grey, moist, dense, silty SAND, some gravel, some clay, cobbles (GLACIAL TILL). 3" to 4" cobble at 16' in spoon. Milky gray wash from 15' to 17' bgs.	#412735 A-4/SM WC: 7.7%
							143	174.7		17.5	
	R1	60/50	17.50 - 22.50	RQD = 33%			NQ			Top of Bedrock at 17.5' bgs.	#306502 q_p: 11473 psi
										R1: Bedrock: Medium dark grey (N4), fine grained, fresh (W1), medium strong rock (R3), PHYLLITE, discontinuity spacing is extremely close to moderately spaced (<20 mm to 460 mm), dipping 55 to 85 degrees relative to horizontal axis, discontinuity surfaces are planar to curved to stepped, texture is smooth to rough [Vassalboro Formation].	
20											
	R2	60/56	22.50 - 27.50	RQD = 68%						Core Times (min:sec): 17.5' - 18.5': (1:20) 18.5' - 19.5': (1:25) 19.5' - 20.5': (1:24)	
25											

Remarks:

- Borehole collapse to 5.2' b.g.s. after casing was removed. Water was absent from the hole. Water level based on observed saturation of soil samples.
- Boring backfilled with cuttings and 3 bags of all purpose gravel (Quikrete) and 20 lbs. cold patch.
- Elevation based on base map data provided by WSP.

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: Twin Bridge #5315 Over West Branch Souababscook Stream
Location: Hampden, Maine

Boring No.: BB-HAMP-102
WIN: 18959.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	192.2	**Auger ID/OD:**	NA/4"
Operator:	Tom Schaefer	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	TRM	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140 lbs./30"
Date Start/Finish:	5/15/17 8:30 AM/2:45 PM	**Drilling Method:**	Cased Wash	**Core Barrel:**	1-7/8" - NQ
Boring Location:	Sta. 101+24.3, 10.2FT R	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	See Remarks
Hammer Efficiency Factor:	0.6	**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 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							↓	164.7		20.5' - 21.5': (1:50) 21.5' - 22.5': (2:08) Recovery 83% R2: Bedrock: Medium dark grey (N4), fine grained, fresh (W1), medium strong (R3), PHYLLITE, discontinuity spacing is very close to moderately spaced, dipping 55 to 85 degrees relative to horizontal axis, discontinuity surfaces are planar to curved, texture is smooth to rough, white seam observed at about 23.5 feet below ground surface [Vassalboro Formation]. Core Times (min:sec): 22.5' - 23.5': (1:32) 23.5' - 24.5': (1:13) 24.5' - 25.5': (1:05) 25.5' - 26.5': (1:06) 26.5' - 27.5': (1:04) Recovery 93%	
										27.5 **Bottom of Exploration at 27.5 feet below ground surface.**	
30											
35											
40											
45											
50											

Remarks:

- Borehole collapse to 5.2' b.g.s. after casing was removed. Water was absent from the hole. Water level based on observed saturation of soil samples.
- Boring backfilled with cuttings and 3 bags of all purpose gravel (Quikrete) and 20 lbs. cold patch.
- Elevation based on base map data provided by WSP.

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-HAMP-102

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Twin Bridge #5315 Over West Branch Souababscook Stream
Location: Hampden, Maine

Boring No.: BB-HAMP-103
WIN: 18959.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	193.5	**Auger ID/OD:**	NA/4"
Operator:	Tom Schaefer	**Datum:**	NAVD 88	**Sampler:**	NA
Logged By:	TRM	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	NA
Date Start/Finish:	5/16/2017 1:55 PM / 2:30 PM	**Drilling Method:**	SSA	**Core Barrel:**	NA
Boring Location:	Sta. 101+23.8, 8.6FT L	**Casing ID/OD:**	NA	**Water Level*:**	Not Observed
Hammer Efficiency Factor:	0.6	**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	N60	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	192.8		8" of asphalt. 0.7	
5											
										Overburden above till layer suspected to be composed of fill and alluvium similar to borings BB-HAMP-101 and BB-HAMP-102	
10											
15								177.5		16.0 Till layer inferred from drilling behavior	
								174.2		Tri-cone refusal. Suspected top of bedrock. 19.3	
20										**Bottom of Exploration at 19.3 feet below ground surface.**	
25											

Remarks:

- No samples were taken.
- Boring collapse to 4.4' upon removal of augers.
- Finished with cuttings and 30 lbs of cold patch.

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.

APPENDIX B

ROCK CORE PHOTOS

APPENDIX B
Rock Core Photos
Twin Bridge #5315 Over West Branch Souadabscook Stream
Hampden, Maine
MaineDOT WIN: 18959.00

			Recovery:		RQD:	
Boring:	**Run:**	**Depth:**	**IN:**	**%:**	**IN:**	**%:**
BB-HAMP-101	R1	18.5-22.9’	53/53”	100%	0/53”	00%
Cored 5/16/17	R2	22.9-28.5’	57/67”	85%	0/67”	00%
BB-HAMP-102	R1	17.5-22.5’	50/60”	83%	20/60”	33%
Cored 5/15/17	R2	22.5-27.5’	56/60”	93%	40.5/60”	68%

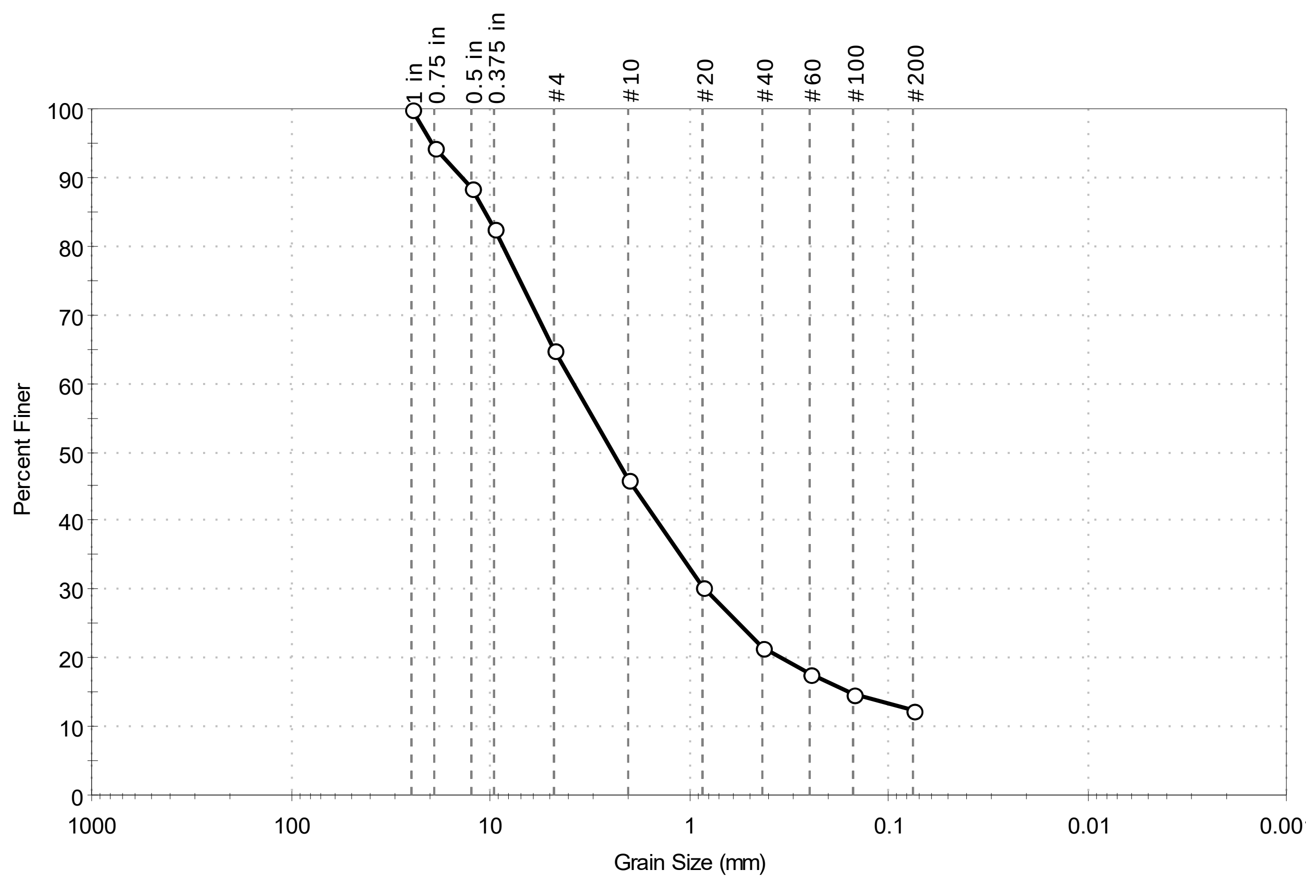
Row 1 = BB-HAMP-102 Run 1: 17.5 - 22.5 ft-bgs.
Row 2 = BB-HAMP-102 Run 2: 22.5 - 27.5 ft-bgs.
Row 3 = BB-HAMP-101 Run 1: 18.5 - 22.9 ft-bgs.
Row 4 = BB-HAMP-101 Run 2: 22.9 - 28.5 ft-bgs

APPENDIX C

LABORATORY TEST RESULTS

Client: Golder Associates			
Project: Rt 69 Culvert Replacement			
Location: Hampden, ME		Project No:	GTX-306502
Boring ID: BB-HAMP-101	Sample Type: jar	Tested By:	jbr
Sample ID: 2	Test Date: 06/02/17	Checked By:	emm
Depth : 3-5 ft	Test Id: 412731		
Test Comment: ---			
Visual Description: Moist, brown silty sand with gravel			
Sample Comment: ---			

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	35.1	52.7	12.2

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	94		
0.5 in	12.50	89		
0.375 in	9.50	82		
#4	4.75	65		
#10	2.00	46		
#20	0.85	30		
#40	0.42	22		
#60	0.25	18		
#100	0.15	15		
#200	0.075	12		

Coefficients

D_{85} =10.6484 mm | D_{30} =0.8324 mm

D_{60} =3.7963 mm | D_{15} =0.1548 mm

D_{50} =2.4037 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-a (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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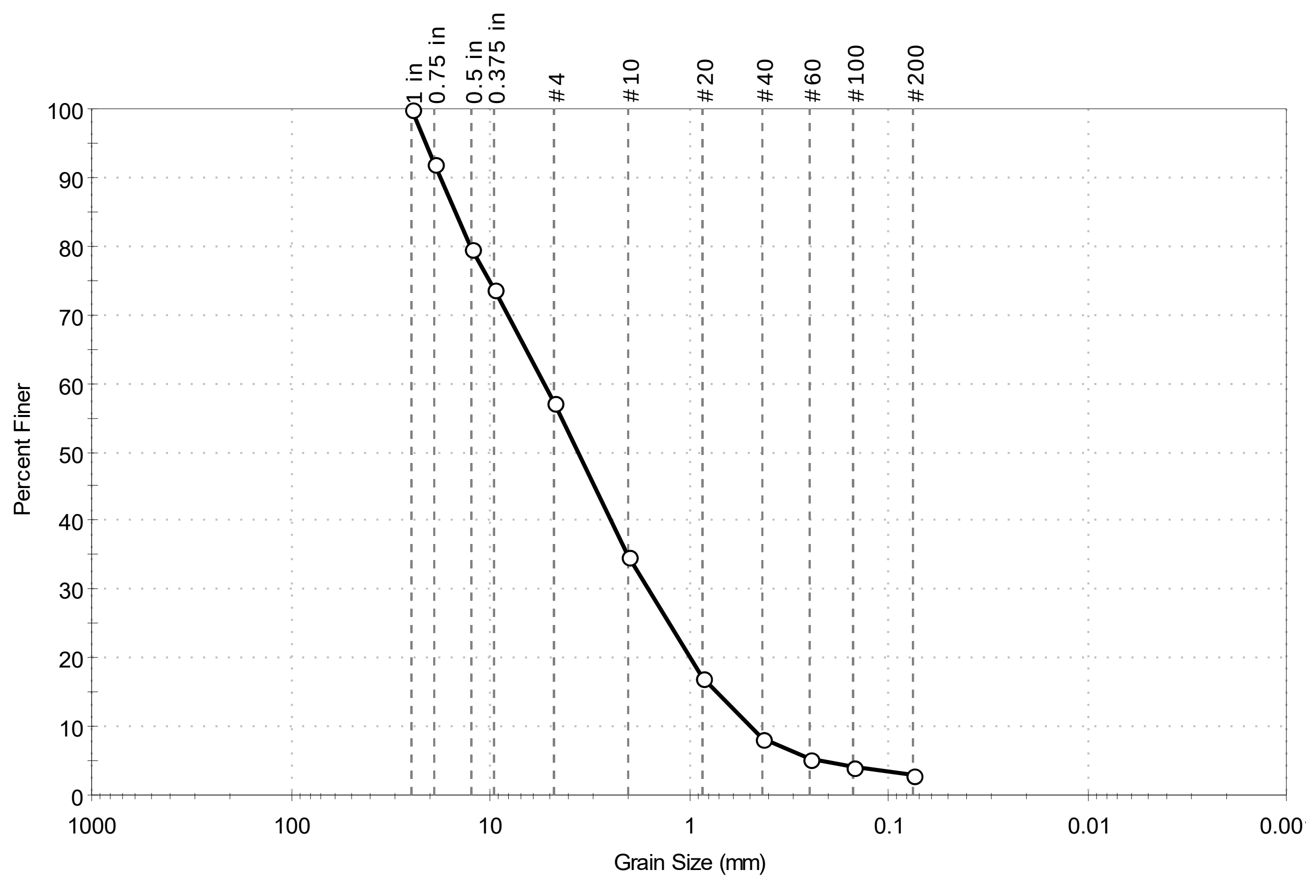
Client:	Golder Associates		
Project:	Rt 69 Culvert Replacement		
Location:	Hampden, ME	Project No:	GTX-306502
Boring ID: BB-HAMP-101	Sample Type: jar	Tested By:	jbr
Sample ID: 5	Test Date: 06/02/17	Checked By:	emm
Depth : 12-14 ft	Test Id: 412732		
Test Comment:	---		
Visual Description:	Moist, grayish brown sand with gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	42.8	54.2	3.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	92		
0.5 in	12.50	80		
0.375 in	9.50	74		
#4	4.75	57		
#10	2.00	35		
#20	0.85	17		
#40	0.42	8		
#60	0.25	5		
#100	0.15	4		
#200	0.075	3.0		

Coefficients

D_{85} = 14.9915 mm D_{30} = 1.5879 mm

D_{60} = 5.3418 mm D_{15} = 0.7256 mm

D_{50} = 3.5971 mm D_{10} = 0.4901 mm

C_u = 10.899 C_c = 0.963

Classification

ASTM Poorly graded sand with gravel (SP)

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

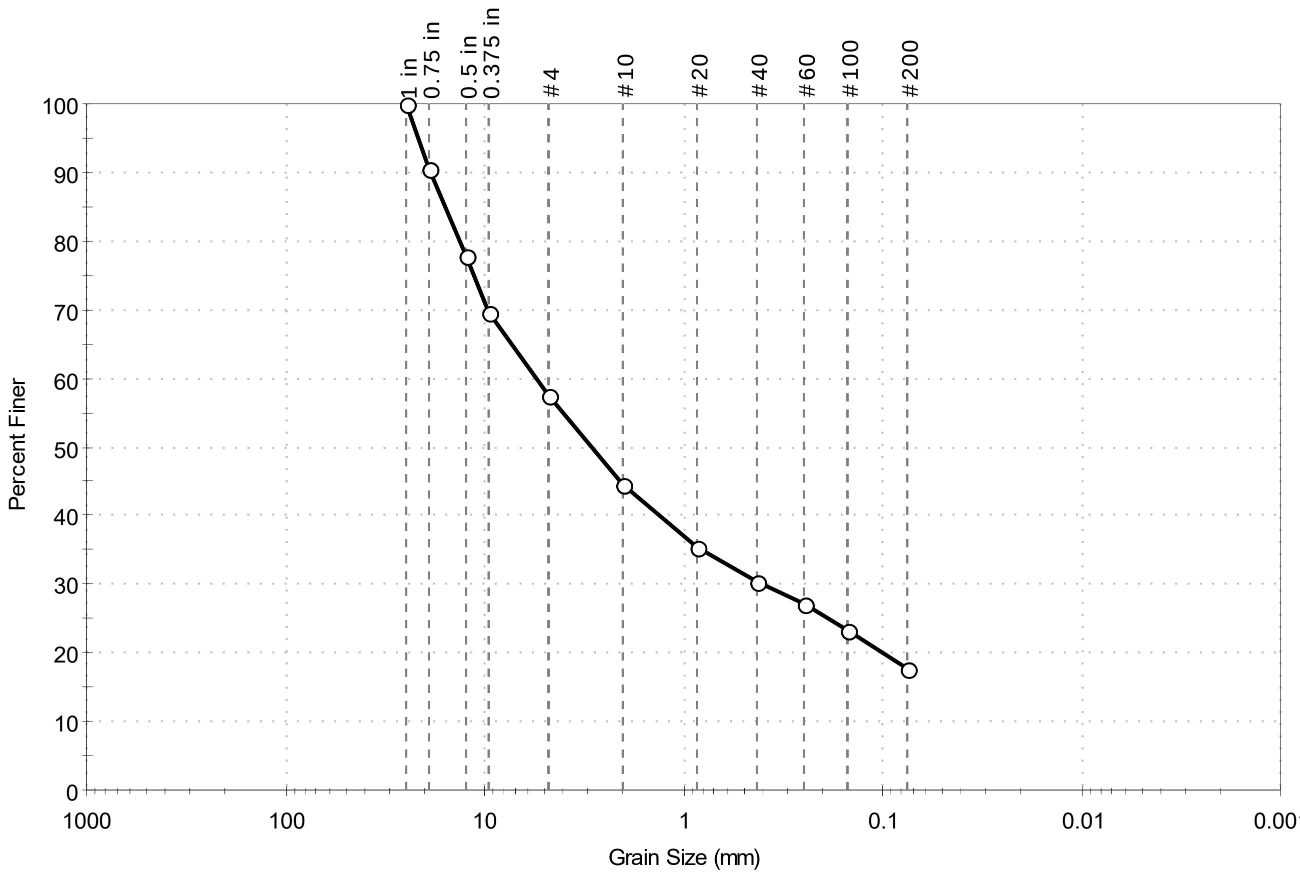
Sand/Gravel Hardness : HARD

printed 6/6/2017 11:27:27 AM

Client:	Golder Associates		
Project:	Rt 69 Culvert Replacement		
Location:	Hampden, ME	Project No:	GTX-306502
Boring ID: BB-HAMP-101	Sample Type: jar	Tested By:	jbr
Sample ID: 6	Test Date: 06/02/17	Checked By:	emm
Depth : 15-17 ft	Test Id: 412733		

