GEOTECHNICAL DESIGN REPORT

FIELDS BRIDGE NO. 0690
Gammon Road over E. Branch of the Nezinscot River
Sumner-Hartford, Maine
MaineDOT WIN 21705.00

FOR

Maine Department of Transportation
Augusta, ME

BY

NOBIS ENGINEERING, INC.

(800) 394-4182
www.nobiseng.com

Nobis Project No. 92270.04
August 3, 2018

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August 3, 2018
Project No. 92270.04

Ms. Laura Krusinski, P.E.
Senior Geotechnical Engineer
Maine Department of Transportation
State House Station 16
Augusta, Maine 04333-016
Laura.Krusinski@maine.gov

Re: Geotechnical Design Report
Fields Bridge No. 0690
Gammon Road over E. Branch of the Nezinscot River
Sumner-Hartford, Maine
WIN 21705.00

Dear Ms. Krusinski,

We are pleased to provide this Geotechnical Design Report to MaineDOT for the Fields Bridge No. 0690 which carries Gammon Road over the East Branch of the Nezinscot River in Sumner-Hartford, Maine. This report has been completed in accordance with the Project-Specific Task Order No, 20150610000000000814 dated December 1, 2017, and our proposal dated February 24, 2017 and contract modification #1, dated November 27, 2017.

It has been a pleasure serving the MaineDOT and Fuss and O'Neill on this project. Please let us know if you have any questions regarding this report.

Very truly yours,

Nobis Engineering, Inc.

Brien Waterman
Project Manager

STATE OF MAINE
KURT JELINEK
No. 11746
LICENSED
PROFESSIONAL ENGINEER
8/3/2018

Kurt Jelinek, P.E.
Director, Transportation Services

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1.0 INTRODUCTION

This geotechnical design report presents Nobis Engineering Inc.'s (Nobis) recommendations for the MaineDOT Fields Bridge No. 0690 WIN 21705.00 located in Sumner-Hartford, Maine. This report is subject to the limitations contained in Appendix A. Nobis performed geotechnical services in accordance with our original proposal dated February 24, 2017, and its addendum dated November 27, 2017.

2.0 PROJECT AND SITE DESCRIPTION

Nobis understands that MaineDOT intends to replace the Fields Bridge in Sumner-Hartford, Maine (the Site). A Site Locus Plan is included as **Figure 1**.

Fields Bridge – Sumner-Hartford, Maine

The existing two- (2-) span bridge was constructed in 1922 and has a total length of approximately 46 feet and a curb-to-curb roadway width of approximately 16.3 feet with no sidewalks. The existing superstructure consists of an open-grate steel deck supported by steel girders and beams, and a combination of concrete and dry-laid stone pier and abutments. Generally, southwest-northeast oriented overhead powerlines cross the bridge on both sides.

The top of the roadway at the existing bridge approaches and along the bridge deck is relatively level, at approximately elevation (El.) 401, whereas the bed of the river is at approximately El. 390. Elevations referenced in this report are relative to the North American Vertical Datum of 1988 (NAVD 88). Refer to **Figure 2** for the approximate location of existing and proposed site features.

It is our understanding that MaineDOT intends to replace the existing structure on the same alignment on new abutments. The proposed bridge will have a span of 68 feet and will be supported by spread footings/concrete seals bearing on bedrock.

3.0 SUBSURFACE EXPLORATIONS

New England Boring Contractors (NEBC) of Hermon, Maine drilled four (4) test borings, of which BB-SHEBNR-101 and BB- SHEBNR-102 were performed in June 2017, and BB-SHEBNR-103 and BB- SHEBNR-104 in December 2017. Test boring BB-SHEBNR-101 was performed in the vicinity of the South Abutment and test borings BB-SHEBNR-102, BB-SHEBNR-103, and BB-SHEBNR-104 were performed in the vicinity of the North Abutment.

The test borings were advanced using drive and wash methods and auger methods. Auger methods were used to advance boring BB-SHEBNR-103, which encountered refusal at approximately 3.75 feet below grades on concrete which was presumed to be the existing abutment. The test borings were observed and logged by Nobis personnel. The borings were drilled to depths ranging from approximately 4 to 31 feet below existing grades. NX-size bedrock cores were collected from three of the four test borings.

Boring logs are included in **Appendix B**. The approximate test boring locations are shown on **Figure 2**, Boring Location Plan. For the NEBC automatic hammer energy transfer ratio (ETR) calibration report, refer to **Appendix C**. Photo logs of the bedrock cores are provided in **Appendix D**.

4.0 LABORATORY TESTING

Samples of rock/soil were selected by Nobis and submitted to GeoTesting Express of Acton, Massachusetts for laboratory testing. Laboratory testing included:

- Four (4) grain size analyses – sieve only (in accordance with ASTM D422); and
- Three (3) compressive strength and elastic moduli of rock test (in accordance with ASTM D7012).

The laboratory test results are included in **Appendix E**.

<u>Streambed Soil Sample</u>

Based on the particle size analysis performed on a soil sample obtained by Nobis, streambed soils consist of brown, fine to medium sand with trace silt and trace fine gravel. Based on laboratory testing, the D_{50} of the streambed sample was 0.44 millimeters.

5.0 SUBSURFACE CONDITIONS

Existing Information

Based on a review of surficial geologic maps, the surficial geologic conditions at the site most likely consist of stream alluvium and/or outwash deposits. A site-focused 2008 USGS surficial geologic map entitled "Surficial Geology of the Worthley Pond Quadrangle, Maine" (Thompson, Eusden Jr., Tolman, Tucker, Marvinney) along with the corresponding description of geologic units are provided on **Figure 3**.

Per a 1978 USGS bedrock geologic map entitled "Reconnaissance Bedrock Geology of the Buckfield Quadrangle, Maine" (Pankiwiskyj), bedrock at the site is that of the Sangerville Formation (Anasagunticook Member), which consists of "…gray metasiltstone and dark gray metapelite, subordinate, light gray, calcareous graywacke metasandstone…" For a site-focused plan view of the bedrock geologic map, including a more detailed description of the bedrock at the site, refer to **Figure 4**.

Subsurface Conditions Encountered

The generalized stratigraphy encountered in the test borings consisted of asphalt overlying granular fill, outwash deposits, and bedrock to the termination depth. An interpretive subsurface profile is included as **Figure 5**. Generalized descriptions of the subsurface conditions encountered in the test borings are discussed below, in order of increasing depth:

<u>Pavement:</u> Approximately three (3) inches of roadway Hot Mix Asphalt (HMA) pavement was encountered in the borings.

Fill: The fill encountered in the test borings generally consisted of dry to moist, brown, medium dense to very dense, fine to coarse sand with varying amounts of silt and gravel, occasionally including root fibers and asphalt particles and fragments. An inferred strata transition based on a change in drilling behavior and wash color indicated that there may be fill up to approximately nine (9) feet deep in the bridge approach embankments.

Outwash Deposits: Based on the samples obtained in the test borings, the outwash deposits generally consisted of tan or light brown, wet, medium dense to dense, fine to coarse sand with varying amounts of gravel and silt. A split-spoon sample of the outwash deposits is pictured right.

Weathered Bedrock: BB-SHEBNR-101 drive Sample 4D contained approximately two (2) inches of weathered bedrock particles and fragments at a depth of approximately 10.3 feet bgs, or El. 390.7.

Bedrock: Bedrock was encountered at approximately 11 feet (corresponding to El. 390) and 18.6 feet (El. 382.4) bgs in BB-SHEBNR-101 and BB-SHEBNR-102, respectively. Bedrock was encountered at approximately 15 feet (El. 385.8) bgs in boring BB-SHEBNR-104.

Core samples from borings BB-SHEBNR-101, -102, and -104 generally consisted of grey, fresh, hard to very hard, fine- to medium-grained metasandstone and metasiltstone with quartzite intrusions, with very close to moderately close low-angle to moderately-dipping joints. Core samples R2 and, R3, and R4 from BB-SHEBNR-104 consisted of grey, fresh to slightly weathered, hard, medium to coarse-grained, quartzite/granite/mica, with horizontal to low-angle joints. Rock quality designations (RQDs) varied between 37% (poor quality) and 86% (good).

Groundwater: The elevation of the water in the East Branch Nezinscot River was measured at approximately 6.1 feet below the existing bridge deck, or El. 394.9, on April 5, 2017. Prior to drilling on June 26, 2017, the depth of the East Branch Nezinscot River was approximately 6.5 feet below the existing bridge deck, or El. 394.5.

Groundwater measurements were obtained under both cased and uncased borehole conditions after 5 to 20 minutes of stabilization. Groundwater measurements for borings BB-SHEBNR-101 and BB-SHEBNR-102 were recorded on June 29, 2017. The measurements varied between approximately 6.7 and 7.6 feet bgs, or El. 394.3 and El. 393.4, in BB-SHEBNR-101 and BB-SHEBNR-102, respectively. The groundwater measurement for BB-SHEBNR-104 was recorded on December 29, 2017 and was 4 feet bgs, or El. 397.

Note that water was introduced to boreholes during the test boring rotary wash process, and that fluctuations in the observed groundwater levels will occur due to variations in precipitation, river water level, temperature, and other factors different from those existing at the time the measurements were made.

6.0 GEOTECHNICAL DESIGN RECOMMENDATIONS

The following sections present our geotechnical engineering evaluations of the subsurface conditions relative to replacement of the North and South Abutments.

6.1 Spread Footings Supported by Bedrock

The test borings encountered bedrock approximately 11 to 18.5 feet below existing ground surface. It is considered feasible that concrete seals and shallow spread footings can be constructed on bedrock. The subgrades should be prepared as discussed in the Construction Considerations section below.

6.2 Bearing Resistance

We evaluated the bearing resistance of the proposed North and South Abutments assuming shallow spread footings/concrete seals are constructed directly on bedrock. The bedrock is considered non-frost susceptible.

Nobis evaluated the bearing resistance of the bedrock using the semi-empirical method by Carter and Kulhawy (1989). In accordance with AASHTO LRFD Bridge Design Specifications: 2014 Edition, Section C10.4.6.4, the bearing resistance of bedrock should be evaluated based on Rock Mass Rating (RMR). The RMR of the bedrock cores collected in the vicinity of the North and South Abutments were evaluated using AASHTO LRFD Bridge Design Specifications: 2012 Edition. We estimate the RMR varies between 42 and 51.

The bearing resistance of the bedrock was calculated using a RMR of 42, and a uniaxial compressive strength of 12.3 ksi of the intact rock. The uniaxial compressive strength of the bedrock is based on laboratory test values. The bearing resistance calculation indicates a factored bearing resistance (q_R) of 35.3 ksf for the Strength Case using a resistance factor of 0.45, and 78.4 ksf for the Extreme Case using a resistance factor of 1.0.

Calculations for bearing resistance and settlements are presented in **Appendix F**.

6.3 Lateral Earth Pressures

We recommend that the proposed abutments be designed for lateral earth pressures using backfill material properties for Soil Type 4 (MaineDOT Bridge Design Guide Section 3.6.1). We recommend that the abutments be designed based on the following soil parameters:

Active earth pressure coefficient = 0.31 (Rankine's Method)
Sliding resistance factor for abutments, φ_t = 0.8 (AASHTO Table 10.5.5.2.2-1)
Friction angle, ϕ, = 32 deg. for retained soil
Soil density, γ = 125 pcf for retained soil

The active earth pressure coefficient is based on a vertical backface for the abutment and level backfill behind the abutments.

In addition, a live load surcharge should be applied to account for vehicular traffic (AASHTO Article 3.11.6.4). The live load surcharge may be estimated as a uniform horizontal earth pressure due to an equivalent height of soil in accordance with MaineDOT Bridge Design Guide Table 3-4.

A sliding coefficient of friction (tan ϕ_f) of 0.7 and a value of C = 1.0 is recommended for the abutments (AASHTO 2016 Interim Revisions 10.6.3.4-2). These values are based on the assumption that the footings/seals are concrete cast against bedrock. We recommend ignoring passive pressure, as the resisting soil could be scoured away. Proposed abutments should be designed to drain.

6.4 Seismic Design Considerations

Based on the SPT-N values, and using Method B (Table C3.10.3.1-1), the average SPT blow count is above 50 bpf. As discussed above, the proposed abutments are to be supported directly over bedrock. We assumed that the bedrock encountered has a shear wave velocity greater than 2,500 ft/sec, therefore, the bridge is in a Site Class "B".

The seismic parameters developed for the proposed bridge are provided below per the AASHTO 7th Edition:

Mapped Ground and Spectral Response Coefficients (USGS Seismic Design Maps):

- Horizontal Peak Ground Acceleration (PGA): 0.089
- Horizontal Response Spectral Acceleration, 0.2 Sec (S_S): 0.180
- Horizontal Response Spectral Acceleration, 1.0 Sec (S_1): 0.048

Site Class: B (AASHTO Table 3.10.1-1):

- Site Factors for Site Class "B" (AASHTO Tables 3.10.3.2-1, -2, and -3):
  Zero-Period (F_{pga}) = 1.0, Short-Period, 0.2 Sec (F_a) = 1.0, and Long Period, 1.0 Sec (F_v) = 1.0.

- Design Spectral Response Parameters for Site Class "B":
  A_S = 0.089, S_{DS} = 0.180, S_{D1} = 0.048.

Per AASHTO Article 3.10.6 the site is assigned Seismic Zone 1 based on a calculated S_{D1} of 0.048.

6.5 Construction Considerations

Subgrade Preparation Procedures

We recommend the following subgrade preparation procedures for the proposed abutments:

- Loose, fractured, or weathered bedrock should be excavated prior to placement of seal or spread footing concrete.

- Granular Borrow for Underwater Backfill should be used as backfill behind the proposed abutments, a minimum of 1.5 feet directly behind the heel of the proposed concrete seal sloping up at a minimum 1.5 Horizontal to 1 Vertical (1.5H:1V).

- Granular Borrow for Underwater Backfill should be placed in maximum 12 inch thick loose lifts, and compacted to at least 95 percent of its maximum dry density as determined by ASTM D-1557, Method C (Modified Proctor).

Re-Use of On-site Soil

Based on soils encountered in test borings, the existing fill material consisted of sand with approximately 10% to more than 20% silt. Materials with fines up to 20% can be difficult to reuse if wet. We recommend those materials be reused in landscape areas or be disposed of offsite. It is possible that existing fill materials with 10% silt or less could be mixed to meet the gradation requirements of Granular Borrow for Underwater Backfill.