Test Comment: ---
Visual Description: Moist, grayish brown silty gravel with sand
Sample Comment: ---

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	42.5	39.7	17.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	91		
0.5 in	12.50	78		
0.375 in	9.50	70		
#4	4.75	57		
#10	2.00	45		
#20	0.85	35		
#40	0.42	30		
#60	0.25	27		
#100	0.15	23		
#200	0.075	18		

Coefficients

D_{85} =15.7974 mm　　D_{30} =0.3958 mm
D_{60} =5.4821 mm　　D_{15} =N/A
D_{50} =2.8713 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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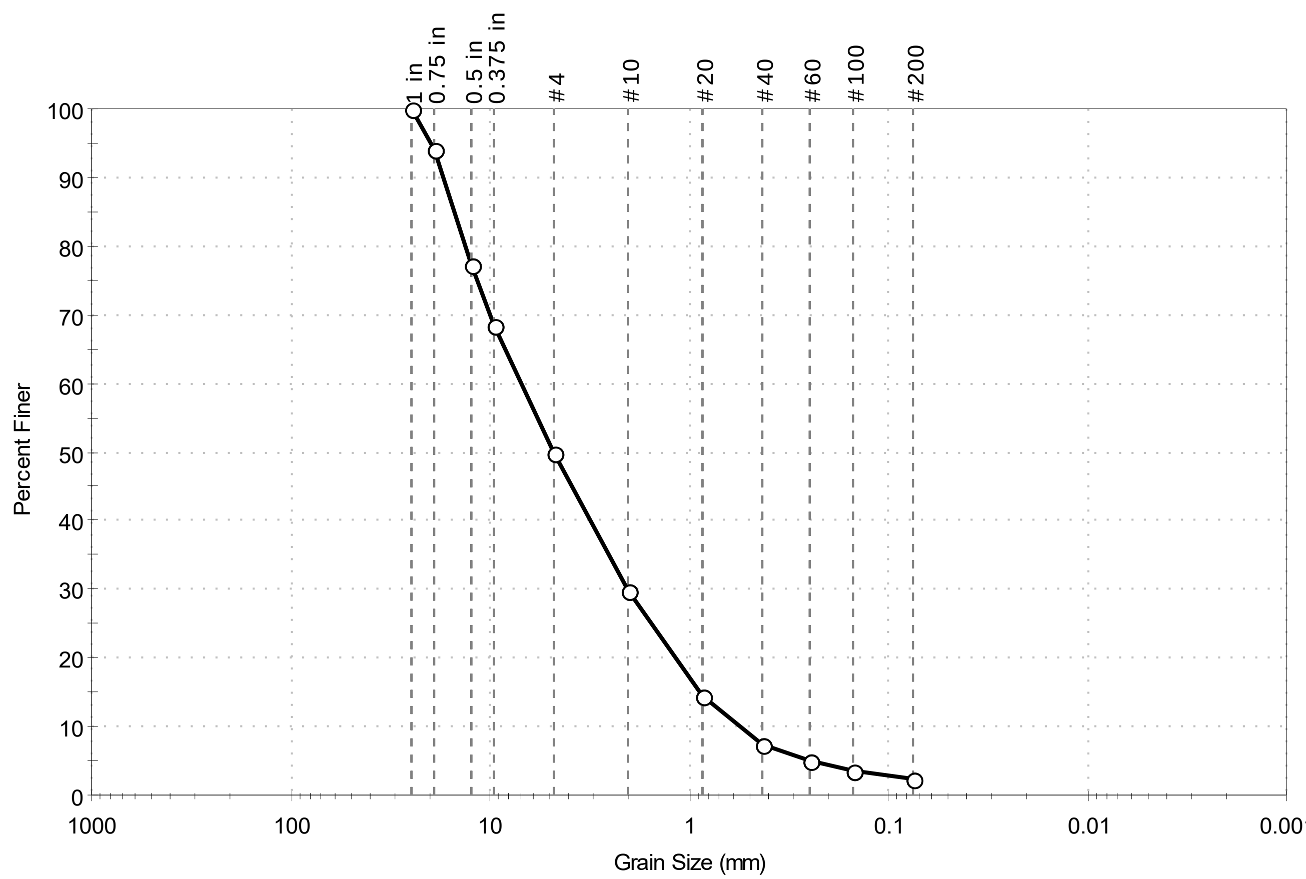
Client: Golder Associates			
Project: Rt 69 Culvert Replacement			
Location: Hampden, ME		Project No:	GTX-306502
Boring ID: BB-HAMP-102	Sample Type: jar	Tested By:	jbr
Sample ID: 5	Test Date: 06/02/17	Checked By:	emm
Depth : 11-13 ft	Test Id: 412734		
Test Comment: ---			
Visual Description: Moist, gray gravel with sand			
Sample Comment: ----			

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	50.0	47.5	2.5

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	94		
0.5 in	12.50	77		
0.375 in	9.50	68		
#4	4.75	50		
#10	2.00	30		
#20	0.85	14		
#40	0.42	7		
#60	0.25	5		
#100	0.15	4		
#200	0.075	2.5		

Coefficients

D_{85} = 15.1833 mm D_{30} = 2.0148 mm

D_{60} = 6.9213 mm D_{15} = 0.8765 mm

D_{50} = 4.7614 mm D_{10} = 0.5496 mm

C_u = 12.593 C_c = 1.067

Classification

ASTM Well-graded gravel with sand (GW)

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

printed 6/6/2017 11:27:29 AM

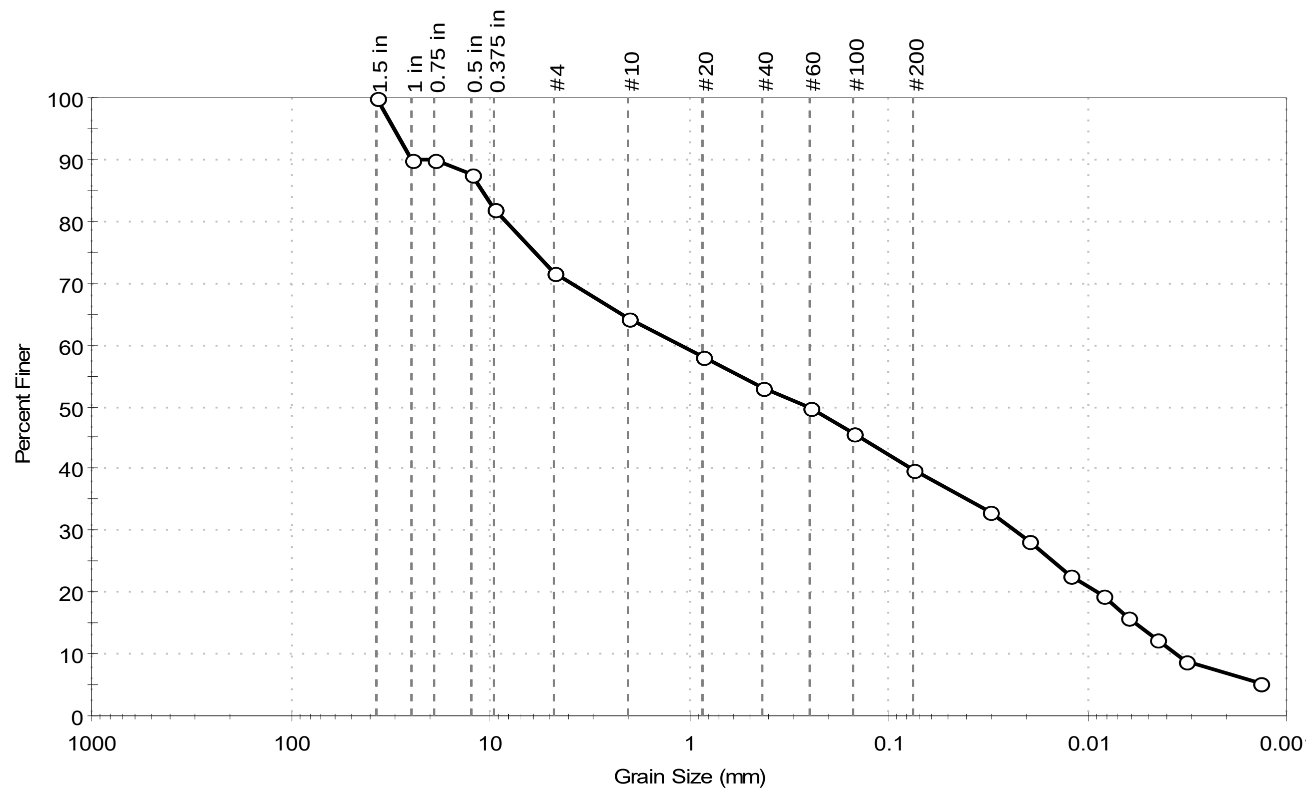
Client:	Golder Associates		
Project:	Rt 69 Culvert Replacement		
Location:	Hampden, ME	Project No:	GTX-306502
Boring ID: BB-HAMP-102	Sample Type: jar	Tested By:	jbr
Sample ID: 6	Test Date: 06/06/17	Checked By:	emm
Depth : 15-17 ft	Test Id: 412735		

Test Comment: ---

Visual Description: Moist, olive gray silty sand with gravel

Sample Comment: ---

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
—	28.3	31.8	39.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	90		
0.75 in	19.00	90		
0.5 in	12.50	88		
0.375 in	9.50	82		
#4	4.75	72		
#10	2.00	64		
#20	0.85	58		
#40	0.42	53		
#60	0.25	50		
#100	0.15	46		
#200	0.075	40		
---	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0308	33		
---	0.0197	28		
---	0.0122	23		
---	0.0085	19		
---	0.0062	16		
---	0.0045	12		
---	0.0033	9		
---	0.0014	5		

Coefficients

D_{85} = 11.0331 mm D_{30} = 0.0230 mm

D_{60} = 1.1084 mm D_{15} = 0.0057 mm

D_{50} = 0.2602 mm D_{10} = 0.0036 mm

C_u = 307.889 C_c = 0.133

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Dispersion Device : Apparatus A - Mech Mixer

Dispersion Period : 1 minute

Specific Gravity : 2.65

Separation of Sample: #200 Sieve

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Client:	Golder Associates				
Project:	Rt 69 Culvert Replacement				
Location:	Hampden, ME			Project No:	GTX-306502
Boring ID:	BB-HAMP-102	Sample Type:	cylinder	Tested By:	rlc
Sample ID:	R1	Test Date:	06/05/17	Checked By:	jsc
Depth :	17.7-19 ft	Test Id:	412758		
Test Comment:	---				
Visual Description:	See photograph(s)				
Sample Comment:	---				

Bulk Density and Compressive Strength of Rock Core Specimens by ASTM D7012 Method C

Boring ID	Sample Number	Depth	Bulk Density, pcf	Compressive strength, psi	Failure Type	Meets ASTM D4543	Note(s)
BB-HAMP-102	R1	17.91-18.29 ft	168	11473	2	Yes	---

Notes: Density determined on core samples by measuring dimensions and weight and then calculating.

All specimens 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.

Failure Type: 1 = Intact Material Failure; 2 = Discontinuity Failure; 3 = Intact Material and Discontinuity Failure
(See attached photographs)

GeoTesting EXPRESS

Client:	Golder Associates	Test Date:	6/5/2017
Project Name:	Rt 69 Culvert Replacement	Tested By:	rlc
Project Location:	Hampden, ME	Checked By:	jsc
GTX #:	306502		
Boring ID:	BB-HAMP-102		
Sample ID:	R1		
Depth:	17.91-18.29 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:	4.19	4.19	4.19		
Specimen Diameter, in:	1.99	1.99	1.99		
Specimen Mass, g:	577.25				
Bulk Density, lb/ft^3	168			**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.? 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	-0.00020	-0.00010	0.00000	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010
Diameter 2, in (rotated 90^o)	-0.00030	-0.00020	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020

Difference between max and min readings, in:

0° = 0.00030 90° = 0.00030

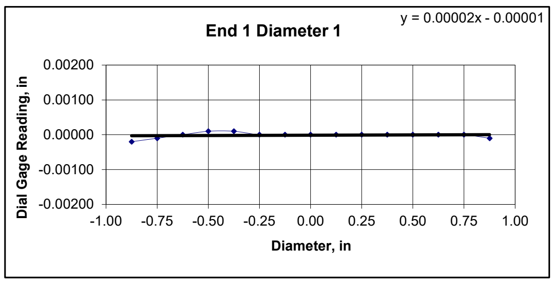
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00010	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010
Diameter 2, in (rotated 90^o)	0.00010	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00000	0.00000	-0.00010	-0.00020

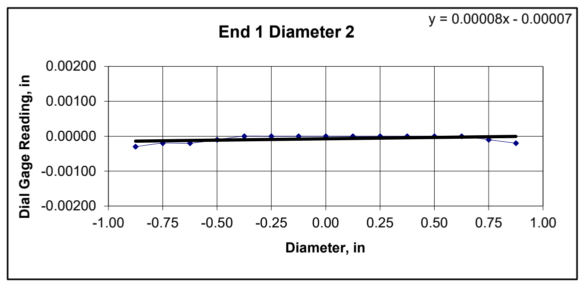
Difference between max and min readings, in:

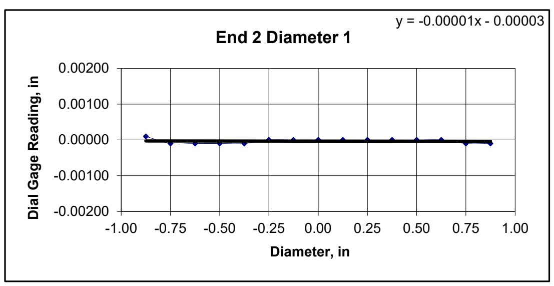
0° = 0.0002 90° = 0.0003

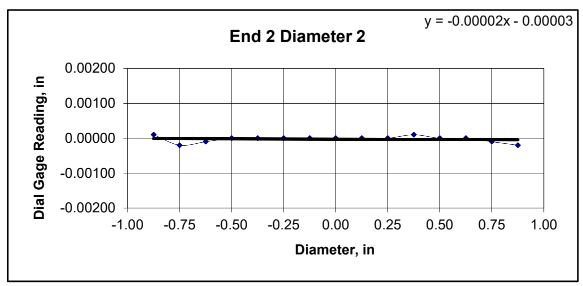
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00015

Flatness Tolerance Met? YES









DIAMETER 1

End 1:

Slope of Best Fit Line	0.00002
Angle of Best Fit Line:	0.00115

End 2:

Slope of Best Fit Line	0.00001
Angle of Best Fit Line:	0.00057

Maximum Angular Difference: 0.00057

Parallelism Tolerance Met? YES
Spherically Seated

DIAMETER 2

End 1:

Slope of Best Fit Line	0.00008
Angle of Best Fit Line:	0.00458

End 2:

Slope of Best Fit Line	0.00002
Angle of Best Fit Line:	0.00115

Maximum Angular Difference: 0.00344

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	0.00030	1.990	0.00015	0.009	YES
Diameter 2, in (rotated 90^o)	0.00030	1.990	0.00015	0.009	YES
END 2					
Diameter 1, in	0.00020	1.990	0.00010	0.006	YES
Diameter 2, in (rotated 90^o)	0.00030	1.990	0.00015	0.009	YES

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

Perpendicularity Tolerance Met? YES

Client:	Golder Associates
Project Name:	Rt 69 Culvert Replacement
Project Location:	Hampden, ME
GTX #:	306502
Test Date:	6/5/2017
Tested By:	rlc
Checked By:	jsc
Boring ID:	BB-HAMP-102
Sample ID:	R1
Depth, ft:	17.91-18.29

After cutting and grinding

After break

APPENDIX D

CALCULATIONS

CALCULATIONS

Date:	6/13/2017	**Made by:**	CJS
Project No.:	1772224	**Checked by:**	MCM
Subject:	Bedrock Erodibility	**Reviewed by:**	MSP
Project Short Title:	Route 69 Culvert Replacement		

1.0 Purpose
To estimate the erodibility of bedrock below the proposed footings for a three sided arch structure with an open (natural streambed) bottom at the West Branch Souadabscook Stream.

2.0 References
1) Annandale, George and Smith, Steve (2001). "Calculation of Bridge Pier Scour Using The Erodibility Index Method," Colorado Department of Transportation Research Report No. CDOT-DTD-R-2000-9.
2) StreamStats data output for 44.71211 latitude, -68.95101 longitude. (2017). Version Alpha 4.1.3. United States Geological Survey. <https://streamstats.usgs.gov/>
3) MEDOT Highway Bridge Inspection Report, TWIN RTE #69 over W.BR.SOUADABSCOOK STR., completed on 06/15/2016.
4) Golder Associates boring log, BB-HAMP-101.
5) USDA, NRCS. (2012). National Engineering Handbook, Part 631 Geology, Chapter 4: Engineering Classification of Rock Materials.
6) Golder Associtès boring location plan (Figure 2)

3.0 Assumptions
1) Two rock samples from Run 1 of BB-HAMP-101 were broken with a geologists hammer. One sample, near the top of the run (approximately 19.5ft bgs), required more than one hammer blow to break the core sample (hard rock, Table 4-3 Ref. 5) and a second sample, near the bottom of the run (approximately 22ft bgs), required only one geologists hammer blow to break the core sample (moderately hard rock, Table 4-3 Ref. 5). Based on this assume that the rock is borderline "moderately hard" to "hard" rock and use 7300 psi for the UCS.
2) Stream depth is roughly 4ft deep at the culvert per the MEDOT Inspection report (Reference 3).
3) The soil in and below the stream channel has eroded due to scour and bedrock is exposed.

3.0 Calculation
3.1 Erodibility Index

$$K = M_s \times K_b \times K_d \times J_s$$ Equation 6, Reference 1

where:
- M_s = intact material strength number
- K_b = block or particle size number
- K_d = discontinuity or inter-particle bond shear strength number
- J_s = relative shape and orientation number

M_s = **35** (Table 3 - Reference 1, for UCS =7,300psi [50.3MPa])

The bedrock on site is as low as RQD = 0, and is too low to determine K_d values for rock. Assume the bedrock is a cohesionless soil and use Equation 15 below (Reference 1).

$$K_b = 1000 \times (D_{50})^3$$ Equation 15, Reference 1

D_{50} = mean particle size at riverbed elevation (meters)

Assume the mean particle size at the river bed elevation is 3-inches. The RQD of the bedrock in BB-HAMP-101 is zero so all segments are 4-inch minus.

D_{50} = 0.076 meters, (3-inches)

Date:	6/13/2017	**Made by:**	CJS
Project No.:	1772224	**Checked by:**	MCM
Subject:	Bedrock Erodibility	**Reviewed by:**	MSP
Project Short Title:	Route 69 Culvert Replacement		

K_b = 0.44

Determine the discontinuity or inter-particle bond shear strength number based on visual observation of the rock core and Tables 5 & 6 (Reference 1).

$$K_d = {J_r}/{J_a}$$ Equation 16, Reference 1

where: J_r = degree of roughness of opposing faces of the rock discontinuity
J_a = degree of alteration of the materials on the faces of the discontinuity

Assume that the joints are tight and planar in shape. From Table 5 (Reference 1):

J_r = 1.0

Assume that joints are tightly healed, consisting of hard non-softening impermeable filling:

J_a = 0.75

K_d = 1.33

Based on observation of the rock core assume that the rock dips at 55° from the horizontal with blocks roughly twice as long as wide to determine the relative shape and orientation number. Assume the rock dips in the direction of the stream flow.

J_s = 0.46 from Table 7, (Reference 1).

Calculate the erodibility index:

K =	**M_s**	**x**	**K_b**	**x**	**K_d**	**x**	**J_s**
K =	35	**x**	0.44	**x**	1.33	**x**	0.46

K = 9.4

3.2 Stream Power

$$P_a = \gamma \times v \times d \times s$$

where:
γ = 9.81 kn/m^3, unit weight of water
d = 1.22 m, depth of flow at existing culvert from 2015 MEDOT inspection report
v = stream velocity, m/s
s = energy slope of the stream

Estimate the stream velocity and energy slope from the StreamStats data in Reference 2. The StreamStats data indicates that the bankfull flow of the stream is 32.6 ft wide. Based on the 2015 MEDOT Bridge Inspection Report, the stream is roughly 4ft deep under the existing culvert. Assuming a rectangular channel shape, the stream section has an area of 130.4 ft^2. The StreamStats data also indicates that the 100 Year Peak Flood is 1680 ft^3/s. Using the peak flood flow and the calculated stream section, calculate a velocity of 12.9 ft/s.

v = 3.9 m/s

Date:	6/13/2017	**Made by:**	CJS
Project No.:	1772224	**Checked by:**	MCM
Subject:	Bedrock Erodibility	**Reviewed by:**	MSP
Project Short Title:	Route 69 Culvert Replacement		

Based on the survey shown in Figure 2 of the design report, the river channel drops 2 feet over 48 feet planar (i.e. slope = 0.042). Assume for the purpose of this calculation that the stream is under uniform flow conditions and the slope angle is channel slope is equal to the energy slope. Therefore:

$s = 0.042$

Calculate the estimated stream power:

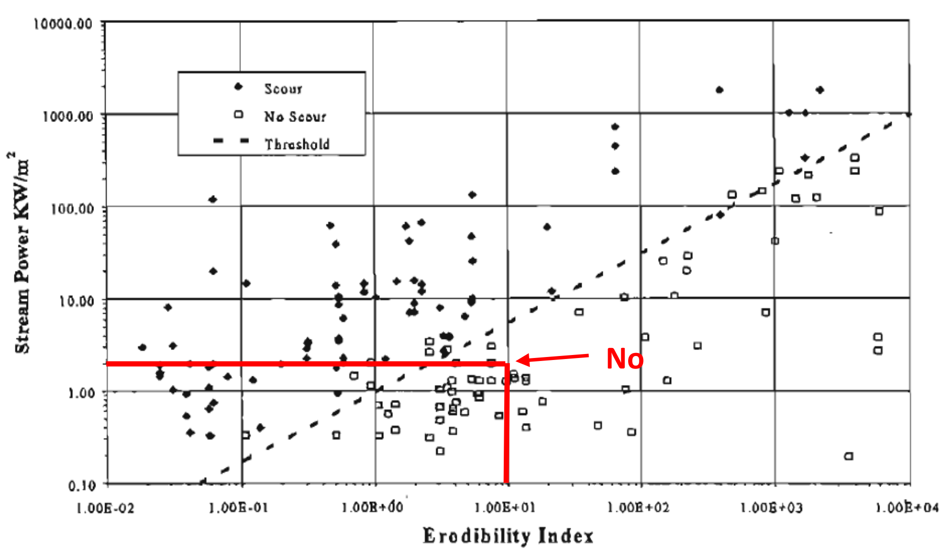
$Pa = \gamma \times d \times v \times s$

$Pa = 9.81 \times 1.22 \times 3.9 \times 0.042$

Pa = 2.0 kW/m^2

4.0 Conclusion

Using the stream power and erodibility index calculated above and Figure 4 from Reference 1 (below), it is concluded that scour of bedrock is not anticipated to occur. Furthermore this calculation was completed with the assumption that the bridge is founded on exposed bedrock which is conservative because there is roughly 5 feet of till lying above rock and would need to erode before reaching the foundation.



Ref 1

Report No. CDOT-DTD-R-2000-9

Calculation of Bridge Pier Scour Using The Erodibility Index Method

George Annandale
Steve Smith

DOT
DEPARTMENT OF TRANSPORTATION
RESEARCH

March 2001
Final Report

Ref 1

where d = flow depth (m), γ = unit weight of water (N/m^3), s = dimensionless energy slope (or bed slope in the case of uniform, steady flow), and v = velocity (m/s).

An estimate of y_{max} can be obtained by making use of the bridge pier scour equation in HEC-18 (FHWA 1995), which is based on an envelope curve embracing a large number of bridge pier scour experiments. This equation (presented below) is considered to provide a conservative estimate of scour depth.

$$\frac{y_s}{y_1} = 2.0 \cdot K_1 \cdot K_2 \cdot K_3 \cdot \left(\frac{a}{y_1}\right)^{0.65} Fr_1^{\ 0.43} \tag{5}$$

where y_s = scour depth (ft), y_1 = flow depth directly upstream of the pier (ft), K_1 = correction factor for pier nose shape, K_2 = correction factor for angle of attack of flow, K_3 = correction factor for bed condition, a = pier width (ft), L = length of pier (ft), Fr_1 = Froude Number = $V_1/(gy_1)^{1/2}$, and V_1 = mean velocity of flow directly upstream of the pier (ft/s).

With y_{max} assumed to be the maximum scour, the scour depth estimated with the Erodibility Index Method can never exceed this value. The range of scour depth estimates for this method is therefore $0 \leq y_s \leq y_{max}$.

Calculation of the Erodibility Index

The geo-mechanical index that is used by the Erodibility Index Method to quantify the relative ability of earth material to resist erosion was developed by Kirsten (1982) and is expressed as (equation (6)):

$$K = M_s \cdot K_b \cdot K_d \cdot J_s \tag{6}$$

The intact mass strength number (M_s) represents the strength of a homogenous, "perfect" sample of earth material. In order to acknowledge the roles of discontinuities and imperfections for de-

Table 3 contains the values of M_s for rock that are related to field identification tests and the unconfined compressive strength (*UCS*) of the rock, expressed in MPa. The latter can be quantified by making use of the procedures described in ASTM D-2938 (Standard Test Method for Unconfined Compressive Strength of Rock Core Specimens).

Table 3. Mass strength number for rock (M_s).

Hardness	Identification in Profile	Unconfined Compressive Strength (MPa)	Mass Strength Number (M_s)
Very soft rock	Material crumbles under firm (moderate) blows with sharp end of geological pick and can be peeled off with a knife; is too hard to cut tri-axial sample by hand.	Less than 1.7 1.7 – 3.3	0.87 1.86
Soft rock	Can just be scraped and peeled with a knife; indentations 1 mm to 3-mm show in the specimen with firm (moderate) blows of the pick point.	3.3 – 6.6 6.6 – 13.2	3.95 8.39
Hard rock	Cannot be scraped or peeled with a knife; hand-held specimen can be broken with hammer end of geological pick with a single firm (moderate) blow.	13.2 – 26.4	17.70
Very hard rock	Hand-held specimen breaks with hammer end of pick under more than one blow.	26.4 – 53.0 53.00 – 106.0	35.0 70.0
Extremely hard rock	Specimen requires many blows with geological pick to break through intact material.	Larger than 212.0	280.0

The values of M_s for rock can also be quantified by making use of the equations listed below.

$$M_s = C_r \cdot (0.78) \cdot (UCS)^{1.05} \text{ when } UCS \leq 10 \text{ Mpa} \quad (7)$$

and

$$M_s = C_r \cdot (UCS) \text{ when } UCS > 10 \text{ MPa} \quad (8)$$

Ref 1

Ref 1

Cohesive and Cohesionless, Granular Soils - K_b is set to one ($K_b = 1$) when indexing massive, intact **cohesive** soils. In the case of **cohesionless**, granular soils (including fine, medium, and coarse sands, and gravel and cobbles), the value of K_b is determined by means of the following equation:

$$K_b = 1000 \cdot (D_{50})^3 \text{ for } D_{50} < 0.1 \text{ m} \tag{15}$$

where D_{50} = median particle diameter (m) at the interface between the bed and the water.

The value of D_{50} is determined by standard gradation tests. If the interface between the bed and the water consists of an armor layer, the D_{50} of the armor layer is determined. If the interface does not consist of an armor layer, but an armor layer can potentially form during the scour process, then D_{50} can be set equal to the D_{85} diameter of the gradation of the bed material. The reason for this is that if an armor layer forms during the scour process, then the D_{50} of the armor layer will represent the particle size that protects the underlying bed material. For practical purposes it has been found that the D_{50} of this layer will be approximately equal to the D_{85} of the entire bed material gradation.

Discontinuity / Interparticle Bond Shear Strength Number

The shear strength number, K_d, is calculated differently for rock and granular material. In the case of rock the discontinuity shear strength number is determined as the ratio between two variables representing different characteristics of the surfaces that make up the discontinuity. In the case of granular material, K_d is proportional to the residual angle of friction of the material.

Rock - The discontinuity or inter-particle bond shear strength number (K_d) is the parameter that represents the relative strength of discontinuities in rock and the strength of particle bonding in granular materials. In **rock** it is determined as the ratio between joint wall roughness (J_r) and joint wall alteration (J_a):

Ref 1

$$K_d = \frac{J_r}{J_a} \quad (16)$$

J_r represents the degree of roughness of opposing faces of a rock discontinuity, and J_a represents the degree of alteration of the materials that form the faces of the discontinuity. Alteration relates to amendments of the rock surfaces, for example weathering or the presence of cohesive material between the opposing faces of a joint. Values of J_r and J_a can be found in tables 5 and 6. The values of K_d calculated with the information in these tables change in sympathy with the relative degree of resistance offered by the joints. Increases in resistance are characterized by increases in the value of K_d. The shear strength of a discontinuity is directly proportional to the degree of roughness of opposing joint faces and inversely proportional to the degree of alteration.

Table 5. Joint roughness number (J_r)

Joint Separation	Condition of Joint	Joint Roughness Number
Joints/fissures tight or closing during excavation	Stepped joints/fissures	4.0
	Rough or irregular, undulating	3.0
	Smooth undulating	2.0
	Slickensided undulating	1.5
	Rough or irregular, planar	1.5
	Smooth planar	1.0
	Slickensided planar	0.5
Joints/fissures open and remain open during excavation	Joints/fissures either open or containing relatively soft gouge of sufficient thickness to prevent joint/fissure wall contact upon excavation.	1.0
	Shattered or micro-shattered clays	1.0

Ref 1

Table 6. Joint alteration number (J_a)

Description of Gouge	Joint Alteration Number (J_a) for Joint Separation (mm)		
	1.0[1]	1.0 –5.0[2]	5.0[3]
Tightly healed, hard, non-softening impermeable filling	0.75	-	-
Unaltered joint walls, surface staining only	1.0	-	-
Slightly altered, non-softening, non-cohesive rock mineral or crushed rock filling	2.0	2.0	4.0
Non-softening, slightly clayey non-cohesive filling	3.0	6.0	10.0
Non-softening, strongly over-consolidated clay mineral filling, with or without crushed rock	3.0	6.0**	10.0
Softening or low friction clay mineral coatings and small quantities of swelling clays	4.0	8.0	13.0
Softening moderately over-consolidated clay mineral filling, with or without crushed rock	4.0	8.00**	13.0
Shattered or micro-shattered (swelling) clay gouge, with or without crushed rock	5.0	10.0**	18.0

Note:
[1]Joint walls effectively in contact.
[2]Joint walls come into contact after approximately 100-mm shear.
[3]Joint walls do not come into contact at all upon shear.
**Also applies when crushed rock occurs in clay gouge without rock wall contact.

Joint roughness is described by referring to both large and small-scale characteristics. The large-scale features are known as stepped, undulating or planar; whereas the small-scale features are referred to as rough, smooth or slickensided. Examples of planar and undulating joints are shown in figure 15 and figure 16 respectively. Figure 17 is a schematic presentation of conventional descriptions of joint roughness.

A planar, rough joint indicates that the large-scale feature is planar, but that the joint surfaces are rough. The concept of closed, open and filled joints, terminology used in table 5, is illustrated in figure 15. The value of K_d that is calculated by means of equation (16) is roughly equal to the tangent of the residual angle of friction between the rock surfaces.

flow direction (*FD*), ground slope (*GS*), apparent dip (*AD*) and effective dip (*EF*).