Excavation and Temporary Lateral Support

We understand that the proposed North and South Abutments will be founded on shallow spread footings/concrete seals over bedrock. Based on the borings, bedrock is shallow within the stream, therefore, sandbags, or other means may be needed to demolish the existing abutments and construct the new abutments.

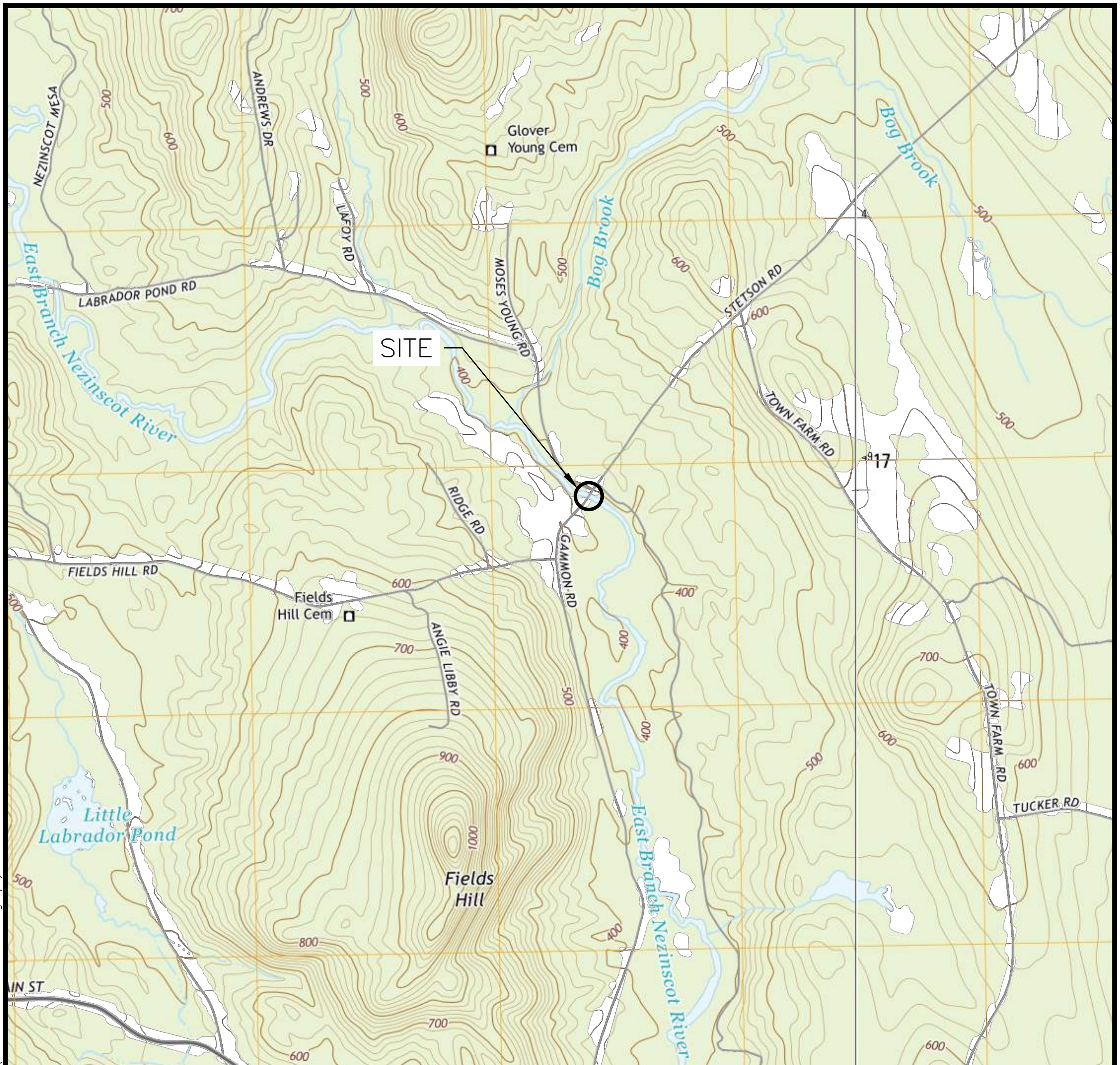
Excavation geometry should conform to OSHA excavation regulations contained in 29 CFR Part 1926, latest edition. In general, temporary soil slopes of 1.5H:1V (Soil Profile Type C), or flatter, appear appropriate but should be confirmed during construction based on conditions at the time of excavation.

Temporary earth support may be needed for construction of the bridge. If needed, temporary earth support systems must be designed by a professional engineer registered in the State of Maine.

Temporary Dewatering

In general, groundwater was encountered above the bottom of the proposed abutments. Temporary excavation dewatering should be performed so that the work conducted is completed in the dry. It is likely that dewatering may be accomplished by filtered sumps installed in low points of the excavation. Discharge water should be managed in accordance with local, state and federal government requirements.

FIGURES



2014 USGS TOPOGRAPHIC MAPS

NORTH

WORTHLEY POND AND CANTON QUADRANGLES
SUMNER-HARTFORD, MAINE
CONTOUR INTERVAL 20 FEET
NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88)

APPROXIMATE SCALE
1 INCH = 2,000 FEET

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QUADRANGLE LOCATION

FIGURE 1

SITE LOCUS PLAN
FIELDS BRIDGE (#0690)
GAMMON ROAD OVER E. BRANCH
NEZINSCOT RIVER
SUMNER-HARTFORD, MAINE

PROJECT NO. 92270.04

DATE: JUNE 2018

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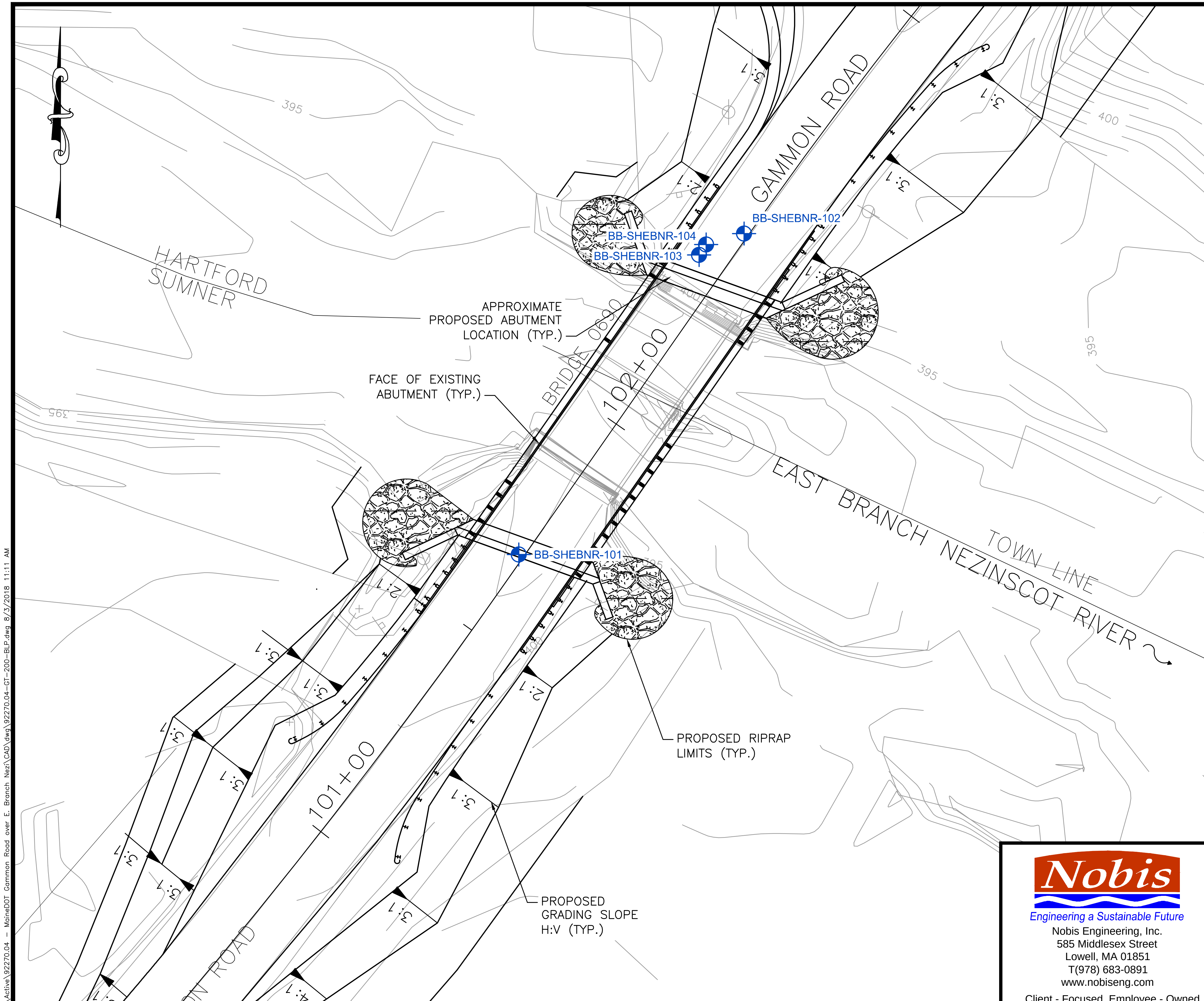

NOTES:

1. THE BASE PLAN WAS DEVELOPED FROM .DGN FILES PROVIDED BY MAINE DEPARTMENT OF TRANSPORTATION ON MAY 18, 2018.
2. SOIL BORINGS WERE DRILLED BY NEW ENGLAND BORING CONTRACTORS OF HERMON, MAINE, AND OBSERVED BY NOBIS ON JUNE 29 AND DECEMBER 4, 2017.
3. LOCATIONS AND SITE FEATURES DEPICTED ARE APPROXIMATE AND GIVEN FOR ILLUSTRATIVE PURPOSES.
4. ELEVATIONS ARE BASED ON THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88).

LEGEND

BB-SHEBNR-101 APPROXIMATE BORING LOCATION

0 20' 40'

GRAPHIC SCALE

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FIGURE 2

BORING LOCATION PLAN
FIELDS BRIDGE (#0690)
GAMMON ROAD OVER E. BRANCH NEZINSCOT RIVER
SUMNER-HARTFORD, MAINE

DRAWN BY:	PCC	CHECKED BY:	BTW
PROJECT NO.	92270.04	DATE:	AUGUST 3, 2018

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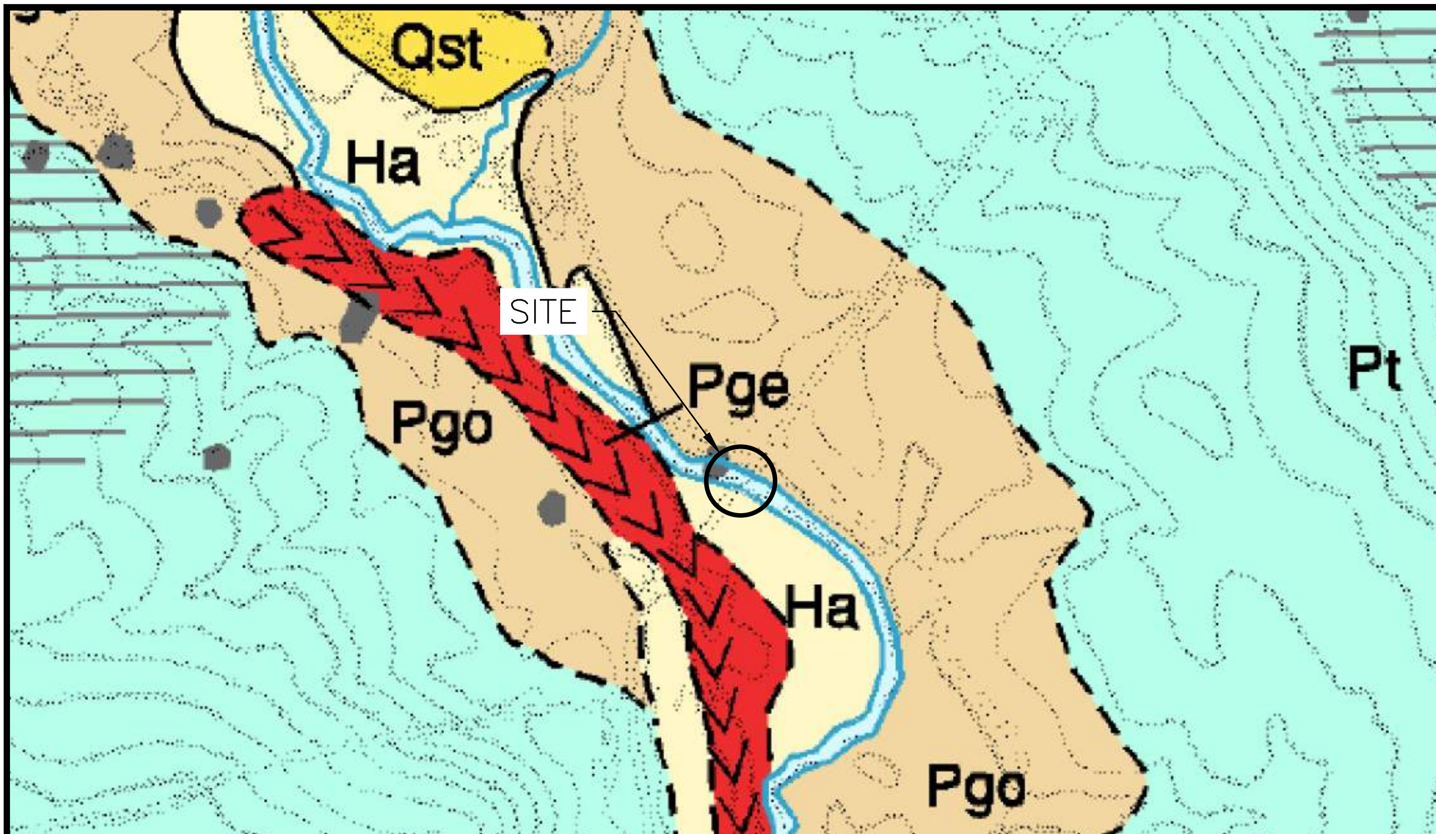

Ha	**Stream alluvium** - Sand, silt, gravel, and organic sediment. Deposited on the flood plain of the Androscoggin River and other modern streams. Unit includes some wetland areas.
Pgo	**Outwash deposits** - Sand and gravel deposited by glacial meltwater streams along the East Branch and West Branch of the Nezinscot River.
Pge	**Esker deposits** - Ridges of sand and gravel deposited by glacial meltwater streams in subglacial tunnels.
Pt	**Till** - Loose to very compact, poorly sorted, massive to weakly stratified mixture of sand, silt, and gravel-size rock debris deposited by glacial ice. Locally includes lenses of waterlaid sand and gravel.
Qst	**Stream terraces** - Sand and gravel terraces in the Androscoggin River and East Branch Nezinscot River valleys. Formed by postglacial erosion and deposition along these rivers.
	Bedrock outcrops / thin-drift areas - Ruled pattern indicates areas where outcrops are common and/or surficial sediments are generally less than 10 ft thick (mapped partly from air photos). Dots show individual outcrops.