Table 7. Relative ground structure number (J_s)

Dip Direction of Closer Spaced Joint Set (degrees)	Dip Angle of Closer Spaced Joint Set (degrees)	Ratio of Joint Spacing, r			
		1:1	1:2	1:4	1:8
180/0	90	1.14	1.20	1.24	1.26
In direction of stream flow	89	0.78	0.71	0.65	0.61
	85	0.73	0.66	0.61	0.57
	80	0.67	0.60	0.55	0.52
	70	0.56	0.50	0.46	0.43
	60	0.50	0.46	0.42	0.40
	50	0.49	0.46	0.43	0.41
	40	0.53	0.49	0.46	0.45
	30	0.63	0.59	0.55	0.53
	20	0.84	0.77	0.71	0.67
	10	1.25	1.10	0.98	0.90
	5	1.39	1.23	1.09	1.01
	1	1.50	1.33	1.19	1.10
0/180	0	1.14	1.09	1.05	1.02
Against direction of stream flow	-1	0.78	0.85	0.90	0.94
	-5	0.73	0.79	0.84	0.88
	-10	0.67	0.72	0.78	0.81
	-20	0.56	0.62	0.66	0.69
	-30	0.50	0.55	0.58	0.60
	-40	0.49	0.52	0.55	0.57
	-50	0.53	0.56	0.59	0.61
	-60	0.63	0.68	0.71	0.73
	-70	0.84	0.91	0.97	1.01
	-80	1.26	1.41	1.53	1.61
	-85	1.39	1.55	1.69	1.77
	-89	1.50	1.68	1.82	1.91
180/0	-90	1.14	1.20	1.24	1.26

Notes:
1. For intact material take $J_s = 1.0$
2. For values of r greater than 8 take J_s as for r = 8

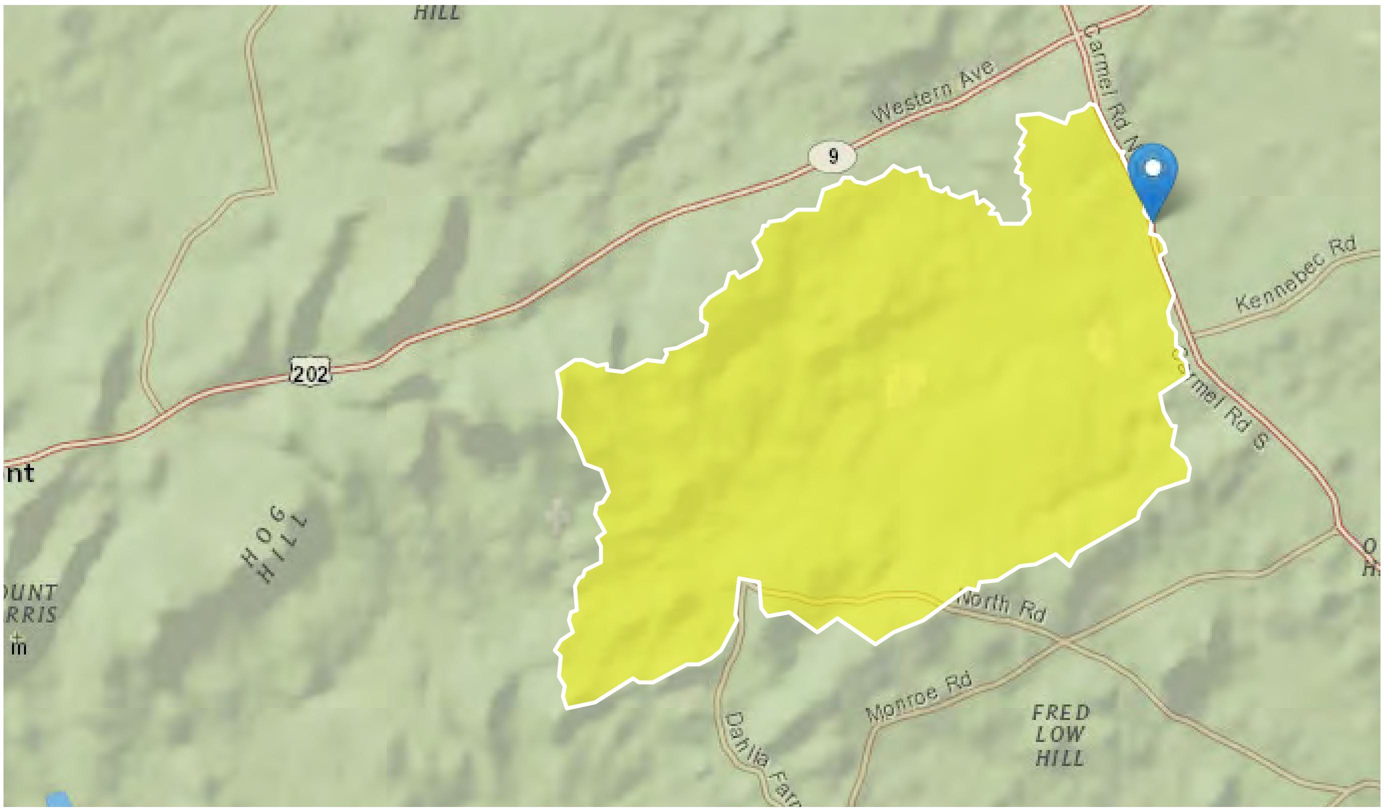
If the flow direction is not in the direction of the true dip, $\left(90^\circ \leq |DD\text{-}FD| \leq 270^\circ\right)$, the effective dip is determined by adding the ground slope to the apparent dip:

$$ED = AD + GS \qquad (20)$$

Ref 2

Estimated Stream Data at Hampden Site Location

Region ID:
ME
Workspace ID:
ME20170612112830040000
Clicked Point (Latitude, Longitude):
44.71211, -68.95101
Time:
2017-06-12 11:29:15 -0400



Basin Characteristics

Parameter Code	Parameter Description	Value	Unit
DRNAREA	Area that drains to a point on a stream	16.2	square miles
STORNWI	Percentage of storage (combined water bodies and wetlands) from the Nationa Wetlands Inventory	9.96	percent
BSLDEM10M	Mean basin slope computed from 10 m DEM	7.98	percent

Ref 2

Peak-Flow Statistics Parameters [100 Percent (16.2 square miles) Statewide Peak Flow Full GT 12sqmi WRI 99 4008]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	16.2	square miles	0.93	1653
STORNWI	Percentage of Storage from NWI	9.96	percent	0.7	26.7

Peak-Flow Statistics Flow Report [100 Percent (16.2 square miles) Statewide Peak Flow Full GT 12sqmi WRI 99 4008]

PIl: Prediction Interval-Lower, PIu: Prediction Interval-Upper, SEp: Standard Error of Prediction, SE: Standard Error (other -- see report)

Statistic	Value	Unit	PIl	PIu	SE	SEp	Equiv. Yrs.
2 Year Peak Flood	490	ft^3/s	273	881	35.1	35.1	1.8
5 Year Peak Flood	762	ft^3/s	420	1380	36.1	36.1	2.5
10 Year Peak Flood	964	ft^3/s	523	1780	36.8	36.8	3.2
25 Year Peak Flood	1240	ft^3/s	654	2340	38.6	38.6	4.1
50 Year Peak Flood	1450	ft^3/s	750	2800	39.9	39.9	4.8
100 Year Peak Flood	1680	ft^3/s	850	3310	41.2	41.2	5.4
500 Year Peak Flood	2240	ft^3/s	1070	4690	44.9	44.9	6.4

Peak-Flow Statistics Citations

Hodgkins, G. A.,1999, Estimating the Magnitude of Peak Flows for Streams in Maine for Selected Recurrence Intervals: U.S. Geological Survey Water-Resources Investigations Report 99-4008, 45 p. (http://me.water.usgs.gov/99-4008.pdf)

Bankfull Statistics Parameters [100 Percent (16.2 square miles) Central and Coastal Bankfull 2004 5042]

Parameter Code	Parameter Name	Value	Units	Min Limit	Max Limit
DRNAREA	Drainage Area	16.2	square miles	2.92	298

Bankfull Statistics Flow Report [100 Percent (16.2 square miles) Central and Coastal Bankfull 2004 5042]

Statistic	Value	Unit
Bankfull Streamflow	96.6	ft^3/s
Bankfull Width	32.6	ft
Bankfull Depth	1.53	ft
Bankfull Area	49.9	ft^2

Ref 2

Bankfull Statistics Citations

Dudley, R.W.,2004, Hydraulic-Geometry Relations for Rivers in Coastal and Central Maine: U.S. Geological Survey Scientific Investigations Report 2004-5042, 30 p (http://pubs.usgs.gov/sir/2004/5042/pdf/sir2004-5042.pdf)

Ref. 3

Highway Bridge Inspection Report

TWIN

RTE #69

over

W.BR.SOUADABSCOOK STR.

Asset Code:	5315
Inspection Date:	06/15/2016
Inspected By:	William Rohman
Inspection Type(s):	Routine

Ref. 3

Inspection Notes

Structure Number: 5315 Town: Hampden

Structure Name: TWIN Inspection Date: 07/27/2015

Structure Notes

1951 24' Single span steel structural plate arch.

Wearing Surface

Deck **NBI Item 58:** N

Superstructure **NBI Item 59:** N

Substructure **NBI Item 60:** N

Culvert **NBI Item 62:** 4

Water depth prevented full entry, see the 2014 U/W Inspection for additional details. Scour under the bridge has created water depth of 4'+ while both up and down stream depths are 1'-2' deep. Footing is exposed 2'-3', per U/W Inspection, with no undermine. Holes at the ends of the culvert with thin steel below waterline where holes could be made with a hammer. Light scaling of steel at the water line with quarter sized rust nodules and moderate pitting below the water line. Large boulder rip-rap at the up stream end is stable with no erosion issues noted.

Ref. 5

Table 4–3 Hardness and unconfined compressive strength of rock materials

Hardness category	Typical range in unconfined compressive strength (MPa)	Strength value selected (MPa)	Field test on sample	Field test on outcrop
Soil*	< 0.60		Use USCS classifications	
Very soft rock or hard, soil-like material	0.60–1.25		Scratched with fingernail. Slight indentation by light blow of point of geologic pick. Requires power tools for excavation. Peels with pocket knife.	
Soft rock	1.25–5.0		Permits denting by moderate pressure of the fingers. Handheld specimen crumbles under firm blows with point of geologic pick.	Easily deformable with finger pressure.
Moderately soft rock	5.0–12.5		Shallow indentations (1–3 mm) by firm blows with point of geologic pick. Peels with difficulty with pocket knife. Resists denting by the fingers, but can be abraded and pierced to a shallow depth by a pencil point. Crumbles by rubbing with fingers.	Crumbles by rubbing with fingers.
Moderately hard rock	12.5–50		Cannot be scraped or peeled with pocket knife. Intact handheld specimen breaks with single blow of geologic hammer. Can be distinctly scratched with 20d common steel nail. Resists a pencil point, but can be scratched and cut with a knife blade.	Unfractured outcrop crumbles under light hammer blows.
Hard rock	50–100		Handheld specimen requires more than one hammer blow to break it. Can be faintly scratched with 20d common steel nail. Resistant to abrasion or cutting by a knife blade, but can be easily dented or broken by light blows of a hammer.	Outcrop withstands a few firm blows before breaking.
Very hard rock	100–250		Specimen breaks only by repeated, heavy blows with geologic hammer. Cannot be scratched with 20d common steel nail.	Outcrop withstands a few heavy ringing hammer blows but will yield large fragments.
Extremely hard rock	> 250		Specimen can only be chipped, not broken by repeated, heavy blows of geologic hammer.	Outcrop resists heavy ringing hammer blows and yields, with difficulty, only dust and small fragments.

Method used to determine consistency or hardness (check one):
Field assessment: _____ Uniaxial lab test: _____ Other: __________ Rebound hammer (ASTM D5873): _____
* See NEH631.03 for consistency and density of soil materials. For very stiff soil, SPT N values = 15 to 30. For very soft rock or hard, soil-like material, SPT N values exceed 30 blows per foot.

Objective: - Estimate the coefficient of friction for the proposed spread footing on rock.
- Provide recommended resistance factors for sliding.

Method: Use MEDOT and AASHTO design methods for sliding resistance.

References: 1. AASHTO. (2014). "LRFD Bridge Design Specifications", Seventh Edition. Page 3-108.
2. MEDOT. (2003). "Bridge Design Guide", includes updates through (2014).

Calculations: Proposed design includes abutment and retaining wall spread footings will be founded directly on bedrock or a subfooting bearing on bedrock.

Coefficient of friction (tanδ) from AASHTO Table 3.11.5.3-1

$\tan\delta =$ **0.60** Due to fractured rock, assume "clean gravel, gravel-sand mixutes, coarse sand" for mass concrete applies.

Strenght limit state resistance factos (φ_s) from MaineDOT Bridge Design Guide - Table 5-3.

$\varphi_s =$ **0.80** For cast-in-place concrete on rock

Based on the Maine DOT Bridge Design Guide Section 5.3.8, the factored (R_r) resistance can be calculated as follows:

$$R_r = \varphi R_n = \varphi_s R_f + \varphi_{ep} R_{ep}$$

where:

R_n = nominal sliding resistance
φ_s = resistance factor for shear resistance between soil and foundation specified in Table 5-3.
R_f = nominal sliding resistance between soil and foundation
φ_{ep} = resistance factor for passive resistance = 0.50
R_{ep} = nominal passive resistance of the soil available throughout the design life of the structure.

Conclusion: Golder recommends ignoring the passive resistance term ($\varphi_{eq}R_{eq}$) due to scour. Threrefore, the factored resistance can be calculated as: $\mathbf{R_r = \phi_s R_f}$

Table 3.11.5.3-1—Friction Angle for Dissimilar Materials (U.S. Department of the Navy, 1982a)

Interface Materials	Friction Angle, δ (degrees)	Coefficient of Friction, tan δ (dim.)
Mass concrete on the following foundation materials:		
• Clean sound rock	35	0.70
• Clean gravel, gravel-sand mixtures, coarse sand	29 to 31	0.55 to 0.60
• Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	24 to 29	0.45 to 0.55
• Clean fine sand, silty or clayey fine to medium sand	19 to 24	0.34 to 0.45
• Fine sandy silt, nonplastic silt	17 to 19	0.31 to 0.34
• Very stiff and hard residual or preconsolidated clay	22 to 26	0.40 to 0.49
• Medium stiff and stiff clay and silty clay	17 to 19	0.31 to 0.34
Masonry on foundation materials has same friction factors.		
Steel sheet piles against the following soils:		
• Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls	22	0.40
• Clean sand, silty sand-gravel mixture, single-size hard rock fill	17	0.31
• Silty sand, gravel or sand mixed with silt or clay	14	0.25
• Fine sandy silt, nonplastic silt	11	0.19
Formed or precast concrete or concrete sheet piling against the following soils:		
• Clean gravel, gravel-sand mixture, well-graded rock fill with spalls	22 to 26	0.40 to 0.49
• Clean sand, silty sand-gravel mixture, single-size hard rock fill	17 to 22	0.31 to 0.40
• Silty sand, gravel or sand mixed with silt or clay	17	0.31
• Fine sandy silt, nonplastic silt	14	0.25
Various structural materials:		
• Masonry on masonry, igneous and metamorphic rocks:		
○ dressed soft rock on dressed soft rock	35	0.70
○ dressed hard rock on dressed soft rock	33	0.65
○ dressed hard rock on dressed hard rock	29	0.55
• Masonry on wood in direction of cross grain	26	0.49
• Steel on steel at sheet pile interlocks	17	0.31

3.11.5.4—Passive Lateral Earth Pressure Coefficient, k_p

For noncohesive soils, values of the coefficient of passive lateral earth pressure may be taken from Figure 3.11.5.4-1 for the case of a sloping or vertical wall with a horizontal backfill or from Figure 3.11.5.4-2 for the case of a vertical wall and sloping backfill. For conditions that deviate from those described in Figures 3.11.5.4-1 and 3.11.5.4-2, the passive pressure may be calculated by using a trial procedure based on wedge theory, e.g., see Terzaghi et al. (1996). When wedge theory is used, the limiting value of the wall friction angle should not be taken larger than one-half the angle of internal friction, ϕ_f.

For cohesive soils, passive pressures may be estimated by:

C3.11.5.4

The movement required to mobilize passive pressure is approximately 10.0 times as large as the movement needed to induce earth pressure to the active values. The movement required to mobilize full passive pressure in loose sand is approximately five percent of the height of the face on which the passive pressure acts. For dense sand, the movement required to mobilize full passive pressure is smaller than five percent of the height of the face on which the passive pressure acts, and five percent represents a conservative estimate of the movement required to mobilize the full passive pressure. For poorly compacted cohesive soils, the movement required to mobilize full passive pressure is larger than five percent of the height of the face on which the pressure acts.

Table 5-3 Resistance Factors for Sliding of Spread Footings at the Strength Limit State

Soil/Condition	Sliding Resistance Factor, φ_s
Precast concrete on sand	0.90
Cast-in-place concrete on sand	0.80
Cast-in-place or precast concrete on clay	0.85
Soil on soil	0.90
Cast-in-place concrete on rock (based on reliability theory analysis of footings on sand)	0.80
Cast-in-place concrete on rock (calibrated to ASD Factor of Safety of 1.5)	0.90

Spread footings should be designed such that the factored resistance to sliding, R_f, is greater than the factored force effects due to the horizontal components of loads. Load factors selected should produce the extreme force effect. The live load surcharge is not included over the heel. Specific guidance for selection of load factors for sliding are provided in LRFD Figure C11.5.6-2.

The nominal sliding resistance between footings and cohesionless soils is taken as:

$$R_f = V \times \tan\delta$$

where:

$\tan\delta = \tan\phi$ for cast-in-place footings on soil
$\tan\delta = 0.80 \tan\phi$ for precast footings on soil
V = total vertical force

The coefficient of friction, $\tan\phi$, for sliding should be as shown in Table 3-3 for the soil type under the footing and LRFD Table 3.11.5.3-1.

The nominal sliding resistance between footings and silt and/or clay soils should be taken to be the lesser of: (1) the undrained shear strength of the silt/clay, or, (2) one-half of the normal stress on soil when the footing is founded on at least 6 inches of compacted granular fill on silt/clay.

For footings on bedrock, the Geotechnical Designer will provide a coefficient of friction for sliding. If smooth bedrock is present at the bearing elevation or if the coefficient of sliding is insufficient to resist lateral forces, the bedrock should be doweled to improve stability. When a footing is doweled into rock, the dowels should be #9 reinforcing bars or larger and be embedded into the footings and bedrock by depths determined by the Designer. The spacing of

Golder Associates	**SUBJECT:** Siesmic Site Class **Project Number:** 1772224 **Project Name:** Hampden, ME - Route 69 Culvert Replacement **Prepared by:** LLM **Checked by:** CJS **Date:** 5/31/2017 **Reviewed by:** MCM

Objective: Determine the siesmic site class at Twin Bridge culvert in Hampden, ME.

Method: Follow the procedure outlined in AASHTO Table C3.10.3.1-1

References: 1. AASHTO. (2014). "LRFD Bridge Design Specifications", Seventh Edition. Pages 3-88 to 3-90.
2. Borehole logs from Golder field explorations completed May 15 and 16, 2017.