2008 USGS SURFICIAL GEOLOGIC MAP

"SURFICIAL GEOLOGY OF THE WORTHLEY POND QUADRANGLE, MAINE"
(THOMPSON, EUSDEN JR., TOLMAN, TUCKER, MARVINNEY)
NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88)
CONTOUR INTERVAL 20 FEET
ORIGINAL SCALE 1:24,000

NORTH

NOT TO SCALE

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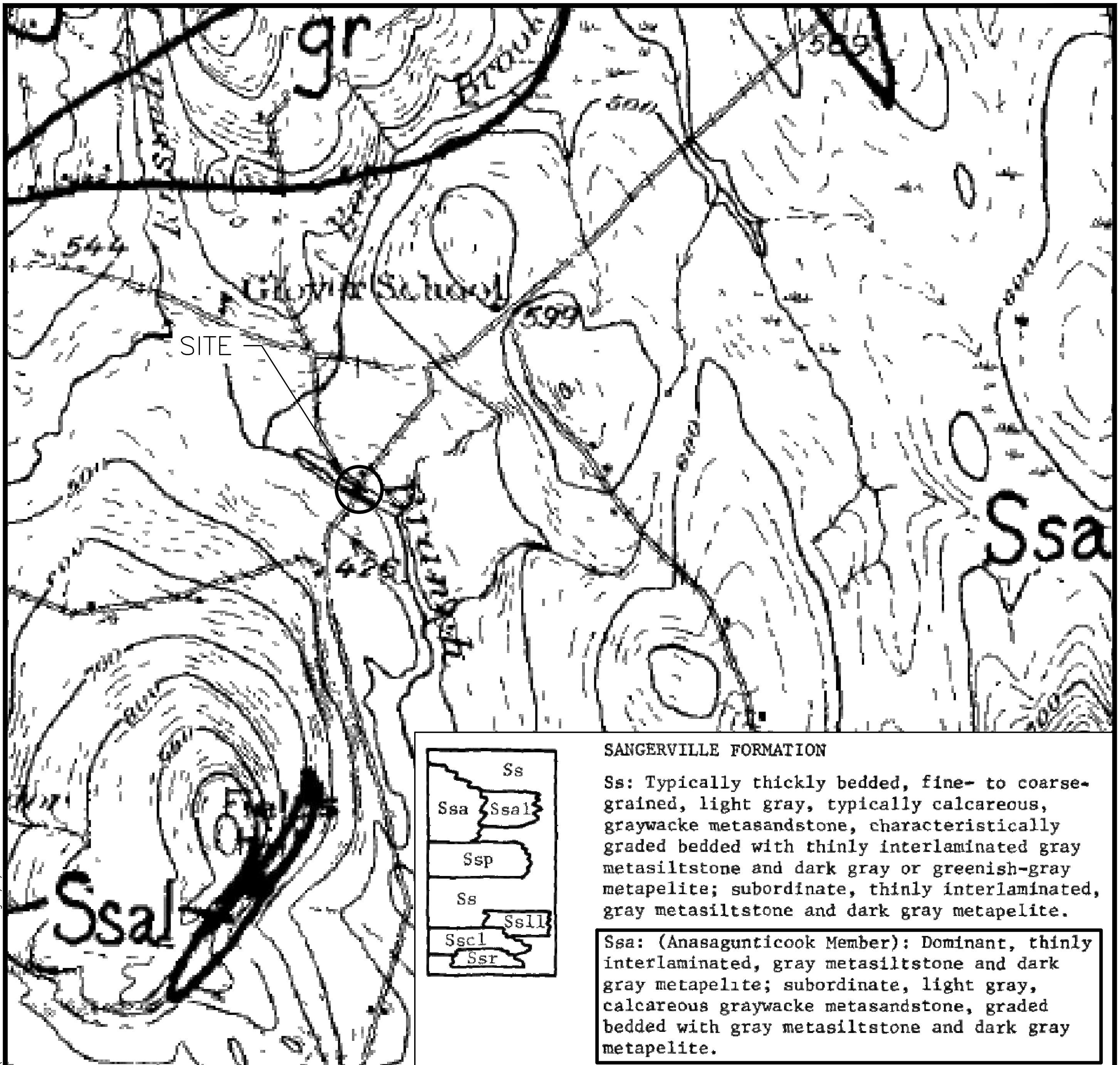
QUADRANGLE LOCATION

FIGURE 3

SURFICIAL GEOLOGIC MAP
FIELDS BRIDGE (#0690)
GAMMON ROAD OVER E. BRANCH
NEZINSCOT RIVER
SUMNER-HARTFORD, MAINE

PROJECT NO. 92270.04 | DATE: JUNE 2018



1978 USGS BEDROCK GEOLOGIC MAP

"RECONNAISSANCE BEDROCK GEOLOGY OF THE BUCKFIELD QUADRANGLE, MAINE"
(PANKIWSKYJ)
ORIGINAL SCALE 1:62,500

NORTH

NOT TO SCALE

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QUADRANGLE LOCATION

FIGURE 4

BEDROCK GEOLOGIC MAP

FIELDS BRIDGE (#0690)
GAMMON ROAD OVER E. BRANCH
NEZINSCOT RIVER
SUMNER-HARTFORD, MAINE

PROJECT NO.	92270.04	DATE:	JUNE 2018

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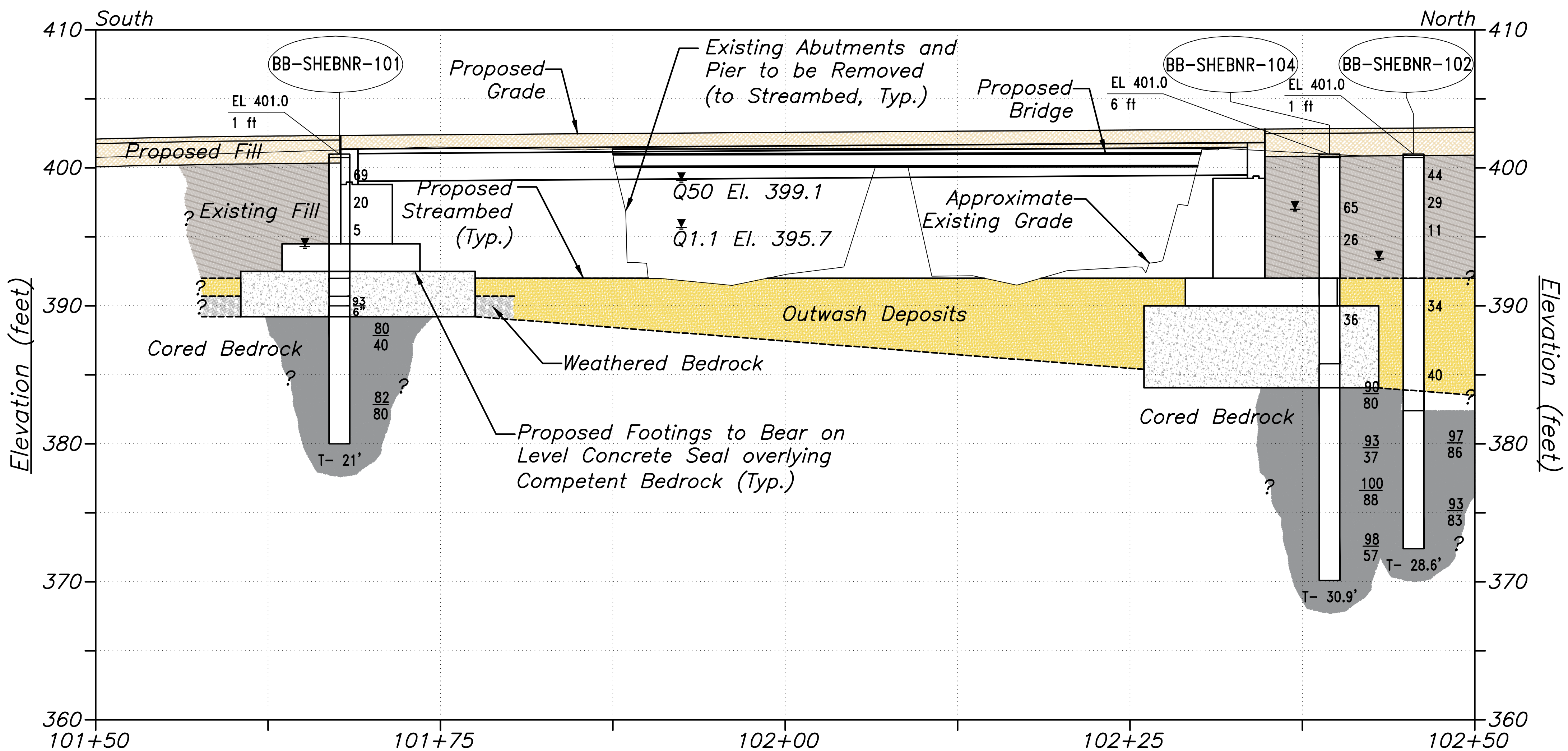


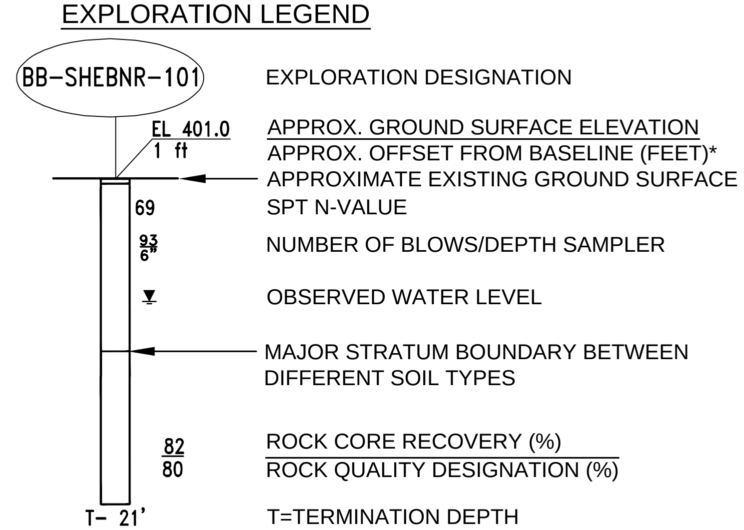
Interpretive Subsurface Profile

Station along Centerline of Proposed Bridge #0690 (feet)

NOTES:

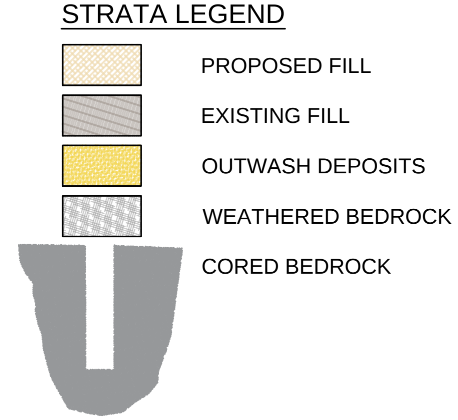
1. THIS SUBSURFACE PROFILE WAS DEVELOPED FROM .DGN FILES PROVIDED BY MAINE DEPARTMENT OF TRANSPORTATION ON MAY 18, 2018.
2. LINES REPRESENTING LIMITS OF STRATA ARE INTERPOLATED FROM SUBSURFACE EXPLORATION LOGS. THE SUBSURFACE EXPLORATIONS ARE WIDELY SPACED AND ARE BASED ON LIMITED SUBSURFACE INFORMATION COLLECTED DURING DRILLING. OTHER INTERPRETATIONS ARE POSSIBLE AND ACTUAL CONDITIONS MAY VARY FROM THOSE PRESENTED.
3. TOP OF BEDROCK SHOULD BE CONSIDERED APPROXIMATE AND CAN VARY SIGNIFICANTLY OVER SHORT DISTANCES.
4. WATER LEVELS PRESENTED WERE COLLECTED DURING DRILLING AND MAY NOT REPRESENT STABILIZED GROUNDWATER CONDITIONS. GROUNDWATER LEVELS WILL FLUCTUATE WITH SEASON, PRECIPITATION, AND NEARBY ACTIVITIES.
5. EXISTING AND PROPOSED SITE FEATURES DEPICTED ARE APPROXIMATE AND GIVEN FOR ILLUSTRATIVE PURPOSES.
6. SOIL BORINGS WERE DRILLED BY NEW ENGLAND BORING CONTRACTORS OF DERRY, NEW HAMPSHIRE, AND OBSERVED BY NOBIS ON JUNE 29 AND DECEMBER 4, 2017.
7. ELEVATIONS ARE PROVIDED IN FEET, AND ARE BASED ON THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88).
8. REFER TO FIGURE 2 FOR SUBSURFACE EXPLORATION LOCATIONS AND A PLAN VIEW OF THE SITE.

EXPLORATION LEGEND



*POSITIVE OFFSETS ARE IN THE WEST DIRECTION.
NEGATIVE OFFSETS ARE IN THE EAST DIRECTION.