Calculations: Determine the average N value for each of the layer of the soil profile.

Boring	Layer	Depth	N Value
BB-HAMP-101	Fill	1.00' - 3.00'	27
		3.00' - 5.00'	52
	Alluvium	8.00' - 10.00'	10
		10.00' - 12.00'	6
		12.00' - 14.00'	7
	Glacial Till	15.00' - 17.00'	50
BB-HAMP-102	Fill	1.00' - 3.00'	33
		3.00' - 5.00'	14
	Alluvium	7.00' - 9.00'	12
		9.00' - 11.00'	8
		11.00' - 13.00'	13
	Glacial Till	15.00' - 17.00'	38

Average N Values By Layer

Layer	Average Depth		Average N Value
	Top	Bottom	
Fill	0.00	7.50	32
Alluvium	7.50	15.00	9
Glacail Till	15.00	18.00	44

Determine the average N for the top 100 ft using the following calculation:

$$\bar{N} = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{N_i}}$$

N_i = Standard Penetration Test blow count for a layer (not exceeding 100 blows/ft in the above expression)

d_{Fill} = 7.50 N_{Fill} = 32

$d_{Alluvium}$ = 7.50 $N_{Alluvium}$ = 9

d_{Till} = 3.00 N_{Till} = 44

d_{Rock} = 82.00 N_{Rock} = 100.00 *

*Note: Where refusal is met for a rock layer, N_i should be taken as 100 blows/ft.

$\overline{N}$ = 52

Conclusion: The seismic site class is C beacause the $\overline{N}$ > 50 blows/ft. Classification based on definitions from Table C3.10.3.1-1.

Table 3.10.3.1-1—Site Class Definitions

Site Class	Soil Type and Profile
A	Hard rock with measured shear wave velocity, $\bar{v}_s > 5{,}000$ ft/s
B	Rock with 2,500 ft/sec < $\bar{v}_s$ < 5,000 ft/s
C	Very dense soil and soil rock with 1,200 ft/sec < $\bar{v}_s$ < 2,500 ft/s, or with either $\bar{N} > 50$ blows/ft, or $\bar{s}_u > 2.0$ ksf
D	Stiff soil with 600 ft/s < $\bar{v}_s$ < 1,200 ft/s, or with either $15 < \bar{N} < 50$ blows/ft, or $1.0 < \bar{s}_u < 2.0$ ksf
E	Soil profile with $\bar{v}_s$ < 600 ft/s or with either $\bar{N}$ < 15 blows/ft or $\bar{s}_u$ < 1.0 ksf, or any profile with more than 10 ft of soft clay defined as soil with $PI > 20$, $w > 40$ percent and $\bar{s}_u$ < 0.5 ksf
F	Soils requiring site-specific evaluations, such as: • Peats or highly organic clays ($H > 10$ ft of peat or highly organic clay where H = thickness of soil) • Very high plasticity clays ($H > 25$ ft with $PI > 75$) • Very thick soft/medium stiff clays ($H > 120$ ft)

Exceptions: Where the soil properties are not known in sufficient detail to determine the site class, a site investigation shall be undertaken sufficient to determine the site class. Site classes E or F should not be assumed unless the authority having jurisdiction determines that site classes E or F could be present at the site or in the event that site classes E or F are established by geotechnical data.

where:

$\bar{v}_s$ = average shear wave velocity for the upper 100 ft of the soil profile

$\bar{N}$ = average Standard Penetration Test (SPT) blow count (blows/ft) (ASTM D1586) for the upper 100 ft of the soil profile

$\bar{s}_u$ = average undrained shear strength in ksf (ASTM D2166 or ASTM D2850) for the upper 100 ft of the soil profile

PI = plasticity index (ASTM D4318)

w = moisture content (ASTM D2216)

Ref 1

Table C3.10.3.1-1—Steps for Site Classification

Step	Description
1	Check for the three categories of Site Class F in Table 3.10.3.1-1 requiring site-specific evaluation. If the site corresponds to any of these categories, classify the site as Site Class F and conduct a site-specific evaluation.
2	Check for existence of a soft layer with total thickness > 10 ft, where soft layer is defined by $s_u < 0.5$ ksf, $w > 40\%$, and $PI > 20$. If these criteria are met, classify site as Site Class E.
3	Categorize the site into one of the site classes in Table 3.10.3.1-1 using one of the following three methods to calculate: • $\bar{v}_s$ for the top 100 ft ($\bar{v}_s$ method) • $\bar{N}$ for the top 100 ft ($\bar{N}$ method) • $\bar{N}_{ch}$ for cohesionless soil layers ($PI < 20$) in the top 100 ft and $\bar{s}_u$ for cohesive soil layers ($PI > 20$) in the top 100 ft ($\bar{s}_u$ method) To make these calculations, the soil profile is subdivided into n distinct soil and rock layers, and in the methods below the symbol *i* refers to any one of these layers from 1 to *n*. **Method A: $\bar{v}_s$ method** The average $\bar{v}_s$ for the top 100 ft is determined as: $\bar{v}_s = \dfrac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \dfrac{d_i}{v_{si}}}$ where: $\sum_{i=1}^{n} d_i = 100$ ft v_{si} = shear wave velocity in ft/s of a layer d_i = thickness of a layer between 0 and 100 ft **Method B: $\bar{N}$ method** The average $\bar{N}$ for the top 100 ft shall be determined as: $\bar{N} = \dfrac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \dfrac{d_i}{N_i}}$ where: N_i = Standard Penetration Test blow count of a layer (not to exceed 100 blows/ft in the above expression)

Note: When using Method B, $\bar{N}$ values are for cohesionless soils and cohesive soil and rock layers within the upper 100 ft. Where refusal is met for a rock layer, N_i should be taken as 100 blows/ft.

Ref 1

Table C3.10.3.1-1 (continued)—Steps for Site Classification

Method C: $\bar{s}_u$ method

The average $\bar{N}_{ch}$ for cohesionless soil layers in the top 100 ft is determined as:

$$\bar{N}_{ch} = \frac{d_s}{\sum_{i=1}^{m} \frac{d_i}{N_{chi}}}$$

in which:

$$\sum_{i=1}^{m} d_i = d_s,$$

where:

m = number of cohesionless soil layers in the top 100 ft
N_{chi} = blow count for a cohesionless soil layer (not to exceed 100 blows/ft in the above expression)
d_s = total thickness of cohesionless soil layers in the top 100 ft

The average $\bar{s}_u$ for cohesive soil layers in the top 100 ft is determined as:

$$\bar{s}_u = \frac{d_c}{\sum_{i=1}^{k} \frac{d_i}{s_{ui}}}$$

in which:

$$\sum_{i=1}^{k} d_i = d_c,$$

where:

k = number of cohesive soil layers in the top 100 ft
s_{ui} = undrained shear strength for a cohesive soil layer (not to exceed 5.0 ksf in the above expression)
d_c = total thickness of cohesive soil layers in the top 100 ft

Note: When using Method C, if the site class resulting from $\bar{N}_{ch}$ and $\bar{s}_u$ differ, select the site class that gives the highest site factors and design spectral response in the period range of interest. For example, if $\bar{N}_{ch}$ was equal to 20 blows/ft and $\bar{s}_u$ was equal to 0.8 ksf, the site would classify as D or E in accordance with Method C and the site class definitions of Table 3.10.3.1-1. In this example, for relatively low response spectral acceleration and for long-period motions, Table 3.10.3.2-3 indicates that the site factors are highest for Site Class E. However, for relatively high short-period spectral acceleration ($Ss > 0.75$), short period site factors, Fa, are higher for Site Class D.

Golder Associates	**SUBJECT:** Frost Depth Calculation **Project Number:** 1772224 **Project Name:** Hampden, ME - Route 69 Culvert Replacement **Prepared by:** LLM **Checked by:** CJS **Date:** 5/31/2017 **Reviewed by:** MCM

Objective: Determine the frost depth penetration depth in Hampden, ME.

Method: Use MEDOT design methods for determining frost depth.

References:
1. Moisture Contents of Laboratory Samples emailed by Joseph Tomei (GeoTesting Express) on 6/7/2017 at
2. MEDOT. (2003). "Bridge Design Guide", includes updates through (2014). Pages 5-3 to 5-6.

Calculations: The proposed is spread footings bearing directly on bedrock, therefore, frost depth is not a concern. However, to evaluate potential frost depth in soil in the area, Golder followed the following proceedure. Use Figure 5-1 (MEDOT 2003) to determine the design freezing index in Hampden, ME.

Design Freezing Index: 1700 degree days

Determine soil water content (WC) from GeoTesting Express laboratory data (Reference 1).

Boring	Sample	Depth (ft.)	WC (%)
BB-HAMP-101	2D	3-5	6.4
BB-HAMP-101	5D	12-14	11.5
BB-HAMP-101	6D	15-17	8.4
BB-HAMP-102	5D	11-13	9.4
BB-HAMP-102	6D	15-17	7.7

Use table 5-1 (Reference 2, MEDOT 2003) with design freezing index and soil water content. Footings for the arch will be below the water table/river level. Assume the WC for these layers are 10% (above the water table) and 30% (below water table) for the purpose of using Table 5-1. The range of potential frost depths in the area is shown below.

Coarse Grained Soils (Alluvium Soils)

Index: 1700	Water Content
	10%
Frost Penetation (in)	87.5

Fine Grained Soils (Glacial Till)

Index: 1700	Water Content
	30%
Frost Penetation (in)	48.4

Conclusion: For the existing design conditions at the proposed culvert location Golder recommends a design frost depth of 68" should be used based on an average of the coarse grained and fine grained values.

Ref 1

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

Ref 1

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.

Ref 1

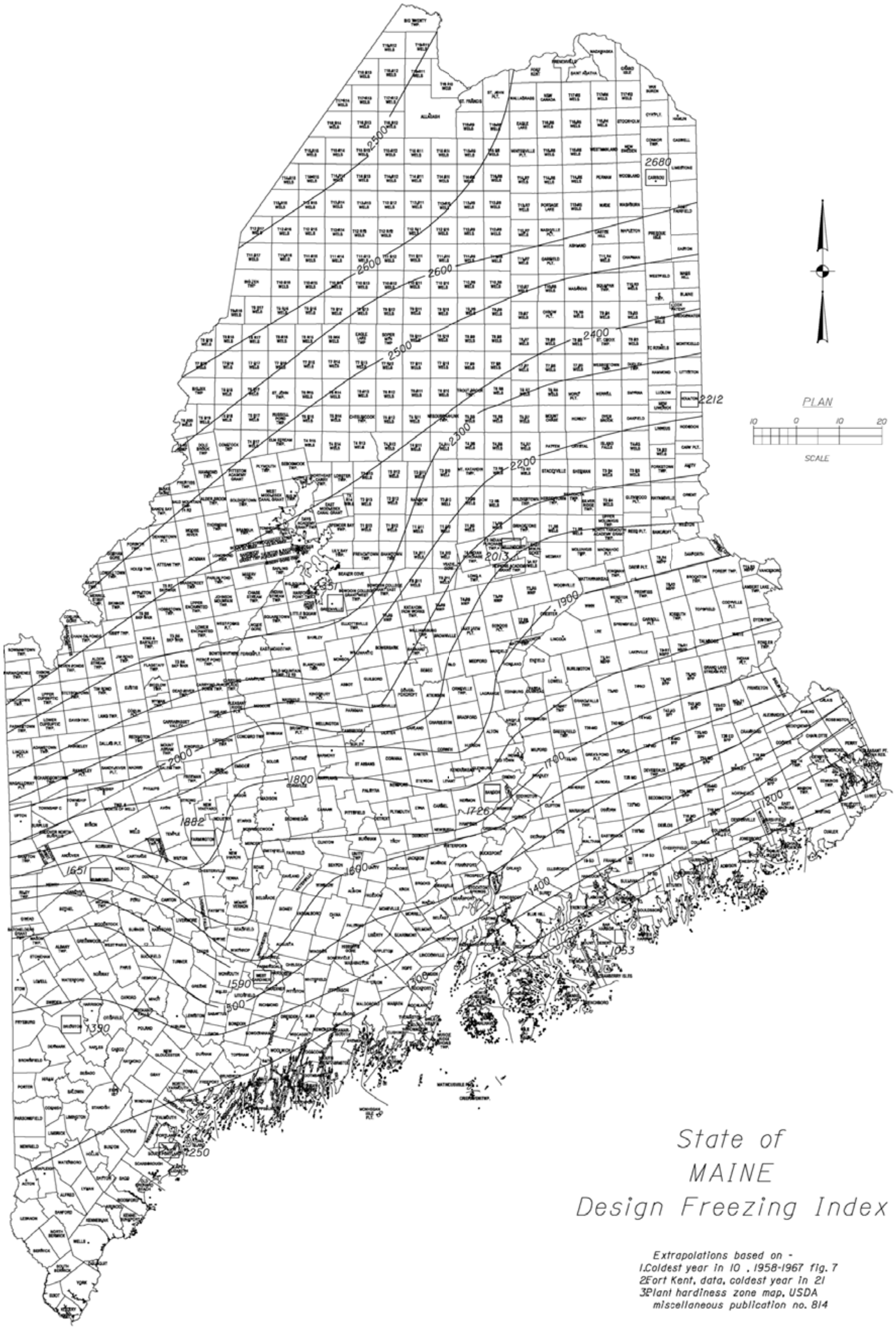
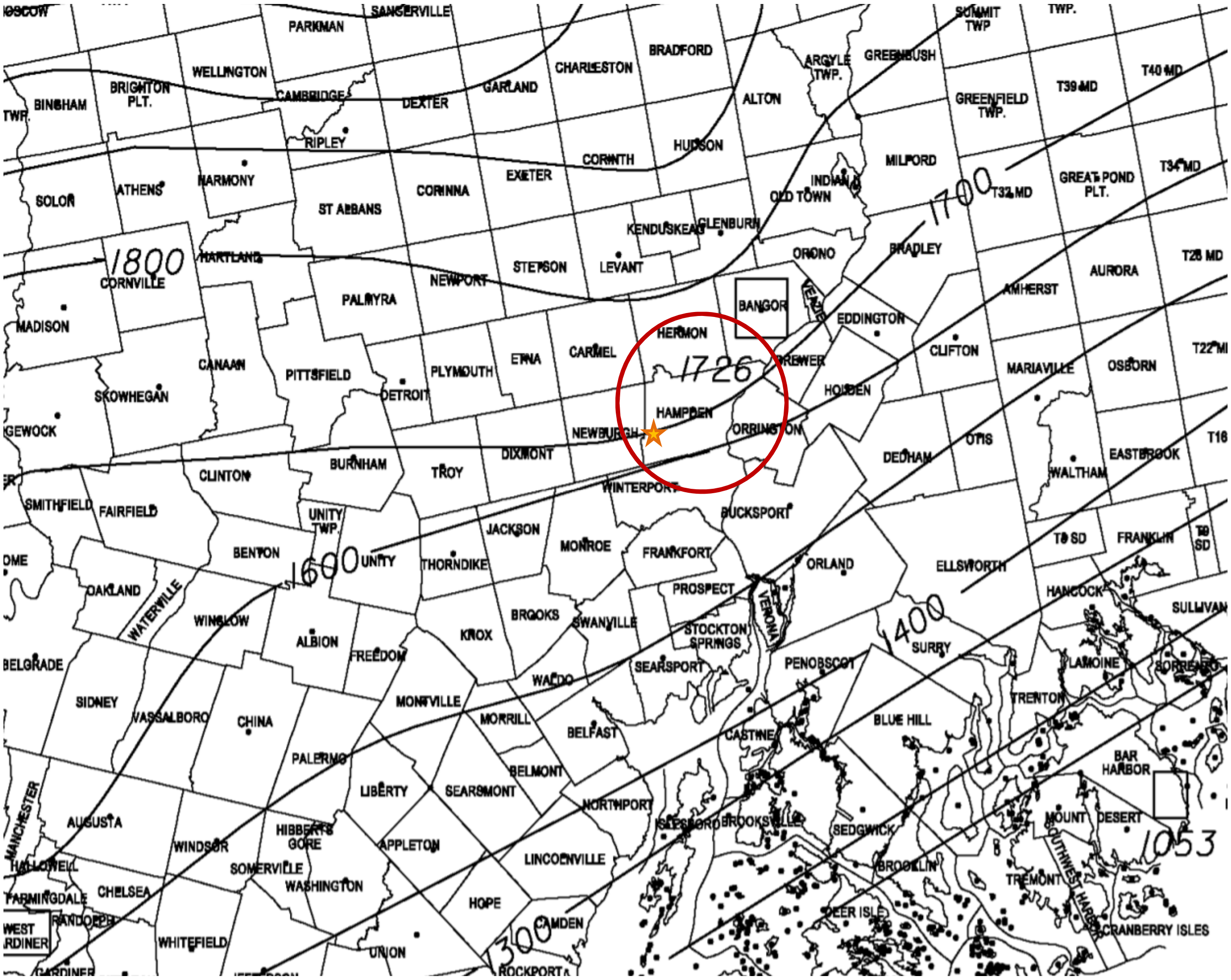