STRATA LEGEND



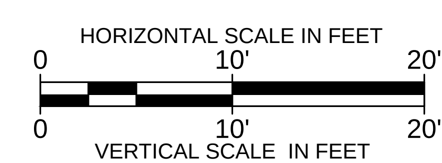


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FIGURE 5

INTERPRETIVE SUBSURFACE PROFILE
FIELDS BRIDGE (#0690)
GAMMON ROAD OVER E. BRANCH
NEZINSCOT RIVER
SUMNER-HARTFORD, MAINE

DRAWN BY:	PCC	CHECKED BY:	BTW
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APPENDIX A – Limitations

GEOTECHNICAL LIMITATIONS

Explorations and Subsurface Conditions

1. The analyses and design recommendations submitted in this report are based in part upon the data obtained from subsurface explorations. The nature and extent of variations between these explorations may not become evident until construction. If variations then appear evident, it will be necessary to reevaluate the recommendations of this report.

   In preparing this report, Nobis relied on certain information provided by the Client and other parties referenced therein which were made available to Nobis at the time of our evaluation. Nobis did not attempt to independently verify the accuracy or completeness of all information reviewed or received during the course of this evaluation.

2. The generalized soil profile described in the text is intended to convey trends in subsurface conditions. The boundaries between strata are approximate and idealized and have been developed by interpretations of widely spaced explorations and samples; actual soil transitions are probably more erratic. For specific information, refer to the exploration logs.

3. Water level readings have been made in the explorations at times and under conditions stated on the logs. These data have been reviewed and interpretations have been made in the text of this report. However, it must be noted that fluctuations in the level of the groundwater may occur due to variations in rainfall, temperature, and other factors occurring since the time measurements were made. The water table encountered in the course of the work may differ from that indicated in the Report.

   Recommendations for foundation drainage, waterproofing, and moisture control address the conventional geotechnical engineering aspects of seepage control. These recommendations may not preclude an environment that allows the infestation of mold or other biological pollutants.

4. Nobis' geotechnical services did not include an assessment of the presence of oil or hazardous materials at the property. Consequently, we did not consider the potential impacts (if any) that contaminants in soil or groundwater may have on construction activities, or the use of structures on the property.

Additional Services

5. Nobis recommends that we be retained to provide services during future site observations, design, implementation activities, construction and/or property development/ redevelopment. This will allow us the opportunity to: i) observe conditions and compliance with our recommendations, design concepts and/or opinions; ii) allow for changes in the event that conditions are other than anticipated; iii) provide modifications to our design recommendations; and iv) assess the consequences of changes in technologies and/or regulations.

Use of Report

6. Nobis prepared this report on behalf of, and for the exclusive use of our Client for the stated purpose(s) and location(s) identified in our proposal and/or report. Use of this report, in whole or in part, at other locations, or for other purposes, may lead to inappropriate conclusions; and we do not accept any responsibility for the consequences of such use(s). Reliance by any party not expressly identified in the agreement, for any use, without our prior written permission, shall be at that party's sole risk, and without any liability to Nobis.

   This report is for design purposes only and is not sufficient to prepare an accurate construction bid. Contractors wishing a copy of the report may secure it with the understanding that its scope is limited to design considerations only.

7. Nobis' findings and conclusions are based on the work conducted as part of the scope of work set forth in our proposal and/or report, and reflect our professional judgment. These findings and conclusions must be considered not as scientific or engineering certainties, but rather as our professional opinions considering the limited data gathered during the course of our work. If conditions other than those described in this report are found at the subject location(s), or the project design has been altered in any way, Nobis shall be so notified and afforded the opportunity to revise the report, as appropriate, to reflect the unanticipated changed conditions.

8. Nobis' services were performed using the degree of skill and care ordinarily exercised by qualified professionals performing the same type of services, at the same time, under similar conditions, at the same or a similar property. No warranty, expressed or implied, is made.

Compliance with Codes and Regulations

9. Nobis used reasonable care in identifying and interpreting applicable codes and regulations. These codes and regulations are subject to various, and possibly contradictory, interpretations. Compliance with codes and regulations by other parties is beyond our control.

Opinion of Cost

10. This report may contain or be based on comparative cost opinions for the purpose of evaluating alternative foundation schemes. These opinions may also involve approximate quantity evaluations. It should be noted that quantity estimates may not be accurate enough for construction bids. In addition, since we are not professional estimators of labor and materials cost, the evaluation of construction costs should be considered as approximate guidelines and could vary significantly from actual costs. Nobis does not guarantee the accuracy of our cost opinions as compared to contractor's bids for construction costs.

END OF LIMITATIONS

APPENDIX B – Boring Logs

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS	Project: Fields Bridge (#0690) Gammon Road over East Branch Nezinscot River Location: Sumner-Hartford, Maine	Boring No.: BB-SHEBNR-101 WIN: 21705.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	401	**Auger ID/OD:**	N/A
Operator:	M. Porter	**Datum:**	NAVD-88	**Sampler:**	1-3/8" Split-Spoon
Logged By:	P. Clarke (Nobis)	**Rig Type:**	B-53 Mobile ATV	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	June 29, 2017/June 29, 2017	**Drilling Method:**	Drive and Wash	**Core Barrel:**	NQ2
Boring Location:	STA 101+68, 1' LT	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	6.7' bgs
Hammer Efficiency Factor:	.75	**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	1D	24/12	0.50 - 2.50	15/25/30/21	55	69	3	400.75		Asphalt (3"). 0.25	
							5			Brown, dry, very dense, sandy fine to coarse GRAVEL, trace silt, few asphalt particles and fragments. (Fill).	
	2D	24/16	2.50 - 4.50	12/10/6/10	16	20	6			Brown, dry, very dense, fine to medium SAND, some fine to coarse gravel, little silt, very few asphalt particles. (Fill).	A-2-4 (0), SM
							7				
	3D	24/8	4.50 - 6.50	4/2/2/3	4	5	27			Brown, dry, very loose, fine to coarse SAND, some fine gravel, little silt, very few roots. (Fill).	
5							29				
							20				
							29				
							34	392.00		9.00	
							64				
10	4D	12/6	10.00 - 11.00	18/43/50/0"				390.67		4D-A (4"): Light brown, wet, fine to coarse SAND, little fine gravel, little silt (Outwash Deposit). 10.33	
	R1	60/48	11.00 - 16.00	RQD = 40%			NQ2	390.00		4D-B (2"): Grey, wet, WEATHERED BEDROCK particles and fragments. (Weathered Bedrock). 11.00	
										Top of Bedrock Elevation at El. 390. R1: Grey, fine to medium-grained METASANDSTONE/ METASILTSTONE, hard to very hard, fresh, low-angle to moderately-dipping, very close to moderately close joints. Sangerville Formation (Anasagunticook Member). RQD = 40% (Poor). R1: Core Times (min:sec) 11'-12' (1:40) 12'-13' (1:20) 13'-14' (1:30) 14'-15' (1:20) 15'-16' (1:00)	
15											
	R2	60/49	16.00 - 21.00	RQD = 80%				385.00		16.00 R2: Grey, fine to medium-grained METASANDSTONE/ METASILTSTONE, hard to very hard, low-angle to moderately-dipping, close to moderately close joints. Sangerville Formation (Anasagunticook Member). RQD = 80% (Good). R2: Core Times (min:sec) 16'-17' (1:10) 17'-18' (1:10) 18'-19' (1:10) 19'-20' (1:10) 20'-21' (1:30)	UCT q_p= 12,268 psi
20								380.00		21.00	
										Bottom of Exploration at 21.00 feet below ground surface.	
25											

Remarks:

-Borehole backfilled with native soils.
-Pavement restored with asphalt cold patch.
-bgs = below ground surface.
-Automatic Hammer ID# NEBC 1.

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:** Fields Bridge (#0690) Gammon Road over East Branch Nezinscot River **Location:** Sumner-Hartford, Maine	**Boring No.:** BB-SHEBNR-102 **WIN:** 21705.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	401	**Auger ID/OD:**	N/A
Operator:	M. Porter	**Datum:**	NAVD-88	**Sampler:**	1-3/8" Split-Spoon
Logged By:	P. Clarke (Nobis)	**Rig Type:**	B-53 Mobile ATV	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	June 29, 2017/June 29, 2017	**Drilling Method:**	Drive and Wash	**Core Barrel:**	NQ2
Boring Location:	STA 102+46, 1' LT	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	7.6' bgs
Hammer Efficiency Factor:	.75	**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	1D	24/12	0.50 - 2.50	15/14/21/33	35	44		400.75		Asphalt (3"). 0.25 Brown, dry, dense, fine to coarse SAND, some fine to coarse gravel, trace silt, few asphalt particles and fragments. (Fill).	
	2D	24/16	2.50 - 4.50	21/12/11/10	23	29	3			Brown, dry, medium dense, fine to coarse SAND, little fine to coarse gravel, trace silt, very few asphalt particles. (Fill).	
							15				
	3D	24/14	4.50 - 6.50	5/6/3/1	9	11	28			3D-A (11"): Orange-brown, dry, loose, fine to coarse SAND, some fine gravel, trace silt. (Fill).	
5							27			3D-B (3"): Dark brown with red staining, moist, loose, fine to coarse SAND, some silt, trace fine gravel, very few roots. (Fill).	
							9				
							12				
							15				
							70	392.00		9.00	
10	4D	24/5	10.00 - 12.00	18/17/10/11	27	34	59			Tan, wet, medium dense, fine to coarse SAND, some fine gravel, trace silt. (Outwash Deposit).	
							45				
							51				
							89				
							66				
15	5D	24/7	15.00 - 17.00	8/13/19/17	32	40				Light brown, wet, dense, gravelly fine to coarse SAND, little silt. (Outwash Deposit).	A-1-b (0), SP-SM
	R1	60/58	18.60 - 23.60	RQD = 86%			NQ2	382.40		18.60 Top of Bedrock Elevation at El. 382.4. R1: Grey, fine to medium-grained METASANDSTONE/ METASILTSTONE, hard to very hard, fresh, low-angle to moderately-dipping, very close to moderately close joints. Sangerville Formation (Anasagunticook Member). RQD = 86% (Good). R1: Core Times (min:sec) 18.6'-19.6' (1:20) 19.6'-20.6' (1:30) 20.6'-21.6' (1:40) 21.6'-22.6' (1:30) 22.6'-23.6' (2:00)	UCT q_p= 25,755 psi
20											
	R2	60/56	23.60 - 28.60	RQD = 83%						R2: Grey, fine to medium-grained METASANDSTONE/ METASILTSTONE with Quartzite intrusions, hard to very hard,	
25											

Remarks:

-Borehole backfilled with native soils.
-Pavement restored with asphalt cold patch.
-bgs = below ground surface.
-Automatic Hammer ID# NEBC 1.

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: Fields Bridge (#0690) Gammon Road over East Branch Nezinscot River Location: Sumner-Hartford, Maine	Boring No.: BB-SHEBNR-102 WIN: 21705.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	401	**Auger ID/OD:**	N/A
Operator:	M. Porter	**Datum:**	NAVD-88	**Sampler:**	1-3/8" Split-Spoon
Logged By:	P. Clarke (Nobis)	**Rig Type:**	B-53 Mobile ATV	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	June 29, 2017/June 29, 2017	**Drilling Method:**	Drive and Wash	**Core Barrel:**	NQ2
Boring Location:	STA 102+46, 1' LT	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	7.6' bgs
Hammer Efficiency Factor:	.75	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25								372.40		fresh, low-angle to moderately-dipping, very close to moderately close joints. Sangerville Formation (Anasagunticook Member). RQD = 83% (Good). R2: Core Times (min:sec) 23.6'-24.6' (6:30) 24.6'-25.6' (2:00) 25.6'-26.6' (2:00) 26.6'-27.6' (2:00) 27.6'-28.6' (2:30) 28.60 **Bottom of Exploration at 28.60 feet below ground surface.**	
30											
35											
40											
45											
50											

Remarks:

-Borehole backfilled with native soils.
-Pavement restored with asphalt cold patch.
-bgs = below ground surface.
-Automatic Hammer ID# NEBC 1.

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-SHEBNR-102

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Fields Bridge (#0690) Gammon Road over East Branch Nezinscot River
Location: Sumner-Hartford, Maine

Boring No.: BB-SHEBNR-103
WIN: 21705.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	401	**Auger ID/OD:**	N/A
Operator:	M. Porter	**Datum:**	NAVD-88	**Sampler:**	1-3/8" Split-Spoon
Logged By:	K. Kocia (Nobis)	**Rig Type:**	B-59 Mobile Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	December 4, 2017/December 4, 2017	**Drilling Method:**	Auger	**Core Barrel:**	NQ2
Boring Location:	STA 102+37, 6' LT	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	N/A
Hammer Efficiency Factor:	0.869	**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							RC	400.75		Asphalt (3"). 0.25	
	1D	24/11	1.00 - 3.00	6/6/6/10	12	17				Brown-grey, dry, medium dense, gravelly fine to coarse SAND, trace silt. (Fill).	
								398.00		3.00	
	2D	9/9	3.00 - 3.75	17/50/3"				397.25		Light grey, dry, CONCRETE particles and fragments, little fine to coarse sand, trace silt. (Fill). 3.75	
										Bottom of Exploration at 3.75 feet below ground surface.	
5											
10											
15											
20											
25											

Remarks:

-Borehole backfilled with gravel (1 bag) and native soils.
-Pavement restored with asphalt cold patch.
-bgs = below ground surface.
-Automatic Hammer ID# B-24.