Figure 5-1 Maine Design Freezing Index Map

Ref 1


PARKMAN
SANGERVILLE
WELLINGTON
BRIGHTON PLT.
BINGHAM
CAMBRIDGE
DEXTER
GARLAND
CHARLESTON
BRADFORD
ALTON
ARGYLE TWP.
GREENBUSH
SUMMIT TWP
TWP.
GREENFIELD TWP.
T39 MD
T40 MD
RIPLEY
CORINTH
HUDSON
MILFORD
HARMONY
ATHENS
SOLON
ST ALBANS
CORINNA
EXETER
INDIAN
OLD TOWN
1700
T32 MD
GREAT POND PLT.
T34 MD
KENDUSKEAG
GLENBURN
ORONO
BRADLEY
1800
HARTLAND
CORNVILLE
PALMYRA
NEWPORT
STETSON
LEVANT
BANGOR
EDDINGTON
AMHERST
AURORA
T28 MD
MADISON
HERMON
CLIFTON
CARMEL
ETNA
PLYMOUTH
PITTSFIELD
CANAAN
1726
BREWER
MARIAVILLE
OSBORN
T22 MD
SKOWHEGAN
DETROIT
HOLDEN
HAMPDEN
NEWBURGH
ORRINGTON
DIXMONT
TROY
BURNHAM
CLINTON
DEDHAM
OTIS
WALTHAM
EASTBROOK
T16
WINTERPORT
SMITHFIELD
FAIRFIELD
UNITY TWP.
JACKSON
BUCKSPORT
MONROE
FRANKFORT
BENTON
1600
UNITY
THORNDIKE
ORLAND
ELLSWORTH
T9 SD
FRANKLIN
T9 SD
OAKLAND
PROSPECT
WATERVILLE
WINSLOW
BROOKS
SWANVILLE
STOCKTON SPRINGS
VERONA
1400
HANCOCK
SULLIVAN
KNOX
ALBION
FREEDOM
SURRY
PENOBSCOT
LAMOINE
SORRENTO
BELGRADE
SEARSPORT
WALDO
SIDNEY
VASSALBORO
CHINA
MONTVILLE
MORRILL
BELFAST
CASTINE
BLUE HILL
TRENTON
PALERMO
BELMONT
BAR HARBOR
LIBERTY
SEARSMONT
NORTHPORT
ISLESBORO
BROOKSVILLE
SEDGWICK
MOUNT DESERT
AUGUSTA
WINDSOR
HIBBERTS GORE
APPLETON
1053
HALLOWELL
SOMERVILLE
LINCOLNVILLE
BROOKLIN
TREMONT
SOUTHWEST HARBOR
FARMINGDALE
CHELSEA
WASHINGTON
HOPE
DEER ISLE
RANDOLPH
WEST GARDINER
CAMDEN
CRANBERRY ISLES
WHITEFIELD
UNION
1300
ROCKPORT
GARDINER

Ref 2

From: Tomei, Joseph D [mailto:jdt@geotesting.com]
Sent: Wednesday, June 07, 2017 10:04 AM
To: Stuart, Cameron <Cameron_Stuart@golder.com>
Subject: Route 69 Culvert

Moistures below. Let me know if you need anything else.

BB-HAMP-101 3-5 ft: 6.4%
BB-HAMP-101 12-14 ft: 11.5%
BB-HAMP-101 15-17 ft: 8.4%
BB-HAMP-102 11-13 ft: 9.4%
BB-HAMP-102 15-17 ft: 7.7%

Joe Tomei

Laboratory Manager

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jtomei@geotesting.com

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Golder Associates	**SUBJECT:** Bearing Resistance on Rock **Project Number:** 1772224 **Project Name:** Hampden, ME - Route 69 Culvert Replacement **Prepared by:** LLM **Checked by:** CJS **Date:** 5/31/2017 **Reviewed by:** MCM

Objective: Estimate the bearing resistance on rock for the Route 69 culvert replacement in Hampden, ME.

Method: Following procedure outlined in NCHRP Report 651 on the Carter Kulhawy method of using the Hoek-Brown strength criterion to evaluate bearing capacity on rock.

References:
1. Carter & Kulhawy. (1988) from NCHRP Report 651, equation 82b.
2. USDA, NRCS. (2012). National Engineering Handbook, Part 631 Geology, Chapter 4: Engineering Classification of Rock Materials.
3. Hoek, Brown (1988), The Hoek-Brown Failure-Criterion - a 1988 Update.
4. MEDOT. (2014). Bridge Design Guide Update 2014.
5. Bieniawski. (1974) from Wyllie, D.C., 1999. Foundations on Rock, 2nd Edition.
6. Golder RMR calculation dated June 6, 2017.
7. GeoTesting Express Uniaxial Stenght Test Lab Data, June 5, 2017.

Calculations: The bearing capacity of a strip footing is evaluated using the following equation (Carter & Kulhawy, 1988):

$$q_{ult} = \left(\sqrt{s} + \left(m\sqrt{s} + s \right)^{0.5} \right) q_u \qquad (82b)$$

Where s and m are empirically determined strength parameters for the rock mass and q_u is the uniaxial compressive strength of the intact rock.

Estimate q_u for bedrock observed in BB-HAMP-101 using Table 4.3 from the National Engineering Handbook (Ref. 2). No laboratory testing was perfomed on rock core from BB-HAMP-101.

Boring	Rock Type	Rock Hardness Category	Uniaxial Compressive Strength			
			MPa		psi	
			Low	High	Low	High
BB-HAMP-101	Phyllite	Moderately Hard	12.5	50	1813	7252
		Hard	50	100	7252	14504

Two rock samples from Run 1 of BB-HAMP-101 were broken with a geologists hammer. One sample, near the top of the run (approximately 19.5ft bgs), required more than one hammer blow to break the core sample (hard rock, Table 4-3 Ref. 2) and a second sample, near the bottom of the run (approximately 22ft bgs) , required only one geologists hammer blow to break the core sample (moderately hard rock, Table 4-3 Ref. 2). Based on this assume that the rock is borderline "moderately hard" to "hard."

q_u at BB-HAMP-102 is based on USC laboratory data (Ref. 7). Rock hardness category is assigned using Table 4.3 from the National Engineering Handbook (Ref. 2).

Boring	Rock Type	Rock Hardness Category	UCS - Lab Data (psi)
BB-HAMP-102	Phyllite	Hard	11473

Based on laboratory data and published strength for moderately hard to hard rock, assume qu = 7300psi for bedrock present at site.

Representative Rock Mass q_u (psi) = **7300**

Calculate the rock mass rating (RMR) from the rock core.

Boring	Run	Rock Type	RMR*	Average RMR
BB-HAMP-101	Run 1	Phyllite	44	40.5
	Run 2		37	
BB-HAMP-102	Run 1	Phyllite	52	53
	Run 2		54	

*RMR calculated in Reference 6 and presented in Table 3 Summary of Rock Core Quality.

Representative RMR = **46.75**

Calculate m ,s using the 1988 method for the Hoek-Brown failure criterion, assuming a disturbed rock mass.

$$\frac{m}{m_i} = \exp\left(\frac{\text{RMR} - 100}{14}\right)$$

$$s = \exp\left(\frac{\text{RMR} - 100}{6}\right)$$

Where m_i is the m for intact rock.

From Hoek-Brown Table 1:

Rock Type	Lithified Argilaceous Rocks
	Fair Quality Rock Mass

m_i = **0.183**

Therefore:

m = **0.004**

s = **0.00014**

The ultimate bearing capacity (q_{ult}) is:

q_{ult} = **186.44** psi

26.85 ksf

Calculate factored bearing resistance:

$$q_a = q_u \varphi_b$$

Where q_a is the allowable bearing capacity and φ_b is the bearing resistance factor.

φ_b **0.45** (from table 5-2 from the MEDOT Bridge Design Guide)

q_a = **83.90** psi

12.08 ksf

Conclusion: The allowable bearing capcity is 12.08 ksf.

Ref 1

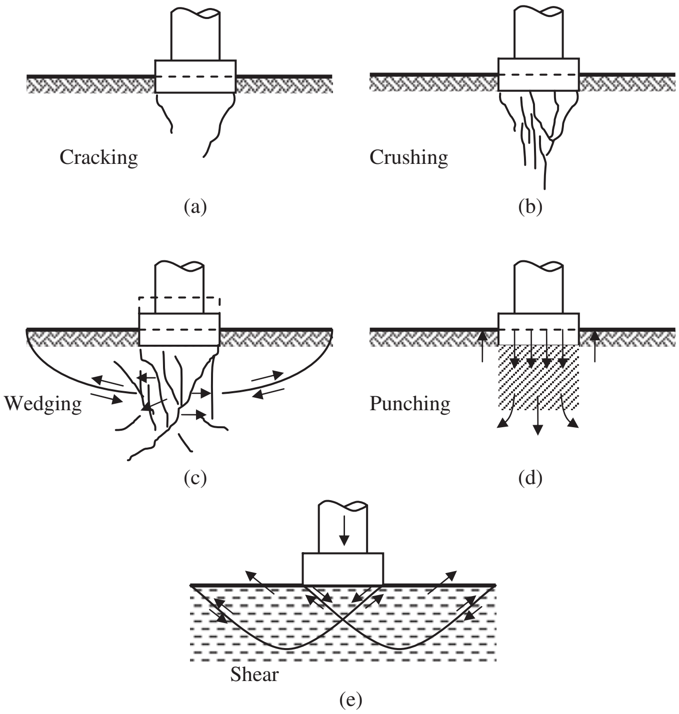


Figure 36. Modes of failure of a footing on rock including development of failure through crack propagation and crushing beneath the footing (a-c), punching through collapse of voids (d), and shear failure (e) (based on Goodman, 1989).

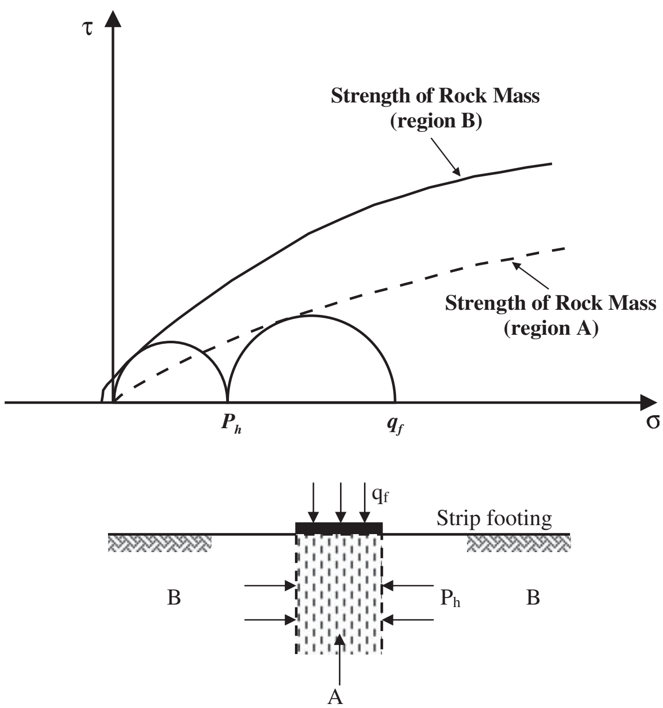


Figure 37. Analysis of bearing capacity on rock (based on Goodman, 1989).

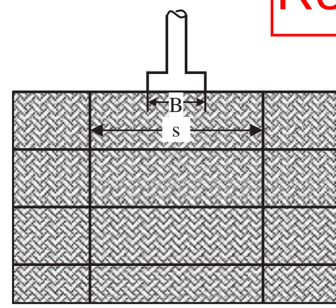


Figure 38. Footing on rock with open, vertical joints (based on Goodman, 1989).

Comparing the results of Goodman's (1989) computations with Equations 79 and 80 shows that open joints reduce the bearing capacity only when the ratio S/B is in the range from 1 to 5. The bearing capacity of footings on rock with open joints increases with increasing ϕ_f for any of the S/B ratios ranging from 1 to 5.

1.7.6 Carter and Kulhawy (1988)

Carter and Kulhawy (1988) suggested that the Hoek and Brown strength criterion for jointed rock masses (Hoek and Brown, 1980, see also Section 1.8.2.4) can be used in the evaluation of bearing capacity. The curved strength envelope for jointed rock mass can be expressed as

$$\sigma_1 = \sigma_3 + \left(mq_u\sigma_3 + sq_u^2\right)^{0.5} \tag{81}$$

where

σ_1 = major principal effective stress,
σ_3 = minor principal effective stress,
q_u = uniaxial compressive strength of the intact rock.
s and m = empirically determined strength parameters for the rock mass, which are to some degree analogous to c and ϕ_f of the Mohr-Coulomb failure criterion.

Carter and Kulhawy (1988) suggested that an analysis of the bearing capacity of a rock mass obeying this criterion can be made using the same approximate technique as used in the Bell (1915) solution. The details of this approach are described in Figure 39. A lower bound to the failure load was calculated by finding a stress field that satisfies both equilibrium and the failure criterion. For a strip footing, the rock mass beneath the foundation may be divided into two zones with homogeneous stress conditions at failure throughout each, as shown in Figure 39. The vertical stress in Zone I is assumed to be zero, while the horizontal stress is equal to the uniaxial compressive strength of the rock mass, given by Equation 81 as $s^{0.5}q_u$. For equilibrium, continuity of the horizontal stress

Ref 1

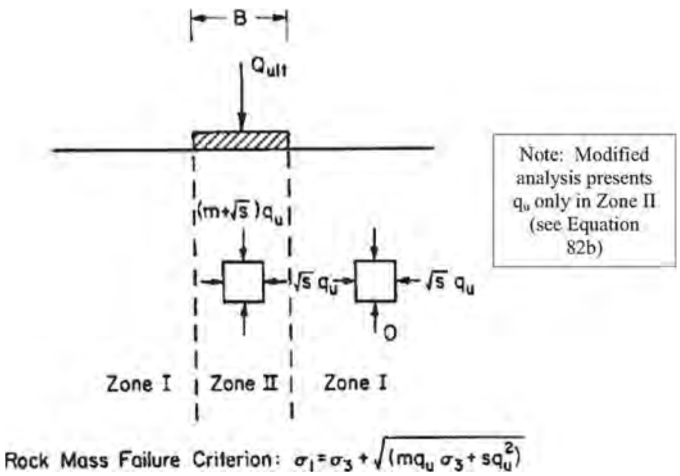


Figure 39. Lower bound solution for bearing capacity (Carter and Kulhawy, 1988).

across the interface must be maintained and therefore the bearing capacity of the strip footing may be evaluated from Equation 81 (with $\sigma_3 = s^{0.5}q_u$) as

$$q_{ult} = \left(m + \sqrt{s}\right)q_u \quad (82a)$$

In an errata to Carter and Kulhawy (1988), Equation (82a) was modified to the following:

$$q_{ult} = \left(\sqrt{s} + \left(m\sqrt{s} + s\right)^{0.5}\right)q_u \quad (82b)$$

A similar approach to the bearing capacity analysis of a strip footing was proposed by Carter and Kulhawy (1988) to be used for a circular foundation with an interface between the two zones that was a cylindrical surface of the same diameter as the foundation. In this axisymmetric case, the radial stress transmitted across the cylindrical surface at the point of collapse of the foundation may be greater than $q_u\sqrt{s}$, without necessarily violating either radial equilibrium or the failure criterion. However, because of the uncertainty of this value, the radial stress at the interface is also assumed to be $q_u\sqrt{s}$ for the case of a circular foundation. Therefore, the predicted (lower bound) bearing capacity is given by Equations 82a and 82b. The m and s constants are determined by the rock type and the conditions of the rock mass, and selecting an appropriate category is easier if either the Rock Mass Rating (RMR) system or the Geological Strength Index (GSI) classification data are available as outlined below. Both bearing capacity formulations expressed in Equations 82a and 82b were investigated in this study.

1.8 Rock Classification and Properties

1.8.1 Overview

A rock mass comprises blocks of intact rock that are separated by discontinuities such as cleavage, bedding planes, joints, and faults. Table 8 provides a summary of rock mass discontinuity definitions and characteristics. These naturally formed discontinuities create weakness surfaces within the rock mass, thereby reducing the material strength. As previously discussed, the influence of the discontinuities upon the material strength depends upon the scale of the foundation relative to the position and frequency of the discontinuities (Canadian Foundation Geotechnical Society, 2006).

This section provides a short review of rock mass classification/characterization systems and rock properties that are relevant to the methods selected for bearing capacity evaluation. Methods allowing engineering classification of rock mass are reviewed including the Rock Mass index (RM*i*) system, RMR system and the Hoek-Brown GSI.

1.8.2 Engineering Rock Mass Classification

1.8.2.1 Classification Methods

A number of classification systems have been developed to provide the basis for engineering characterization of rock masses. A comprehensive overview of this subject is provided by Hoek et al. (1995). Most of the classification systems incorporating various parameters were derived from civil engineering case histories in which all components of the engineering geological parameters of the rock mass were considered (Wickham et al., 1972; Bieniawski, 1973, 1979, 1989; Barton et al., 1974). More recently, the systems have been modified to account for the conditions affecting rock mass stability in underground mining. While no single classification system has been developed for or applied to foundation design, the type of information collected for the two more common civil engineering classification schemes—the Q system (Barton et al., 1974), used in tunnel design, and RMR (Bieniawski, 1989), used in tunnel and foundation design—are often considered. These techniques have been applied to empirical design situations, where previous experience greatly affects the design of the excavation in the rock mass. Table 9 outlines the many classification systems and their uses. Detailed descriptions of the different systems and the engineering properties associated with them are beyond the scope of this work and are restricted to the methods relevant to the current research.