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:	Fields Bridge (#0690) Gammon Road over East Branch Nezinscot River	**Boring No.:**	BB-SHEBNR-104		
Location:	Sumner-Hartford, Maine	**WIN:**	21705.00		
Driller:	New England Boring Contractors	**Elevation (ft.)**	401	**Auger ID/OD:**	N/A
Operator:	M. Porter	**Datum:**	NAVD-88	**Sampler:**	1-3/8" Split-Spoon
Logged By:	K. Kocia (Nobis)	**Rig Type:**	B-59 Mobile Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	December 4, 2017/December 4, 2017	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NQ2
Boring Location:	STA 102+39, 6' LT	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	4' bgs
Hammer Efficiency Factor:	0.869	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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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							RC	400.75		Asphalt (3"). 0.25	
	1D	21/11	3.00 - 4.75	2/17/28/50/3"	45	65				1D-A (5"): Brown, dry, very dense, fine to coarse SAND, little fine gravel, trace silt, few asphalt particles and fragments, very few roots. (Fill). 1D-B (6"): Light grey-brown, wet, very dense, fine to coarse GRAVEL, some fine to coarse sand, trace silt. (Fill).	
5	2D	24/4	5.20 - 7.20	75/9/9/10	18	26				Brown-dark grey, wet, medium dense, fine to coarse SAND, some silt, little fine gravel, very few metal particles and fragments, very few roots. (Fill).	
								392.00		9.00 Change in resistance in Roller Cone at 9 feet bgs, color change from brown to tan in wash.	
10	3D	24/8	11.00 - 13.00	15/12/13/13	25	36				Grey-brown, wet, dense, fine to coarse GRAVEL, some fine to coarse sand, trace silt. (Outwash Deposit).	A-1-a, GP
15	R1	60/54	15.20 - 20.20	RQD = 80%			NQ2	385.80		15.20 Top of Bedrock Elevation at El. 385.8. R1: Grey, fine to medium-grained METASANDSTONE/ METASILTSTONE, medium hard to very hard, fresh, low-angle to 45 degree angle-dipping, very close to moderately close joints. Sangerville Formation (Anasagunticook Member). RQD = 80% (Good). R1: Core Times (min:sec) 15.2'-16.2' (3:00) 16.2'-17.2' (1:30) 17.2'-18.2' (2:00) 18.2'-19.2' (2:00) 19.2'-20.2' (1:30)	
20	R2	30/28	20.20 - 22.70	RQD = 37%				380.80		20.20 R2: Light grey, medium to coarse-grained, QUARTZITE/GRANITE/ MICA, medium hard to hard, fresh to slightly weathered, horizontal to low-angle-dipping, close to moderately close joints. (Agglomerate Granite). RQD = 37% (Poor). R2 Core Times (min:sec) 20.2'-21.2' (2:00) 21.2'-22.2' (1:45)	UCT q_p= 18,185 psi
	R3	42/42	22.70 - 26.20	RQD = 88%							
25											

Remarks:

-Borehole backfilled with gravel (3 bags) and native soils.
-Pavement restored with asphalt cold patch.
-bgs = below ground surface.
-Automatic Hammer ID# B-24.

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:** Fields Bridge (#0690) Gammon Road over East Branch Nezinscot River **Location:** Sumner-Hartford, Maine	**Boring No.:** BB-SHEBNR-104 **WIN:** 21705.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	401	**Auger ID/OD:**	N/A
Operator:	M. Porter	**Datum:**	NAVD-88	**Sampler:**	1-3/8" Split-Spoon
Logged By:	K. Kocia (Nobis)	**Rig Type:**	B-59 Mobile Truck	**Hammer Wt./Fall:**	140#/30"
Date Start/Finish:	December 4, 2017/December 4, 2017	**Drilling Method:**	Drive & Wash	**Core Barrel:**	NQ2
Boring Location:	STA 102+39, 6' LT	**Casing ID/OD:**	4"/4.5"	**Water Level*:**	4' bgs
Hammer Efficiency Factor:	0.869	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
25										22.2'-22.7' (1:30) R3: Light grey, medium to coarse-grained, GRANITE/MICA, medium hard to very hard, fresh, horizontal -dipping, very close to moderately close joints. (Agglomerate Granite). RQD = 88% (Good). R3: Core Times (min:sec) 22.7'-23.7' (1:30) 23.7'-24.7' (1:45) 24.7'-25.7' (2:00) 25.7'-26.2' (1:00)	
	R4	56/55	26.20 - 30.87	RQD = 57%						R4: Light grey, medium to coarse-grained, QUARTZITE/GRANITE/ MICA, medium hard to very hard, fresh to slightly weathered, horizontal to low-angle-dipping,very close to moderately close joints. (Agglomerate Granite). RQD = 57% (Fair). R4: Core Times (min:sec) 26.2'-27.2' (2:00) 27.2'-28.2' (2:00) 28.2'-29.2' (1:30) 29.2'-30.2' (2:00) 30.2'-30.9' (1:00)	
30								370.10		30.90	
										Bottom of Exploration at 30.90 feet below ground surface.	
35											
40											
45											
50											

Remarks:

-Borehole backfilled with gravel (3 bags) and native soils.
-Pavement restored with asphalt cold patch.
-bgs = below ground surface.
-Automatic Hammer ID# B-24.

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-SHEBNR-104

APPENDIX C – Automatic Hammer Calibration Report

Summary of Energy Measurements - Chelsea, Massachusettes

Test Boring No.	Drill Rig	Type of Test Hammer Type	Sample No.	Sample Depth top bottom	SPT Blows per 6" from field logs				N-Value	Distance to bottom of sampler from center of instrumented rod (feet)	Rated Energy (ft.-lbs.)	Average Transferred Energy (2) (ft.-lbs.)	Average Transfer Efficiency (2) (%)	Average Hammer Blow Rate (2) (blows/min.)
Test Boring 7-13-2017	Truck 83 Mobile B-59 International USDOT 383455 LIC#: 4368	Mobile 140 lb. Automatic Hammer	S1	17' 19'	8	9	9	10	18	21'	350	305	87.1	55.7
		Mobile 140 lb. Automatic Hammer	S2	10' 12'	4	7	9	11	16	14'	350	298	85.2	57.0
		Mobile 140 lb. Automatic Hammer	S3	12' 14'	15	14	14	15	28	16'	350	310	88.5	57.0

NOTES: (1) Driller of Test Boring : Jerry Rednicki - New England Boring Contractors
(2) Averaged only for impacts during the middle 1 ft. of the test which relates to the observed N-Value

Average Transfer Efficiency = (87.1% + 85.2% + 88.5%)/3 = 86.9%

Use hammer efficiency factor of 0.869

New England Boring Contractors (NEBC) – ETR for MaineDOT Projects
based on March 27 2017 Calibration

RIG B53 Track							
Hammer #NEBC 1							
	Average E over all blows	72.2	78.3	71.2	75.5		
	N (Ni)	19	40	26	37	∑Ni	122
	Average E over N (Ei)	71.8	78.5	71.2	75.5		
	NiEi	1364.9	3139.0	1851.4	2793.5	∑NiEi	9149
						∑NiEi/∑Ni	**75.0**

RIG B53 Rubber Track							
Hammer #NEBC 2							
	Average E over all blows	66.5	68.6	67.2	68.5		
	N (Ni)	30	26	16	23	∑Ni	95
	Average E over N (Ei)	66.9	68.8	66.9	68.8		
	NiEi	2008.2	1788.1	1070.8	1582.0	∑NiEi	6449
						∑NiEi/∑Ni	**67.7**

RIG CME Trailer							
Hammer #MTB AH3							
	Average E over all blows	68.0	70.0	70.5	66.5		
	N (Ni)	20	31	21	25	∑Ni	97
	Average E over N (Ei)	68.3	69.4	70.4	65.4		
	NiEi	1365.2	2152.4	1478.0	1636.2	∑NiEi	6632
						∑NiEi/∑Ni	**68.4**

APPENDIX D – Photo Logs of Rock Core Samples

92270.04 Fields Bridge (#0690) – Sumner-Hartford, ME – Boring BB-SHEBNR-101 Rock Core Photo Log – June 29, 2017

Photo 1: Boring BB-SHEBNR-101 Rock Core Sample R-1 (Row 1, Top)

Photo 2: Boring BB-SHEBNR-101 Rock Core Sample R-1 (Row 1, Bottom)

Photo 3: Boring BB-SHEBNR-101 Rock Core Sample R-2 (Row 2, Top)

Photo 4: Boring BB-SHEBNR-101 Rock Core Sample R-2 (Row 2, Bottom)

92270.04 Fields Bridge (#0690) – Sumner-Hartford, ME – Boring BB-SHEBNR-102 Rock Core Photo Log – June 29, 2017

Photo 1: Boring BB-SHEBNR-102 Rock Core Sample R-1 (Row 3)

Photo 2: Boring BB-SHEBNR-102 Rock Core Sample R-2 (Row 4, Top)

Photo 3: Boring BB-SHEBNR-102 Rock Core Sample R-2 (Row 2, Bottom

92270.04 Fields Bridge (#0690) – Sumner-Hartford, ME – Boring BB-SHEBNR-104 Rock Core Photo Log – December 4, 2017

Photo 1: Boring BB-SHEBNR-104 Rock Core Sample R-1 (Row 1)

Photo 2: Boring BB-SHEBNR-104 Rock Core Sample R-2 (Row 2)

Photo 3: Boring BB-SHEBNR-104 Rock Core Sample R-3 (Row 3)

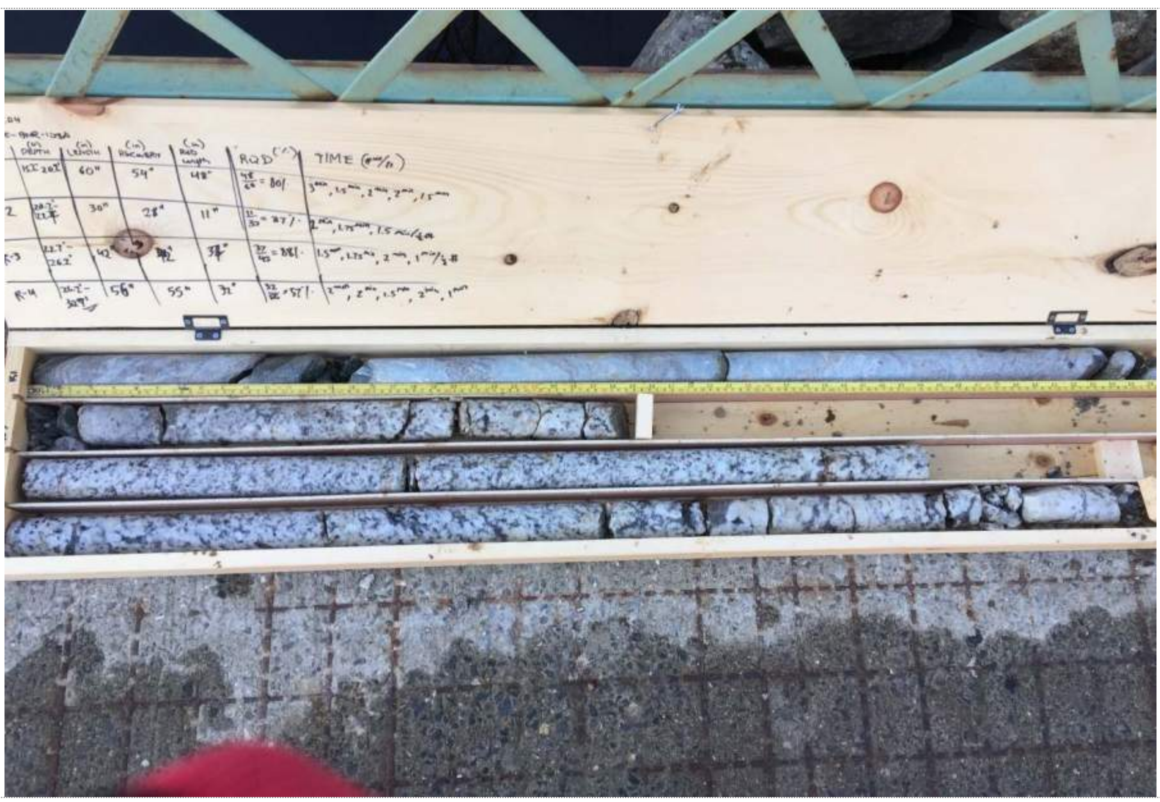


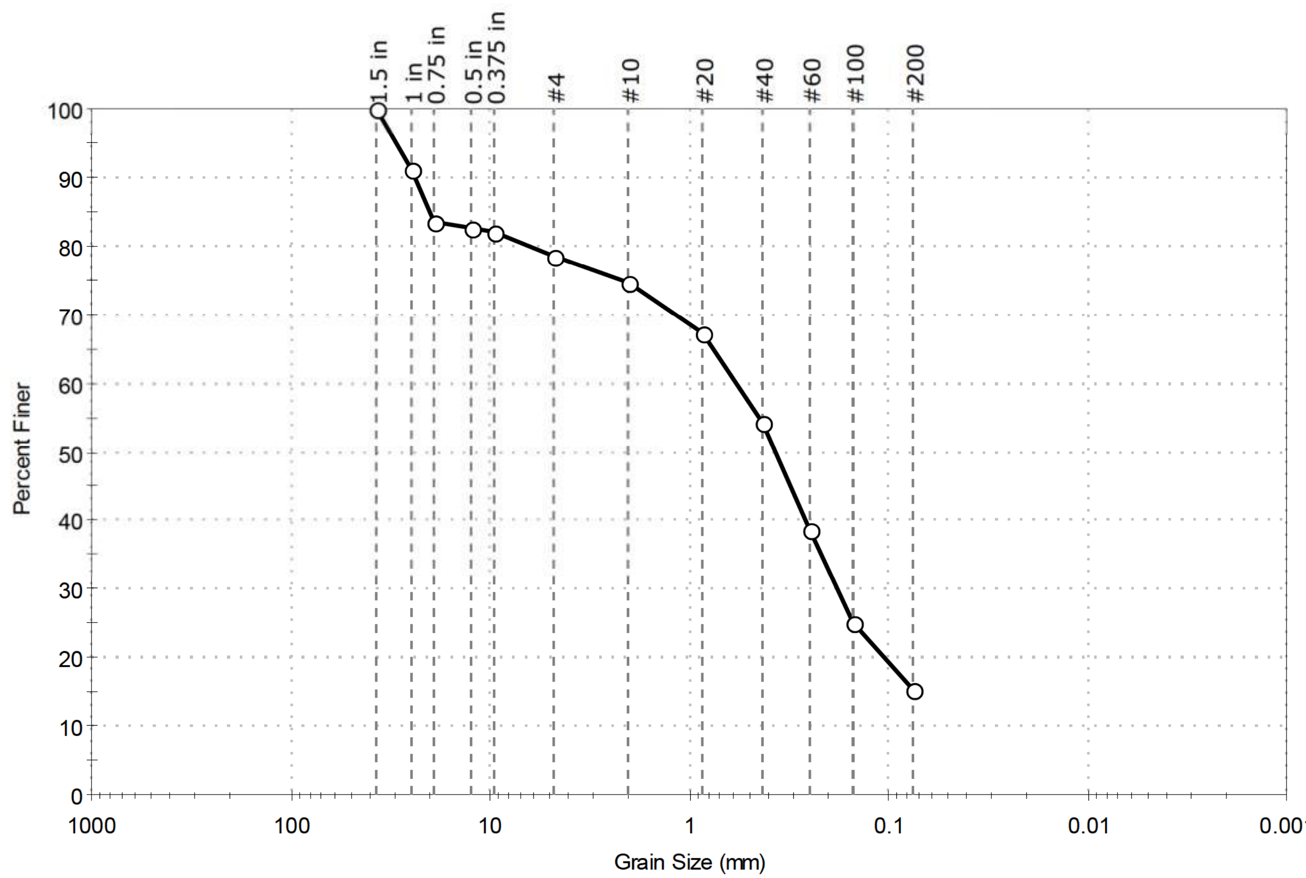
Photo 4: Boring BB-SHEBNR-104 Rock Core Sample R-4 (Row 4)