The two most commonly used rock mass classification systems today are RMR, developed by Bieniawski (1973) and

Table 4–3 Hardness and unconfined compressive strength of rock materials

Hardness category	Typical range in unconfined compressive strength (MPa)	Strength value selected (MPa)	Field test on sample	Field test on outcrop
Soil*	< 0.60		Use USCS classifications	
Very soft rock or hard, soil-like material	0.60–1.25		Scratched with fingernail. Slight indentation by light blow of point of geologic pick. Requires power tools for excavation. Peels with pocket knife.	
Soft rock	1.25–5.0		Permits denting by moderate pressure of the fingers. Handheld specimen crumbles under firm blows with point of geologic pick.	Easily deformable with finger pressure.
Moderately soft rock	5.0–12.5		Shallow indentations (1–3 mm) by firm blows with point of geologic pick. Peels with difficulty with pocket knife. Resists denting by the fingers, but can be abraded and pierced to a shallow depth by a pencil point. Crumbles by rubbing with fingers.	Crumbles by rubbing with fingers.
Moderately hard rock	12.5–50		Cannot be scraped or peeled with pocket knife. Intact handheld specimen breaks with single blow of geologic hammer. Can be distinctly scratched with 20d common steel nail. Resists a pencil point, but can be scratched and cut with a knife blade.	Unfractured outcrop crumbles under light hammer blows.
Hard rock	50–100		Handheld specimen requires more than one hammer blow to break it. Can be faintly scratched with 20d common steel nail. Resistant to abrasion or cutting by a knife blade, but can be easily dented or broken by light blows of a hammer.	Outcrop withstands a few firm blows before breaking.
Very hard rock	100–250		Specimen breaks only by repeated, heavy blows with geologic hammer. Cannot be scratched with 20d common steel nail.	Outcrop withstands a few heavy ringing hammer blows but will yield large fragments.
Extremely hard rock	> 250		Specimen can only be chipped, not broken by repeated, heavy blows of geologic hammer.	Outcrop resists heavy ringing hammer blows and yields, with difficulty, only dust and small fragments.

Method used to determine consistency or hardness (check one):
Field assessment: _____ Uniaxial lab test: _____ Other: __________ Rebound hammer (ASTM D5873): _____

* See NEH631.03 for consistency and density of soil materials. For very stiff soil, SPT N values = 15 to 30. For very soft rock or hard, soil-like material, SPT N values exceed 30 blows per foot.

Ref 3

order to permit construction of the models. Consequently, our ability to predict the strength of jointed rock masses on the basis of direct tests or of model studies is severely limited.

In searching for a solution to this problem in order to provide a basis for the design of underground excavations in rock, Hoek and Brown (1980a) felt that some attempt had to be made to link the constants m and s of their criterion to measurements or observations which could be carried out by any competent geologist in the field. Recognizing that the characteristics of the rock mass which control its strength and deformation behaviour are similar to the characteristics which had been adopted by Bieniawski (1974) and by Barton, Lien and Lunde (1974) for their rock mass classifications, Hoek and Brown (1980a) proposed that these rock mass classifications could be used for estimating the material constants m and s.

Because of the lack of suitable methods for estimating the strength of rock masses, the first table relating rock mass classifications to material properties published by Hoek and Brown (1980a) was widely accepted by the geotechnical community and has been used on a large number of projects. Experience gained from these applications showed that the estimated rock mass strengths were reasonable when used for slope stability studies in which the rock mass is usually disturbed and loosened by relaxation due to excavation of the slope. However, the estimated rock mass strengths generally appeared to be too low in applications involving underground excavations where the confining stresses do not permit the same degree of loosening as would occur in a slope.

In order to incorporate the lessons learned from practical applications, Brown and Hoek (1988) proposed a revised set of relationships between the rock mass rating (RMR) from Bieniawski's (1974) rock mass classification and the constants m and s. Following Priest and Brown (1983), the relationships were presented in the form of the following equations:

Disturbed rock masses :

$$\frac{m}{m_i} = \exp\left(\frac{\text{RMR} - 100}{14}\right) \qquad (18)$$

$$s = \exp\left(\frac{\text{RMR} - 100}{6}\right) \qquad (19)$$

Undisturbed or interlocking rock masses:

$$\frac{m}{m_i} = \exp\left(\frac{\text{RMR} - 100}{28}\right) \qquad (20)$$

$$s = \exp\left(\frac{\text{RMR} - 100}{9}\right) \qquad (21)$$

where

m and s are the rock mass constants and
m_i is the value of m for the *intact* rock.

Equations 18 to 21 have been used to construct Table 1 which shows the approximate relationship between rock mass quality and the Hoek-Brown material constants. Note that the value of the Tunnelling Quality Index Q from the NGI rock mass classification by Barton, Lien and Lunde (1974) has been calculated from the relationship proposed by Bieniawski (1976) :

$$\text{RMR} = 9\,\text{Log}_e\,\text{Q} + 44 \qquad (22)$$

Limitations on using failure criterion

Figure 1 illustrates a jointed rock mass in to which a tunnel has been mined. The circles adjacent to the right hand wall of the tunnel enclose different rock mass volumes and the comments on the right hand side of the drawing indicate situations to which the Hoek-Brown failure criterion can be applied.

When the volume of rock under consideration is small enough that it does not contain any structural discontinuities, equation 1 can be applied, using the m and s values for *intact* rock. This condition would apply to small scale specimens which has been extracted for laboratory testing or to the analysis of concentrated forces such as those which may be exerted by an individual pick on a tunnel boring machine cutter.

When the volume of rock being considered is such that only a few structural discontinuities are contained in this volume, the Hoek-Brown criterion should not be used. The behaviour of this rock is likely to be highly anisotropic and the Hoek-Brown failure criterion, which is only applicable to isotropic rock, will give erroneous results.

Ref 3

Table 1 : Approximate relationship between rock mass quality and material constants

Disturbed rock mass m and s values — *undisturbed rock mass m and s values*

EMPIRICAL FAILURE CRITERION $\sigma_1' = \sigma_3' + \sqrt{m\sigma_c\sigma_3' + s\sigma_c^2}$ σ_1' = major principal effective stress σ_3' = minor principal effective stress σ_c = uniaxial compressive strength of intact rock, and m and s are empirical constants.		CARBONATE ROCKS WITH WELL DEVELOPED CRYSTAL CLEAVAGE *dolomite, limestone and marble*	LITHIFIED ARGILLACEOUS ROCKS *mudstone, siltstone, shale and slate (normal to cleavage)*	ARENACEOUS ROCKS WITH STRONG CRYSTALS AND POORLY DEVELOPED CRYSTAL CLEAVAGE *sandstone and quartzite*	FINE GRAINED POLYMINERALLIC IGNEOUS CRYSTALLINE ROCKS *andesite, dolerite, diabase and rhyolite*	COARSE GRAINED POLYMINERALLIC IGNEOUS & METAMORPHIC CRYSTALLINE ROCKS – *amphibolite, gabbro gneiss, granite, norite, quartz-diorite*
INTACT ROCK SAMPLES *Laboratory size specimens free from discontinuities* CSIR rating: RMR = 100 NGI rating: Q = 500	m s *m* *s*	7.00 1.00 *7.00* *1.00*	10.00 1.00 *10.00* *1.00*	15.00 1.00 *15.00* *1.00*	17.00 1.00 *17.00* *1.00*	25.00 1.00 *25.00* *1.00*
VERY GOOD QUALITY ROCK MASS *Tightly interlocking undisturbed rock with unweathered joints at 1 to 3m.* CSIR rating: RMR = 85 NGI rating: Q = 100	m s *m* *s*	2.40 0.082 *4.10* *0.189*	3.43 0.082 *5.85* *0.189*	5.14 0.082 *8.78* *0.189*	5.82 0.082 *9.95* *0.189*	8.56 0.082 *14.63* *0.189*
GOOD QUALITY ROCK MASS *Fresh to slightly weathered rock, slightly disturbed with joints at 1 to 3m.* CSIR rating: RMR = 65 NGI rating: Q = 10	m s *m* *s*	0.575 0.00293 *2.006* *0.0205*	0.821 0.00293 *2.865* *0.0205*	1.231 0.00293 *4.298* *0.0205*	1.395 0.00293 *4.871* *0.0205*	2.052 0.00293 *7.163* *0.0205*
FAIR QUALITY ROCK MASS *Several sets of moderately weathered joints spaced at 0.3 to 1m.* CSIR rating: RMR = 44 NGI rating: Q = 1	m s *m* *s*	0.128 0.00009 *0.947* *0.00198*	0.183 0.00009 *1.353* *0.00198*	0.275 0.00009 *2.030* *0.00198*	0.311 0.00009 *2.301* *0.00198*	0.458 0.00009 *3.383* *0.00198*
POOR QUALITY ROCK MASS *Numerous weathered joints at 30-500mm, some gouge. Clean compacted waste rock* CSIR rating: RMR = 23 NGI rating: Q = 0.1	m s *m* *s*	0.029 0.000003 *0.447* *0.00019*	0.041 0.000003 *0.639* *0.00019*	0.061 0.000003 *0.959* *0.00019*	0.069 0.000003 *1.087* *0.00019*	0.102 0.000003 *1.598* *0.00019*
VERY POOR QUALITY ROCK MASS *Numerous heavily weathered joints spaced <50mm with gouge. Waste rock with fines.* CSIR rating: RMR = 3 NGI rating: Q = 0.01	m s *m* *s*	0.007 0.0000001 *0.219* *0.00002*	0.010 0.0000001 *0.313* *0.00002*	0.015 0.0000001 *0.469* *0.00002*	0.017 0.0000001 *0.532* *0.00002*	0.025 0.0000001 *0.782* *0.00002*

Ref 4

Table 5-2 Bearing Resistance Factors

Method/Soil/Condition	Bearing Resistance Factor, φ_b
Theoretical method (Munfakh et al. 2001) in clay	0.50
Theoretical method (Munfakh et al. 2001) in sand using SPT	0.45
Semi-empirical methods (Meyerhof, 1957, Terzaghi, Vesic) all soils	0.45
Footings on rock	0.45
Plate Load Test	0.50

5.3.6 Settlement

The design of spread footings is frequently controlled by settlement at the service limit state. It is advantageous to proportion spread footings at the service limit state and check for adequate design at the strength and extreme limit states.

Total and differential settlement should be evaluated. The total settlement includes elastic settlement, primary consolidation, and secondary compression. Elastic settlement results from the compression of the material supporting the foundation or from reduction in pore space in nonsaturated soils. Consolidation settlement occurs when saturated, fine-grained soils experience an increase in stress. Some soils, after experiencing primary consolidation settlement, continue to strain after excess pore-water pressures are dissipated. This process is termed secondary compression, or “creep”.

Immediate or elastic settlement should be determined using the Service I Load Combination, specified as unfactored dead load, plus the unfactored component of live loads assumed to extend to the footing level. Time-dependent settlements, i.e., primary consolidation and secondary compression settlement may be determined using the unfactored dead load only. Other factors that can affect settlement, such as embankment loading, lateral and/or eccentric loading, and dynamic or earthquake loads should also be considered, where applicable.

Differential settlement occurs when one load-bearing member of a structure experiences total settlement of a different magnitude than an adjacent load-bearing member. Transportation structures, especially bridges, are not exceptionally tolerant of differential settlements. Deformation limitations will form the upper bound of allowable differential settlements used to design shallow foundations.

Ref 5

Table 3.5 RMR classification of jointed rock masses (extract from Bieniawski, 1974)

Parameter			Ranges of values				
A. Classification parameters and their ratings							
1	Strength of intact rock material	Point load strength index	>8 MPa >1.2 ksi	4–8 MPa 0.6–1.2 ksi	2–4 MPa 0.3–0.6 ksi	1–2 MPa 0.8–0.3 ksi	For this low range uniaxial compressive test is preferred
		Uniaxial compressive strength	>200 MPa	100–200 MPa	50–100 MPa	25–50 MPa	10–25 MPa; 3–10 MPa; 1–3 MPa
	Rating		15	12	7	4	2; 1; 0
2	Drill core quality RQD		90%–100%	75%–90%	50%–75%	25%–50%	<25%
	Rating		20	17	13	8	3
3	Spacing of joints		>3 m (>10 ft)	1–3 m (3–10 ft)	0.3–1 m (1–3 ft)	50–300 mm (2–12 in)	<50 mm (<2 in)
	Rating		30	25	20	10	5
4	Condition of joints		Very rough surfaces Not continuous No separation Hard joint wall rock	Slightly rough surfaces Separation <1 mm Hard joint wall rock	Slightly rough surfaces Separation <1 mm Soft joint wall rock	Slicken sided surfaces or Gouge <5 mm thick or Joints open 1–5 mm Continuous joints	Soft gouge >5 mm thick or Joints open >5 mm Continuous joints
	Rating		25	20	12	6	0
5	Ground water			Completely dry	Moist only (interstitial water)	Water under moderate pressure	Severe water problem
	Rating			10	7	4	0
			* When calculating rock strength using Table 3.7, rating = 10; ground water pressures accounted for in stability analysis.				
B. Rating adjustment for joint orientations							
	Orientation of joints Adjustment for foundations		Very favorable	Favorable	Fair	Unfavorable	Very unfavorable
			0	−2	−7	−15	−25
			* When calculating rock strength using Table 3.7, adjustment = 0; joint orientation accounted for in stability analysis.				

↳ m+s parameters

↳ m+s parameters

Wyllie, D.C., 1999. Foundations on Rock, 2nd Ed., E+FN Spon, London, 401 p.

1/2

Ref 5

Table 3.7 Approximate relationship between rock mass quality and material constants (Hoek and Brown, 1988)

Empirical failure criterion: $\sigma_1' = \sigma_3' + \sqrt{m\sigma_{u(r)}\sigma_3' + s\sigma_{u(r)}^2}$; σ_1' = major principal effective stress; σ_3' = minor principal effective stress; $\sigma_{u(r)}$ = uniaxial compressive strength of intact rock, and *m* and *s* are empirical constants.		CARBONATE ROCKS WITH WELL DEVELOPED CRYSTAL CLEAVAGE *dolomite, limestone and marble*	LITHIFIED ARGILLACEOUS ROCKS *mudstone, siltstone, shale and slate (normal to cleavage)*	ARENACEOUS ROCKS WITH STRONG CRYSTALS AND POORLY DEVELOPED CRYSTAL CLEAVAGE *sandstone and quartzite*	FINE GRAINED POLYMINERALLIC IGNEOUS CRYSTALLINE ROCKS *andesite, dolerite, diabase and rhyolite*	COARSE GRAINED POLYMINERALLIC IGNEOUS & METAMORPHIC CRYSTALLINE ROCKS *amphibolite, gabbro gneiss, granite, norite, quartz-diorite*
INTACT ROCK SAMPLES *Laboratory size specimens free from discontinuities* *CSIR rating: *RMR* = 100 †NGI rating: *Q* = 500	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 1–3 m* CSIR rating: *RMR* = 85 NGI rating: *Q* = 100	m s	2.40 0.082	3.43 0.082	5.14 0.082	5.82 0.082	8.56 0.082
GOOD QUALITY ROCK MASS *Fresh to slightly weathered rock, slightly disturbed with joints at 1–3 m* CSIR rating: *RMR* = 65 NGI rating: *Q* = 10	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 0.3–1 m* CSIR rating: *RMR* = 44 NGI rating: *Q* = 1	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 30–500 mm, some gouge. Clean compacted waste rock* CSIR rating: *RMR* = 23 NGI rating: *Q* = 0.1	m s	0.029 0.000003	0.041 0.000003	0.061 0.000003	0.069 0.000003	0.102 0.000003
VERY POOR QUALITY ROCK MASS *Numerous heavily weathered joints spaced <50 mm with gouge. Waste rock with fines* CSIR rating: *RMR* = 3 NGI rating: *Q* = 0.01	m s	0.007 0.0000001	0.010 0.0000001	0.015 0.0000001	0.017 0.0000001	0.025 0.0000001

*CSIR Council of Scientific and Industrial Research (Bieniawski, 1974).
†NGI Norwegian Geotechnical Institute (Barton *et al.*, 1974).