APPENDIX E – Laboratory Test Results

GeoTesting EXPRESS

Client:	Nobis Engineering, Inc.				
Project:	Fields Bridge #0690				
Location:	Sumner-Hartford, ME			Project No:	GTX-306742
Boring ID:	BB-SHEBNR-101	Sample Type:	bag	Tested By:	jbr
Sample ID:	S-2	Test Date:	07/31/17	Checked By:	jsc
Depth :	2.5-4.5 ft	Test Id:	417888		
Test Comment:	---				
Visual Description:	Moist, brown silty sand with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	21.5	63.0	15.5

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	91		
0.75 in	19.00	84		
0.5 in	12.50	83		
0.375 in	9.50	82		
#4	4.75	78		
#10	2.00	74		
#20	0.85	67		
#40	0.42	54		
#60	0.25	39		
#100	0.15	25		
#200	0.075	15		

Coefficients

D_{85} = 19.9885 mm
D_{30} = 0.1805 mm
D_{60} = 0.5774 mm
D_{15} = N/A
D_{50} = 0.3683 mm
D_{10} = N/A
C_u = N/A
C_c = N/A

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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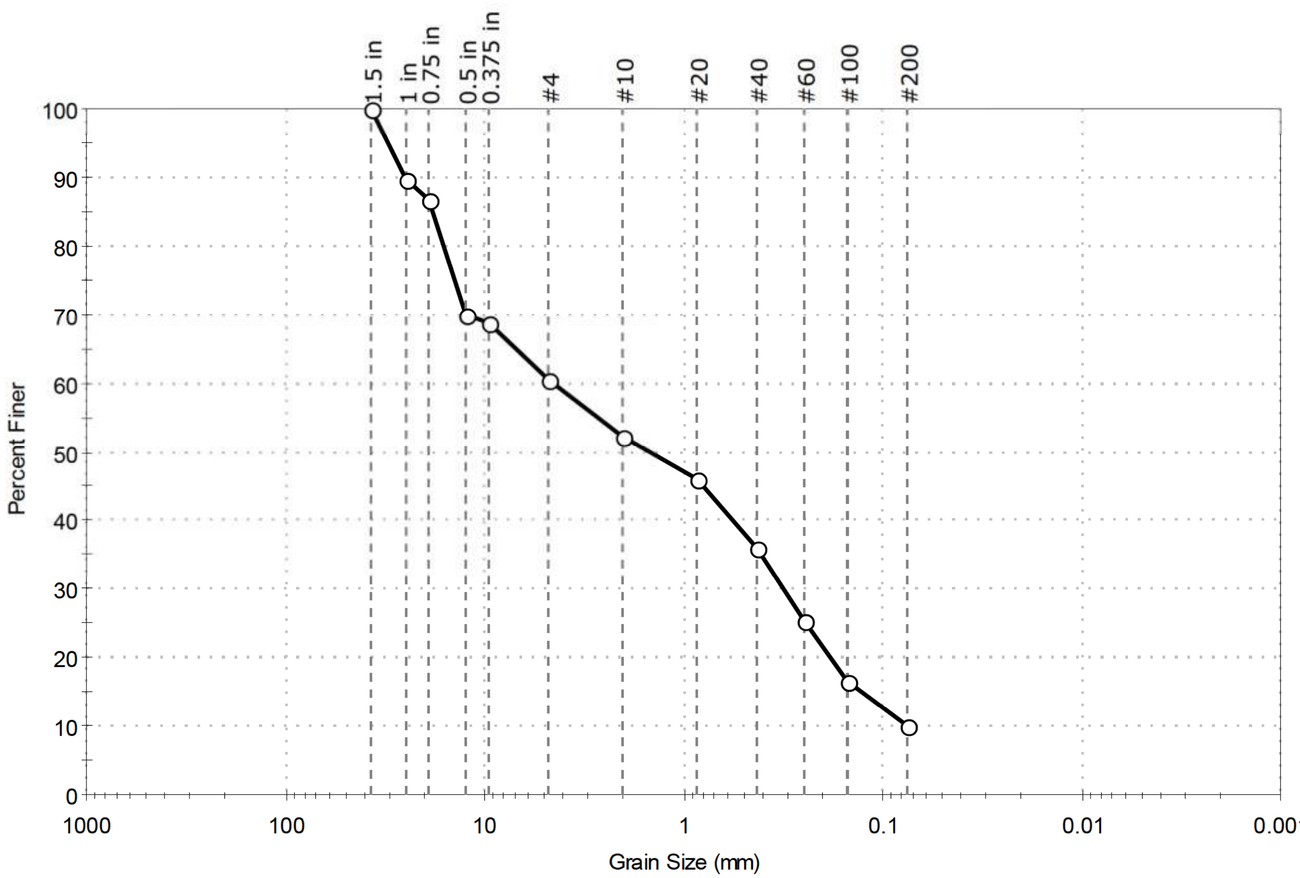
Client:	Nobis Engineering, Inc.				
Project:	Fields Bridge #0690				
Location:	Sumner-Hartford, ME			Project No:	GTX-306742
Boring ID:	BB-SHEBNR-102	Sample Type:	bag	Tested By:	jbr
Sample ID:	S-5	Test Date:	07/31/17	Checked By:	jsc
Depth :	15-17 ft	Test Id:	417889		
Test Comment:	---				
Visual Description:	Moist, brown silty sand with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	39.6	50.3	10.1

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	87		
0.5 in	12.50	70		
0.375 in	9.50	69		
#4	4.75	60		
#10	2.00	52		
#20	0.85	46		
#40	0.42	36		
#60	0.25	25		
#100	0.15	17		
#200	0.075	10		

Coefficients

D_{85} =18.2524 mm	D_{30} =0.3145 mm
D_{60} =4.5604 mm	D_{15} =0.1273 mm
D_{50} =1.4623 mm	D_{10} =N/A
C_u =N/A	C_c =N/A

Classification

ASTM N/A

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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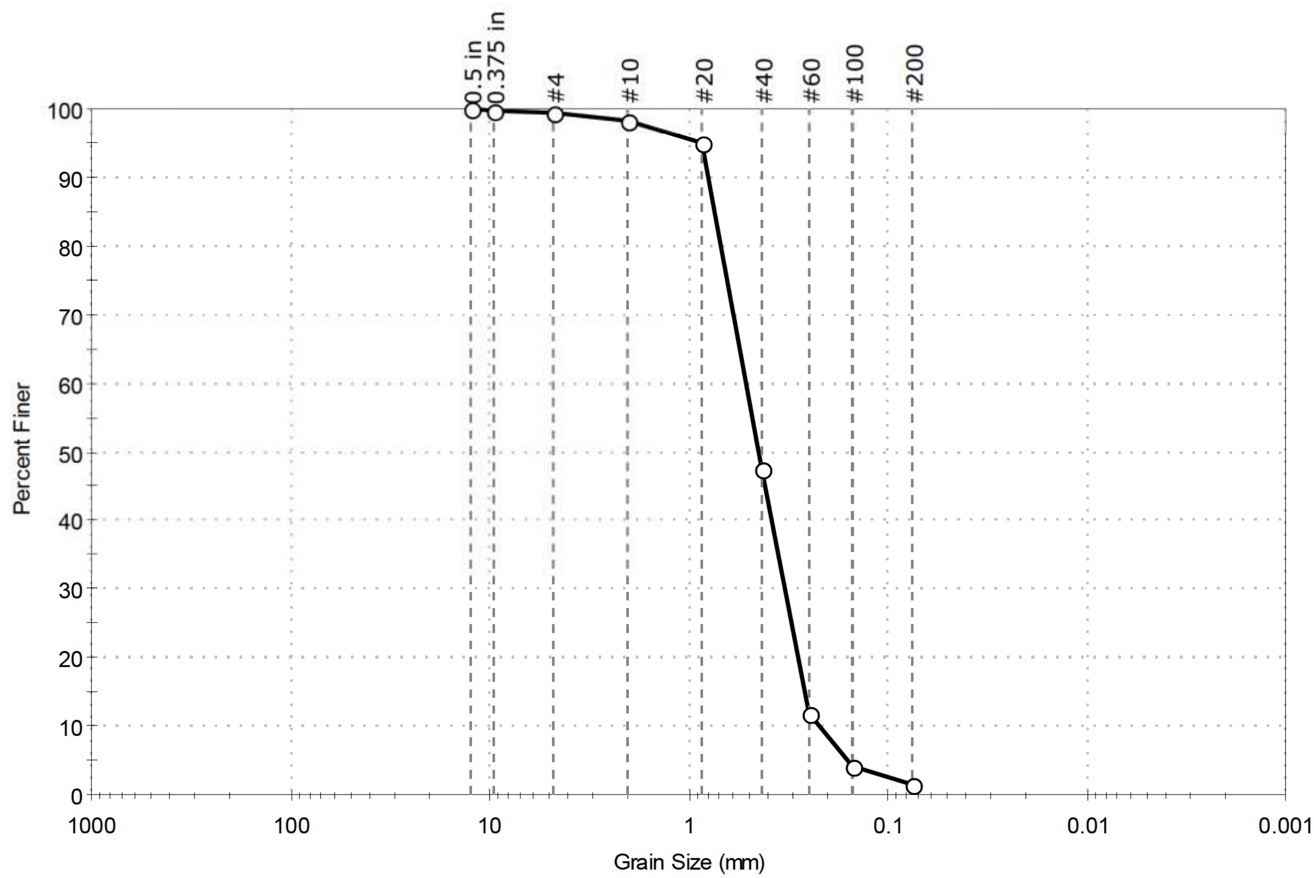
GeoTesting EXPRESS

Client:	Nobis Engineering, Inc.		
Project:	Fields Bridge #0690		
Location:	Sumner-Hartford, ME	Project No:	GTX-306742
Boring ID:	SHEBNR-Grab	Sample Type: bag	Tested By: jbr
Sample ID:	G-1	Test Date: 07/31/17	Checked By: jsc
Depth :	0-0.5 ft	Test Id: 417886	
Test Comment:	---		
Visual Description:	Moist, dark brown sand		
Sample Comment:	Sample contains organics		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	0.6	98.0	1.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.5 in	12.50	100		
0.375 in	9.50	100		
#4	4.75	99		
#10	2.00	98		
#20	0.85	95		
#40	0.42	48		
#60	0.25	12		
#100	0.15	4		
#200	0.075	1.4		

Coefficients

D_{85} =0.7353 mm
D_{30} =0.3273 mm
D_{60} =0.5097 mm
D_{15} =0.2620 mm
D_{50} =0.4402 mm
D_{10} =0.2216 mm
C_u =2.300
C_c =0.948

Classification

ASTM Poorly graded sand (SP)

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

Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : SOFT

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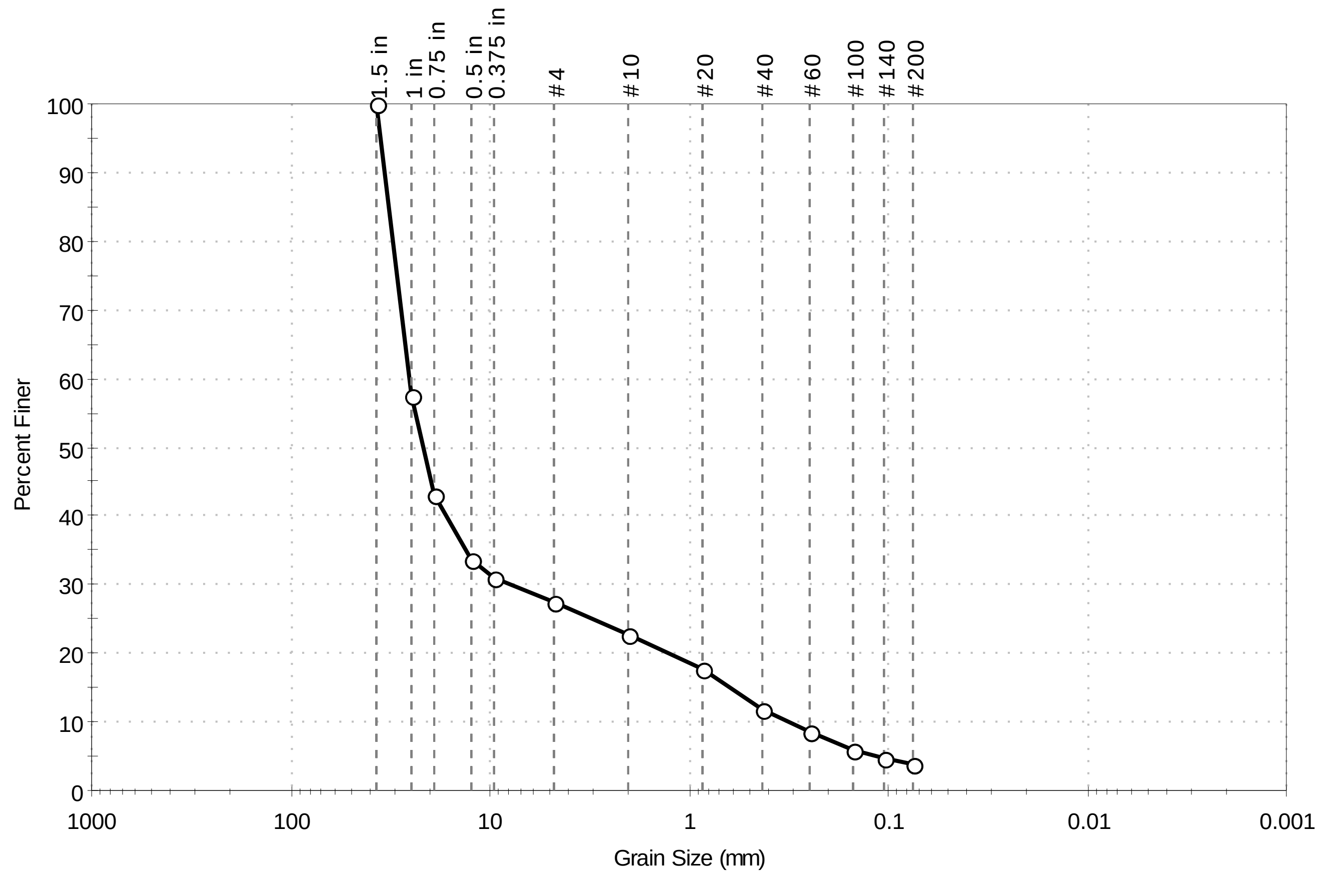
Client:	Nobis Engineering, Inc.				
Project:	Fields Bridge (#0690) Over East Branch Nezinscot R				
Location:	Sumner - Hartford, ME			Project No:	GTX-307433
Boring ID:	BB-SHEBNR-104	Sample Type:	jar	Tested By:	jbr
Sample ID:	3D	Test Date:	12/19/17	Checked By:	jsc
Depth :	11-13 ft	Test Id:	436834		
Test Comment:	---				
Visual Description:	Moist, grayish brown gravel with sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	72.7	23.5	3.8

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1.5 in	37.50	100		
1 in	25.00	58		
0.75 in	19.00	43		
0.5 in	12.50	34		
0.375 in	9.50	31		
#4	4.75	27		
#10	2.00	23		
#20	0.85	18		
#40	0.42	12		
#60	0.25	8		
#100	0.15	6		
#140	0.11	5		
#200	0.075	3.8		

Coefficients

D_{85} = 32.4976 mm D_{30} = 7.7341 mm

D_{60} = 25.5986 mm D_{15} = 0.6266 mm

D_{50} = 21.6452 mm D_{10} = 0.3203 mm

C_u = 79.921 C_c = 7.295

Classification

ASTM Poorly graded GRAVEL with Sand (GP)

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

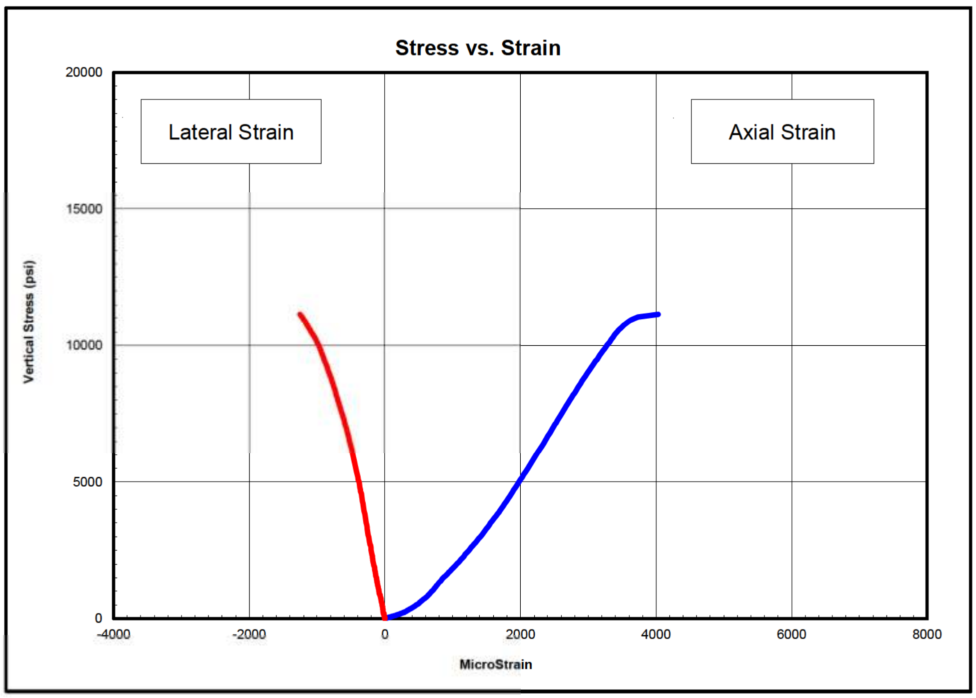
Sample/Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	Nobis Engineering, Inc.
Project Name:	Fields Bridge #0690
Project Location:	Sumner-Hartford, ME
GTX #:	306742
Test Date:	7/27/2017
Tested By:	rlc
Checked By:	jsc
Boring ID:	BB-SHEBNR-101
Sample ID:	C-2
Depth, ft:	16.33-16.69
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

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



Peak Compressive Stress: 12,268 psi

One lateral strain gauge failed to record meaningful data. Poisson's Ratio reported based on results of a single lateral strain gauge.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1200-4500	2,970,000	0.22
4500-7800	3,980,000	0.39
7800-11000	3,430,000	---

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

GeoTesting EXPRESS

Client:	Nobis Engineering, Inc.	Test Date:	7/24/2017
Project Name:	Fields Bridge #0690	Tested By:	rlc
Project Location:	Sumner-Hartford, ME	Checked By:	jsc
GTX #:	306742		
Boring ID:	BB-SHEBNR-101		
Sample ID:	C-2		
Depth:	16.33-16.69 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:	4.21	4.21	4.21		
Specimen Diameter, in:	1.99	1.99	1.99		
Specimen Mass, g:	572.68				
Bulk Density, lb/ft^3	166			**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

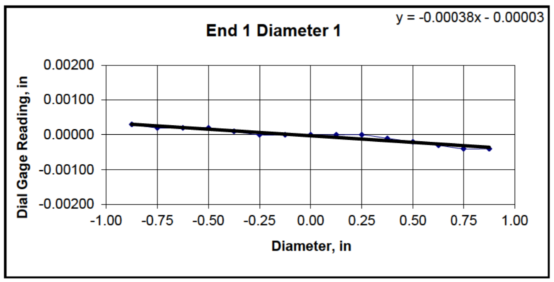
END FLATNESS AND PARALLELISM (Procedure FP1)

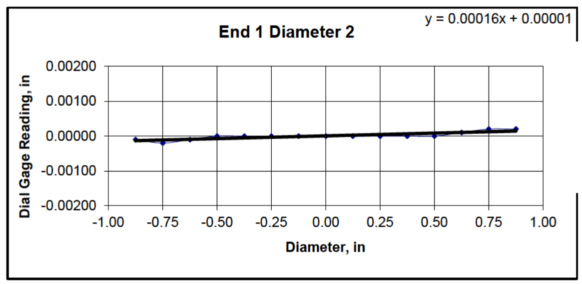
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00030	0.00020	0.00020	0.00020	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00030	-0.00040	-0.00040
Diameter 2, in (rotated 90^o)	-0.00010	-0.00020	-0.00010	0.00000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

Difference between max and min readings, in:
0° = 0.00070 90° = 0.00040

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	0.00020	0.00010	0.00000	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00010	-0.00030	-0.00040	-0.00050
Diameter 2, in (rotated 90^o)	0.00000	-0.00010	0.00000	0.00000	0.00000	0.00010	0.00020	0.00000	0.00010	0.00020	0.00010	-0.00010	-0.00020	-0.00020	-0.00030

Difference between max and min readings, in:
0° = 0.0007 90° = 0.0005
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00035
Flatness Tolerance Met? YES

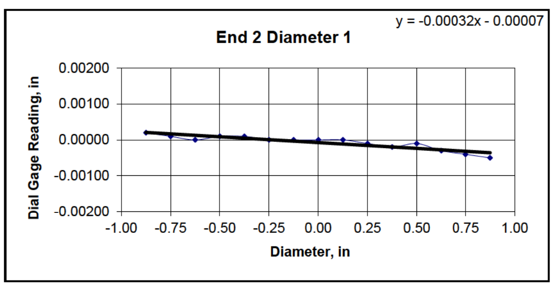


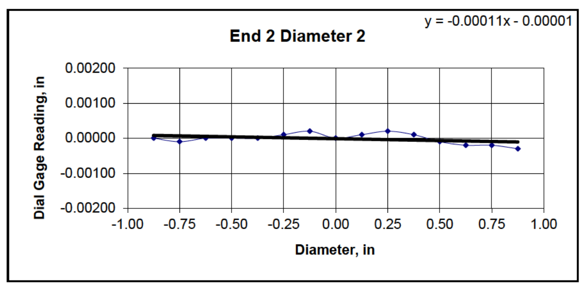


DIAMETER 1

Maximum Angular Difference: [illegible]

Parallelism Tolerance Met? YES
Spherically Seated





DIAMETER 2

End 1:
Slope of Best Fit Line 0.00016
Angle of Best Fit Line: 0.00917

End 2:
Slope of Best Fit Line 0.00011
Angle of Best Fit Line: 0.00630

Maximum Angular Difference: 0.00286

Parallelism Tolerance Met? YES
Spherically Seated

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

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00070	1.990	0.00035	0.020	YES
Diameter 2, in (rotated 90^o)	0.00040	1.990	0.00020	0.012	YES
END 2					
Diameter 1, in	0.00070	1.990	0.00035	0.020	YES
Diameter 2, in (rotated 90^o)	0.00050	1.990	0.00025	0.014	YES

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

Perpendicularity Tolerance Met? YES

Client:	Nobis Engineering, Inc.
Project Name:	Fields Bridge #0690
Project Location:	Sumner-Hartford, ME
GTX #:	306742
Test Date:	7/27/2017
	rlc
	jsc
	BB-SHEBNR-101
	C-2
Depth, ft:	16.33-16.69

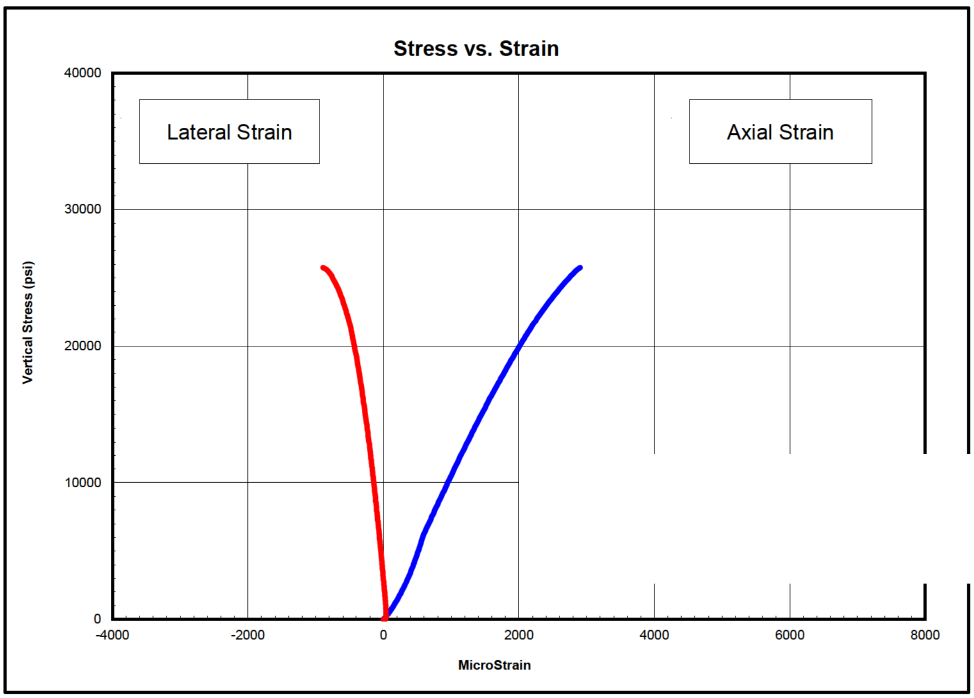
After cutting and grinding

After break

GeoTesting EXPRESS

Client:	Nobis Engineering, Inc.
Project Name:	Fields Bridge #0690
Project Location:	Sumner-Hartford, ME
GTX #:	306742
Test Date:	7/27/2017
Tested By:	rlc
Checked By:	jsc
Boring ID:	BB-SHEBNR-102
Sample ID:	C-1
Depth, ft:	18.60-19.00
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

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



Peak Compressive Stress: 25,755 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
2600-9400	12,300,000	0.24
9400-16300	10,000,000	0.25
16300-23200	8,190,000	0.33

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

GeoTesting EXPRESS

Client:	Nobis Engineering, Inc.	Test Date:	7/24/2017
Project Name:	Fields Bridge #0690	Tested By:	rlc
Project Location:	Sumner-Hartford, ME	Checked By:	jsc
GTX #:	306742		
Boring ID:	BB-SHEBNR-102		
Sample ID:	C-1		
Depth:	18.60-19.00 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:	4.26	4.27	4.27
Specimen Diameter, in:	1.99	1.99	1.99
Specimen Mass, g:	637.99		
Bulk Density, lb/ft^3	183		
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	0.00010	0.00020	0.00030	0.00030	0.00020
Diameter 2, in (rotated 90°)	0.00050	0.00050	0.00030	0.00020	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00020	0.00030	0.00020

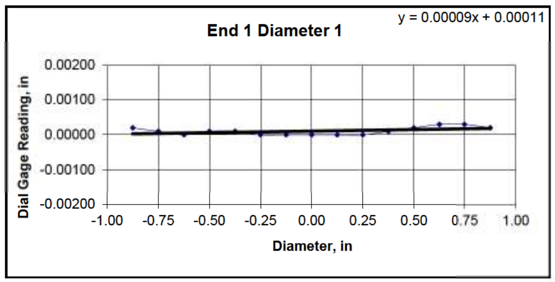
Difference between max and min readings, in:
0° = 0.00030 90° = 0.00050

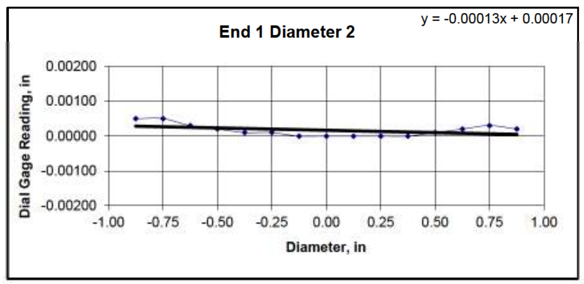
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00010	-0.00020	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00040	-0.00020	-0.00010	-0.00030
Diameter 2, in (rotated 90°)	0.00030	0.00010	0.00000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