2/2

Ref 6

Borehole	Run	Lithologic Description	Strength	RQD	Spacing	Persistence	Aperature	Roughness	Infilling	Weathering	Groundwater*	Orientation*	RMR
BB-HAMP-101	Run 1	Vassalboro Formation (Silurian – Ordovician), geyish black (N2), fine grained, slightly weathered to fresh, medium weak rock (R3), PHYLLITE, discontinuity spacing is extremely close to closely spaced (<20 mm to 80 mm), dipping 55 to 85 degrees relative to horizontal axis, discontinuity surfaces are planar to curved to stepped, texture is smooth to very rough,	5	3	5	2	4	4	6	5	10	0	44
	Run 2	Vassalboro Formation (Silurian – Ordovician), greyish black (N2), fine grained, highly weathered to fresh with highly weathered zone from 26.8 feet to 28.5 feet below ground surface, medium weak rock (R3), weathered zone is very weak rock (R1), PHYLLITE, spacing is extremely close to closely spaced (<20 mm to 100 mm), dipping 85 to 90 degrees relative to horizontal axis, discontinuity surfaces are planar to stepped, texture is smooth to very rough, trace pyrite and possibly iron staining in joints.	4	3	5	2	4	4	2	3	10	0	37
BB-HAMP-102	Run 1	Vassalboro Formation (Silurian – Ordovician), medium dark grey (N4), fine grained, fresh, medium weak (R3), PHYLLITE, discontinuity spacing is extremely close to moderately spaced (<20 mm to 460 mm), dipping 55 to 85 degrees relative to horizontal axis, discontinuity surfaces are planar to curved to stepped, texture is smooth to rough.	5	7	8	2	4	4	6	6	10	0	52
	Run 2	Vassalboro Formation (Silurian – Ordovician), medium dark grey (N4), fine grained, fresh, medium weak (R3), PHYLLITE, discontinuity spacing is very close to moderately spaced, dipping 55 to 85 degrees relative to horizontal axis, discontinuity surfaces are planar to curved, texture is smooth to rough, white seam observed at about 23.5 feet below ground surface.	5	14	8	2	1	3	6	5	10	0	54

When calculating *m,s* , groundwater rating = 10, orientation of joints = 0; ground water pressures and joint orientation accounted for in stability analysis (Bieniawski, 1974)

Client:	Golder Associates				
Project:	Rt 69 Culvert Replacement				
Location:	Hampden, ME			Project No:	GTX-306502
Boring ID:	BB-HAMP-102	Sample Type:	cylinder	Tested By:	rlc
Sample ID:	R1	Test Date:	06/05/17	Checked By:	jsc
Depth :	17.7-19 ft	Test Id:	412758		

Ref 7

Test Comment: ---

Visual Description: See photograph(s)

Sample Comment: ---

Bulk Density and Compressive Strength of Rock Core Specimens by ASTM D7012 Method C

Boring ID	Sample Number	Depth	Bulk Density, pcf	Compressive strength, psi	Failure Type	Meets ASTM D4543	Note(s)
BB-HAMP-102	R1	17.91-18.29 ft	168	11473	2	Yes	---

Notes: Density determined on core samples by measuring dimensions and weight and then calculating.

All specimens 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.

Failure Type: 1 = Intact Material Failure; 2 = Discontinuity Failure; 3 = Intact Material and Discontinuity Failure
(See attached photographs)

Ref 7

GeoTesting EXPRESS

Client:	Golder Associates	Test Date:	6/5/2017
Project Name:	Rt 69 Culvert Replacement	Tested By:	rlc
Project Location:	Hampden, ME	Checked By:	jsc
GTX #:	306502		
Boring ID:	BB-HAMP-102		
Sample ID:	R1		
Depth:	17.91-18.29 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:	4.19	4.19	4.19
Specimen Diameter, in:	1.99	1.99	1.99
Specimen Mass, g:	577.25		
Bulk Density, lb/ft^3	168		
Length to Diameter Ratio:	2.1		

Minimum Diameter Tolerence Met? YES

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 ≤ 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	-0.00020	-0.00010	0.00000	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010
Diameter 2, in (rotated 90°)	-0.00030	-0.00020	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020

Difference between max and min readings, in:

0° = 0.00030 90° = 0.00030

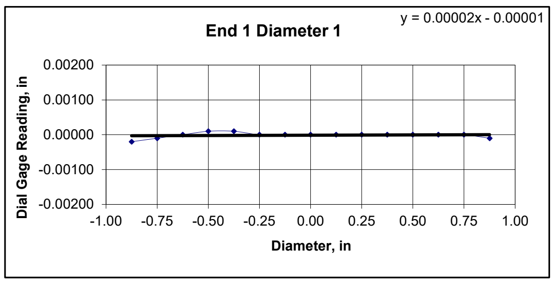
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00010	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010
Diameter 2, in (rotated 90°)	0.00010	-0.00020	-0.00010	0.00000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

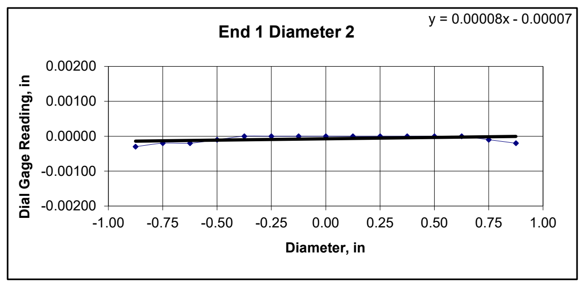
Difference between max and min readings, in:

0° = 0.0002 90° = 0.0003

Maximum difference must be < 0.0020 in. Difference = ± 0.00015

Flatness Tolerance Met? YES





DIAMETER 1

End 1:

Slope of Best Fit Line	0.00002
Angle of Best Fit Line:	0.00115

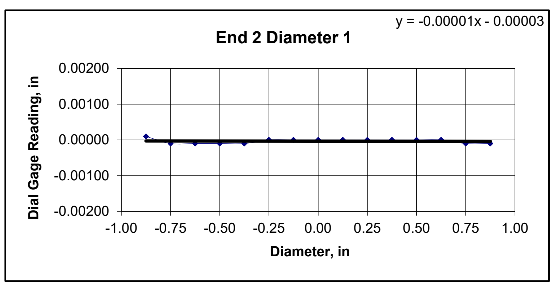
End 2:

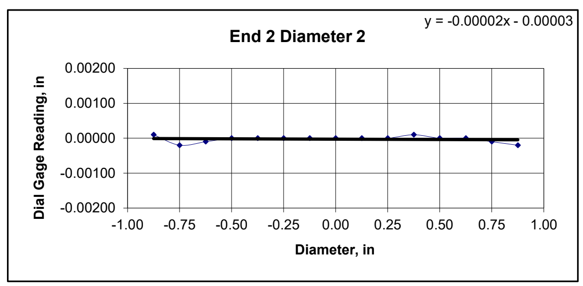
Slope of Best Fit Line	0.00001
Angle of Best Fit Line:	0.00057

Maximum Angular Difference: 0.00057

Parallelism Tolerance Met? YES

Spherically Seated





DIAMETER 2

End 1:

Slope of Best Fit Line	0.00008
Angle of Best Fit Line:	0.00458

End 2:

Slope of Best Fit Line	0.00002
Angle of Best Fit Line:	0.00115

Maximum Angular Difference: 0.00344

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	0.00030	1.990	0.00015	0.009	YES
Diameter 2, in (rotated 90°)	0.00030	1.990	0.00015	0.009	YES
END 2					
Diameter 1, in	0.00020	1.990	0.00010	0.006	YES
Diameter 2, in (rotated 90°)	0.00030	1.990	0.00015	0.009	YES

Maximum angle of departure must be ≤ 0.25°

Perpendicularity Tolerance Met? YES

Ref 7

GeoTesting EXPRESS

Client:	Golder Associates
Project Name:	Rt 69 Culvert Replacement
Project Location:	Hampden, ME
GTX #:	306502
Test Date:	6/5/2017
Tested By:	rlc
Checked By:	jsc
Boring ID:	BB-HAMP-102
Sample ID:	R1
Depth, ft:	17.91-18.29

After cutting and grinding

After break

APPENDIX E

Hager-Richter Report

HAGER-RICHTER
GEOSCIENCE, INC.

GEOPHYSICAL SURVEY
TWIN BRIDGE # 5315
WEST BRANCH SOUADABSCOOK STREAM
ME ROUTE 69
HAMPDEN, MAINE

Prepared for:

Golder Associates Corporation
670 North Commercial Street, Suite 103
Manchester, New Hampshire 03101

Prepared by:

Hager-Richter Geoscience, Inc.
8 Industrial Way - D10
Salem, New Hampshire 03079

File 19MH19
October, 2020

October 14, 2020
File 19MH19

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RE: Geophysical Survey
Twin Bridge # 5315
West Branch Souadabscook Stream
ME Route 69
Hampden, Maine

Dear Ms. Melendy:

In this letter, we report the results of a geophysical survey conducted by Hager-Richter Geoscience, Inc. (HRGS) on September 25, 2020 at the above referenced site for Golder Associates Corporation (Golder). The scope of the project and area of interest for the survey were specified by Golder.

INTRODUCTION

The site is an existing culvert carrying the West Branch of Souadabscook Stream Maine under ME Route 69 in Hampden, Maine. Figure 1 shows the general site location. According to information provided by Golder, the existing culvert was constructed in 1951 and replaced an old twin bridge structure. The twin bridge structure was bearing on dry stacked stone walls/abutments, and it is not known if these structures were removed in 1951 to construct the current culvert. As part of a geotechnical investigation for the design of a culvert replacement structure, Golder requested a geophysical survey to detect, and if detected, to locate the dry stacked stone walls/abutments related to the twin bridge structure.

The area of interest (AOI) for the geophysical survey was specified by Golder to be the full width of the paved roadway of ME Route 69 from approximately 40 feet south of the existing culvert extending to approximately 180 feet north of the existing culvert. The roadway is approximately 30 feet wide in the AOI, and has guardrails present along both the east and west sides.

OBJECTIVE

The objective of the geophysical survey was to detect, and if detected, to determine the locations of stacked stone walls/abutments or other subsurface features related to the former arch bridge in the accessible portions of a specified area of interest.

THE SURVEY

Michael Howley, P.G., and Bryan Carnahan of HRGS conducted the field operations on September 25, 2020. The project was coordinated with Megan Melendy and Cameron Stewart of Golder. Colby Howland of Golder was on-site for the duration of the geophysical survey and specified the limits of the area of interest (AOI) for the survey. Data analysis and interpretation were completed at the Hager-Richter offices. Original data and field notes will be retained in the Hager-Richter files for a minimum of three (3) years.

The geophysical survey was conducted using the ground penetrating radar (GPR) method. The GPR survey was conducted where access allowed along traverses oriented parallel to the roadway spaced 2 feet apart.

A local survey grid was established for the acquisition of the geophysical data. The positions of the survey grid and other site features were recorded using a Trimble Geo7X CM GPS system utilizing a Zephyr-2 external antenna. The results of the survey are presented overlain on a plan provided by Golder showing the modern culvert and location of the former twin bridge spans.

EQUIPMENT

The GPR survey was conducted using a GSSI SIR 4000 digital subsurface imaging radar system. The system was used with a Hyperstacking 350 MHz antenna. Data were recorded using a 96 ns time window for the 350 MHz antenna.

GPR uses a high-frequency electromagnetic pulse (referred to herein as “radar signal”) transmitted from a radar antenna to probe the subsurface. The transmitted radar signals are reflected from subsurface interfaces of materials with contrasting electrical properties. Travel times of the radar signal can be converted to approximate depth below the surface by correlation with targets of known depths and by a curve matching routine. We monitor the acquisition of GPR data in the field and record the GPR data digitally for subsequent processing. Interpretation of the records is based on the nature and intensity of the reflected signals and on the resulting patterns.

Data from the GPR survey were processed using RADAN 7.6, commercially licensed GPR processing software from GSSI, and the profile images were interpreted. Interpretation of the records is based on the nature and intensity of the reflected signals and on the resulting patterns.

LIMITATIONS OF THE METHOD

> HAGER-RICHTER GEOSCIENCE, INC. MAKES NO GUARANTEE THAT ALL TARGETS WERE DETECTED IN THIS SURVEY. HAGER-RICHTER GEOSCIENCE, INC. IS NOT RESPONSIBLE FOR DETECTING TARGETS THAT NORMALLY CANNOT BE DETECTED BY THE METHODS EMPLOYED OR THAT COULD NOT BE DETECTED BECAUSE OF SITE CONDITIONS.

GPR detects and maps interfaces of contrasting electrical properties, and air- or water filled bedrock fractures or voids have electrical properties very different from soil and concrete. The GPR method is useful for detecting fractures or voids and determining their footprint, but in general, GPR data cannot be used to determine the thickness of fracture openings or voids.

There are other limitations of the GPR technique: (1) surface conditions, (2) electrical conductivity and thickness of the subsurface layers, (3) electrical properties of the target(s), and (4) spacing of the traverses. Of these restrictions, only the last is controllable by us in most cases.

The condition of the survey surface can affect the quality of the GPR data and the depth of penetration of the GPR signal. For exterior sites, a surface covered with obstacles such as automobiles, dumpsters, thick leaf debris, materials piles, etc. limit the survey access. Similarly, for interior sites, a surface covered with obstacles such as desks, benches, laboratory equipment, etc. also limit access. Some floor coverings may limit the coupling of the GPR antenna with the subsurface.

The electrical conductivity of the subsurface determines the attenuation of the GPR signals, and thereby limits the maximum depth of exploration. The GPR signal does not penetrate clay-rich soils or soils contaminated with road salt. In some cases, the GPR signal may not penetrate below concrete pavement, and some asphalts are electrically conductive.

A strong contrast in the electrical conductivities of the ground and the target (for examples, UST, pipe, void, dry well, drum, contaminant plume) is required to obtain a reflection of the GPR signal. If the contrast is too small, then the reflection may be too weak to recognize, and the target can be missed.

Spacing of the traverses is limited by access at many sites, but where flexibility of traverse spacing is possible, the spacing is adjusted on the basis of the size of the target.

RESULTS

The geophysical survey was conducted using the ground penetrating radar (GPR) method. The GPR survey was conducted along traverses oriented parallel to the roadway spaced 2 feet apart where access allowed. Figure 2 shows the locations of the GPR transects and the locations of possible features detected on the basis of the GPR data.

Apparent GPR signal penetration for was generally good, with two-way traveltime reflections received from approximately 55 to 65 ns of the 96 ns records acquired with the 350 MHz antenna. Based on velocity matching calibrations made for the site, the GPR signal penetration in the area of interest is estimated to have been about 8 to 10 feet.

GPR records acquired in the active roadway contain reflections attributed to the stacked stone former arch bridge abutments and the filled in former arch bridge span. The locations of the possible former arch bridge abutments and filled in former arch bridge span are shown in Figure 2. The presence of the possible former arch bridge abutments cannot be confirmed on the basis of the GPR data alone and can only be confirmed by test borings or test excavations.

The extent of the modern steel culvert was also determined on the basis of GPR records and is also shown in Figure 2. GPR records acquired north of the modern culvert also contain reflections attributed to a possible former retaining wall or linear structure oriented perpendicular to the roadway located near the northern limit of the geophysical survey area. The location of the interpreted possible former retaining wall is shown in Figure 2.

GPR records also contain reflections attributed to several areas of possible shallow bedrock or other unknown buried objects located both between the modern culvert and the former bridge structure and north of the former bridge structure. The locations of both interpreted features are shown in Figure 2.

CONCLUSIONS

Based on the geophysical survey performed by HRGS at Twin Bridge # 5315 over the West Branch Souadabscook Stream along ME Route 69 in Hampden, Maine, we conclude:

- The possible former bridge abutments and the possible filled in former arch bridge span were detected on the basis of GPR records.

- A possible retaining wall oriented perpendicular to the roadway was detected near the northern limit of the area of interest.

- Several areas of possible shallow bedrock or unknown buried objects were detected north of the modern culvert.

LIMITATIONS ON USE OF THIS REPORT

This letter report was prepared for the exclusive use of Golder Associates Corporation (Client). No other party shall be entitled to rely on this Report, or any information, documents, records, data, interpretations, advice or opinions given to Client by HRGS in the performance of its work. The Report relates solely to the specific project for which HRGS has been retained and shall not be used or relied upon by Client or any third party for any variation or extension of this project, any other project or any other purpose without the express written permission of HRGS. Any unpermitted use by Client or any third party shall be at Client's or such third party's own risk and without any liability to HRGS.

HRGS has used reasonable care, skill, competence and judgment in the performance of' its services for this project consistent with professional standards for those providing similar services at the same time, in the same locale, and under like circumstances. Unless otherwise stated, the work performed by HRGS should be understood to be exploratory and interpretational in character and any results, findings or recommendations contained in this Report or resulting from the work proposed may include decisions which are judgmental in nature and not necessarily based solely on pure science or engineering. It should be noted that our conclusions might be modified if subsurface conditions were better delineated with additional subsurface exploration including, but not limited to, test pits, soil borings with collection of soil and water samples, and laboratory testing.

Except as expressly provided in this limitations section, HRGS makes no other representation or warranty of any kind whatsoever, oral or written, expressed or implied; and all implied warranties of merchantability and fitness for a particular purpose, are hereby disclaimed.

If you have any questions or comments on this letter report, please contact us at your convenience. It has been a pleasure to work with Golder on this project. We look forward to working with you again in the future.

Sincerely,
HAGER-RICHTER GEOSCIENCE, INC.

Michael Howley, P.G.
Geophysicist

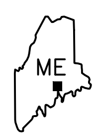
Jeffrey Reid, P.G.
Owner / Principal Geophysicist

Attachments: Figures 1 & 2

N

APPROXIMATE SCALE (feet)

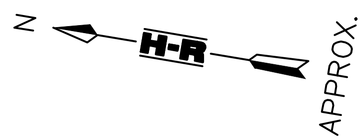
0 1000 2000



NOTE:

Modified from Google Earth Pro aerial photograph.

Figure 1 General Site Location West Branch Souadabscook Stream ME Route 69 Hampden, Maine	
File 19MH19	October, 2020
HAGER—RICHTER Salem, NH | Fords, NJ	



LEGEND

- GPR TRAVERSE
- FILLED IN FORMER ARCH BRIDGE
- POSSIBLE FORMER ARCH BRIDGE ABUTMENTS
- POSSIBLE FORMER RETAINING WALL
- POSSIBLE SHALLOW BEDROCK OR UNKNOWN BURIED OBJECTS
- UNKNOWN BURIED OBJECTS
- MODERN CULVERT PIPE

SCALE (feet)

0 20 40

NOTE:

Modified from site plan provided by Golder Associates Corporation, identified as 190313 Hampden Final PDR (page 17).pdf.

Figure 2 GPR Survey West Branch Souadabscook Stream ME Route 69 Hampden, Maine	
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