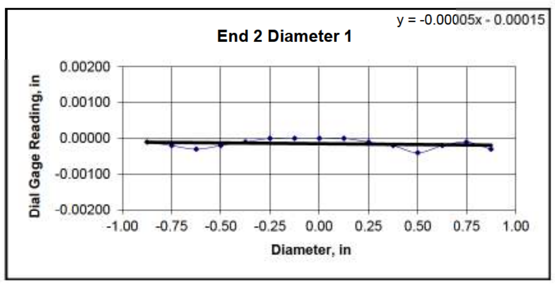
Difference between max and min readings, in:
0° = 0.0004 90° = 0.0006

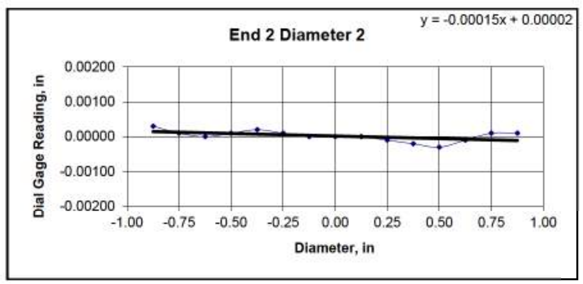
Maximum difference must be < 0.0020 in. Difference = ± 0.00030

Flatness Tolerance Met? YES









DIAMETER 1

End 1:
Slope of Best Fit Line 0.00009
Angle of Best Fit Line: 0.00516

End 2:
Slope of Best Fit Line 0.00005
Angle of Best Fit Line: 0.00286

Maximum Angular Difference: 0.00229

Parallelism Tolerance Met? YES
Spherically Seated

DIAMETER 2

End 1:
Slope of Best Fit Line 0.00013
Angle of Best Fit Line: 0.00745

End 2:
Slope of Best Fit Line 0.00015
Angle of Best Fit Line: 0.00859

Maximum Angular Difference: 0.00115

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°)	0.00050	1.990	0.00025	0.014	YES
END 2					
Diameter 1, in	0.00040	1.990	0.00020	0.012	YES
Diameter 2, in (rotated 90°)	0.00060	1.990	0.00030	0.017	YES

Maximum angle of departure must be ≤ 0.25°

Perpendicularity Tolerance Met? YES

GeoTesting EXPRESS

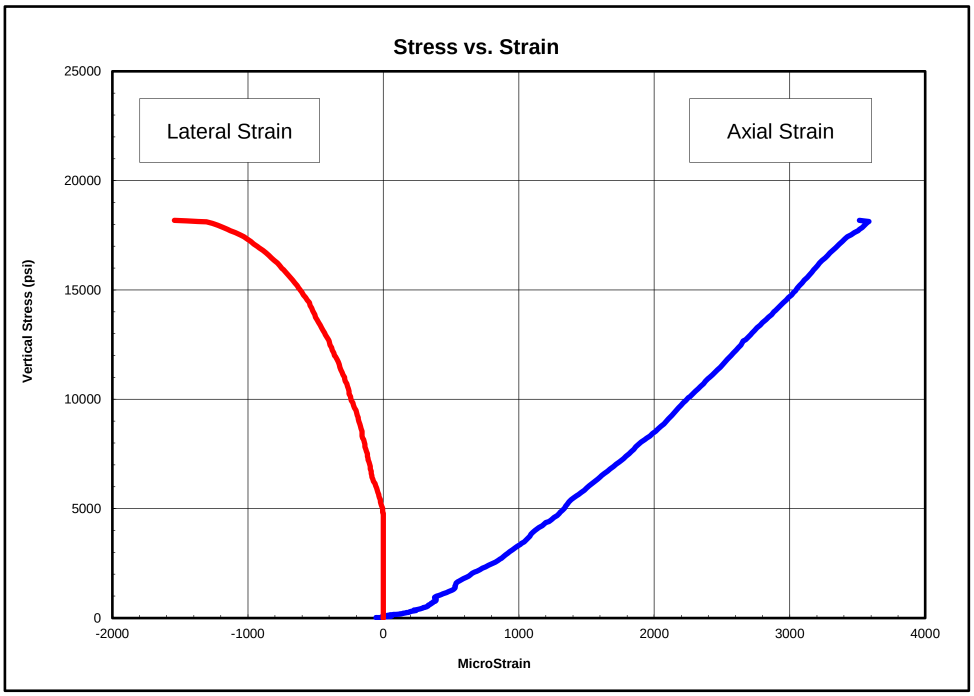
Client:	Nobis Engineering, Inc.
Project Name:	Fields Bridge #0690
Project Location:	Sumner-Hartford, ME
GTX #:	306742
Test Date:	7/27/2017
	rlc
	jsc
	BB-SHEBNR-102
	C-1
Depth, ft:	18.60-19.00

After cutting and grinding

After break

Client:	Nobis Engineering, Inc.
Project Name:	Fields Bridge (#0690) Over East Branch Nezinscot R
Project Location:	Sumner- Hartford, ME
GTX #:	307433
Test Date:	12/19/2017
Tested By:	rlc
Checked By:	jsc
Boring ID:	BB-SHEBNR-104
Sample ID:	R2
Depth, ft:	20.5-21.4
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

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



Peak Compressive Stress: 18,185 psi

One lateral strain gauge failed to record meaningful data. Poisson's Ratio reported based on results of a single lateral strain gauge.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1800-6700	4,760,000	0.11
6700-11500	5,790,000	0.28
11500-16400	6,320,000	---

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

Client:	Nobis Engineering, Inc.	Test Date:	12/18/2017
Project Name:	Fields Bridge (#0690) Over East Branch Nezinscot R	Tested By:	rlc\trm
Project Location:	Sumner- Hartford, ME	Checked By:	jsc
GTX #:	307433		
Boring ID:	BB-SHEBNR-104		
Sample ID:	R2		
Depth:	20.5-21.4 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:	4.04	4.40	4.22		
Specimen Diameter, in:	1.98	1.97	1.98		
Specimen Mass, g:	526.12				
Bulk Density, lb/ft^3	155			**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	-0.00010	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00000	-0.00010	-0.00010	-0.00020	-0.00020
Diameter 2, in (rotated 90°)	0.00020	0.00020	0.00010	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	0.00000	0.00000	-0.00010	-0.00010

Difference between max and min readings, in:

0° = 0.00030 90° = 0.00030

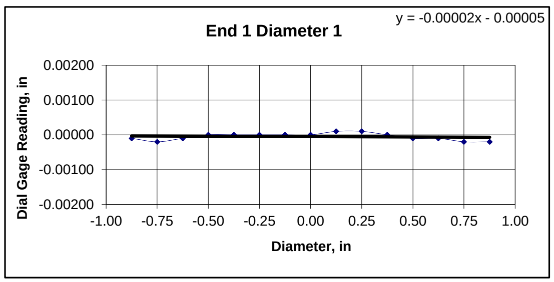
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00020	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00030
Diameter 2, in (rotated 90°)	-0.00010	-0.00020	-0.00010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

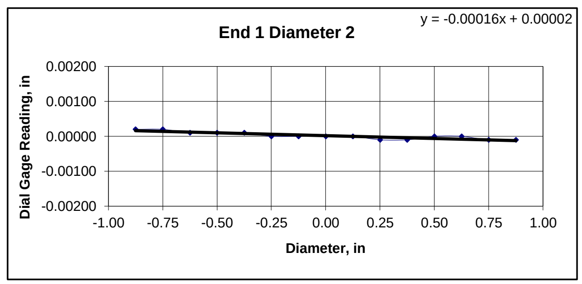
Difference between max and min readings, in:

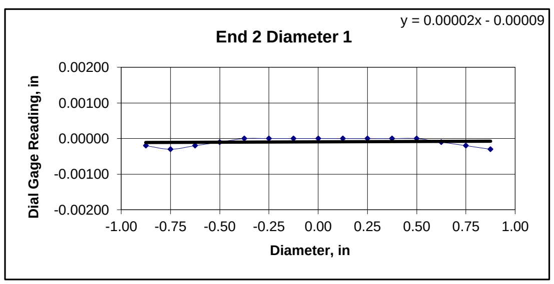
0° = 0.0003 90° = 0.0004

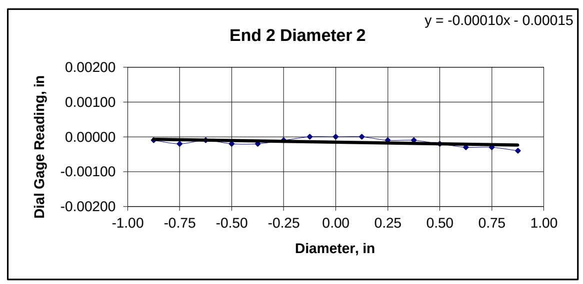
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00020

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	0.00002
Angle of Best Fit Line:	0.00115
Maximum Angular Difference:	0.00000

Parallelism Tolerance Met? **YES**
Spherically Seated

DIAMETER 2

End 1:	
Slope of Best Fit Line	0.00016
Angle of Best Fit Line:	0.00917
End 2:	
Slope of Best Fit Line	0.00010
Angle of Best Fit Line:	0.00573
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	1.975	0.00015	0.009	YES
Diameter 2, in (rotated 90°)	0.00030	1.975	0.00015	0.009	YES
END 2					
Diameter 1, in	0.00030	1.975	0.00015	0.009	YES
Diameter 2, in (rotated 90°)	0.00040	1.975	0.00020	0.012	YES

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

Perpendicularity Tolerance Met? **YES**

Client:	Nobis Engineering, Inc.
Project Name:	Fields Bridge (#0690) Over East Branch Nezinscot R
Project Location:	Sumner- Hartford, ME
GTX #:	307433
Test Date:	12/19/2017
Tested By:	rlc
Checked By:	jsc
Boring ID:	BB-SHEBNR-104
Sample ID:	R2
Depth, ft:	20.5-21.4

After cutting and grinding

After break

APPENDIX F - Calculations

APPENDIX F.1 – Bearing Resistance and Settlement Calculation

Bearing Resistance Calculation for South Abutment

Objective: Evaluate bearing resistance of proposed South concrete abutment supported by bedrock.

References:

1) AASHTO LRFD Bridge Design Specifications: 2014 Edition.
2) AASHTO LRFD Bridge Design Specifications: 2012 Edition.
3) Boring BB-SHEBNR-101 observed by Nobis on June 29, 2017.
4) Design plans and profile provided by MaineDOT on May 18, 2018.
5) MaineDOT Bridge Design Guide: 2003 Edition; with updates through 2014.
6) Hoek and Brown, 1988.
7) Carter and Kulhawy, 1989.

Bearing Properties/Subsurface Information

Bearing Condition	Bedrock
Footing El.	Bottom of Seal Approximate El. 389
Groundwater El.	Approximate El. 395
Proposed Ground Surface El.	Approximate El. 402

Footing Geometry

Eccentricities	e_B, e_L	
Footing Depth (D_f)	0.0 ft	Reference 4
Width (B)	10.0 ft	Reference 4
Length (L)	25.0 ft	Approximate
Effective Width (B')	$B' = B - 2e_B$	
Effective Length (L')	$L' = L - 2e_L$	
Bedrock Slope Angle (β)	0 degs	

Table Key

e_B = Base Eccentricity
e_L = Length Eccentricity
q_R = Factored Bearing Resistance
q_n = Nominal Bearing Resistance
ϕ_b = Resistance Factor

Figure 10.4.6.4-1 - Determination of GSI for Jointed Rock Mass (Hoek and Marinos, 2000), Reference 1.

Core Sample	GSI Value	GSI Classification
BB-SHEBNR-101, R-1 (11'-16')	**50**	FAIR: Smooth, moderately weathered and altered surfaces

Check - Table 10.4.6.4-1 - Geomechanics Classification of Rock Masses, Reference 2. See Note 1.

Core Sample	Relative Rating (Table 10.4.6.4-1)						RMR
	1. Strength of Intact Rock	2. RQD (%)	3. Spacing of Joints	4. Condition of Joints	5. Groundwater Conditions	6. Orientation of Joints	
BB-WMR-101, R-1C (12'-13.9')	7	8	10	20	4	-7	**42**
						Use RMR =	**42**

Nominal Bearing Resistance Calculation

$q_n = (\sqrt{s} + (m\sqrt{s} + s)^{0.5})q_u$ Eq. 82b from Reference 7, presented in NCHRP Report 651.

$m = mi * \exp[(RMR-100)/14]$ Eq. 18, Reference 6, for disturbed rock masses.

m_i = **10** Table 10.4.6.4-1, Reference 1.

$s = \exp[(RMR-100)/6]$ Eq. 19, Reference 6, for disturbed rock masses.

Where RMR = 42, m = **0.159**
s = **6.34E-05**

q_u= 12268.0 (psi) Use minimum of UCS values from laboratory test results
1766.6 (ksf)

$q_n = (\sqrt{s} + (m\sqrt{s} + s)^{0.5})q_u$ = q_n= **78.4** (ksf)

Notes:
1) Both the Geological Strength Index (GSI), Reference 1, and the Rock Mass Rating (RMR), Reference 2, were used for comparison. In accordance with Section C.10.4.6.4 (AASHTO 2014), the RMR system was used to evaluate bearing resistance of the spread footing on rock.
2) Bearing resistance values assume cantilever abutments supported by spread footings/seals constructed on bedrock.

Bearing Resistance Calculation for North Abutment

Objective: Evaluate bearing resistance of proposed North concrete abutment supported by bedrock.

References:

1) AASHTO LRFD Bridge Design Specifications: 2014 Edition.
2) AASHTO LRFD Bridge Design Specifications: 2012 Edition.
3) Boring BB-SHEBNR-104 observed by Nobis on December 4, 2017.
4) Design plans and profile provided by MaineDOT on May 18, 2018.
5) MaineDOT Bridge Design Guide: 2003 Edition; with updates through 2014.
6) Hoek and Brown, 1988.
7) Carter and Kulhawy, 1989.

Bearing Properties/Subsurface Information

Bearing Condition	Bedrock
Footing El.	Bottom of Seal Approximate El. 385.8
Groundwater El.	Approximate El. 397
Proposed Ground Surface El.	Approximate El. 402

Footing Geometry

Eccentricities	e_B, e_L	
Footing Depth (D_f)	0.0 ft	Reference 4
Width (B)	10.0 ft	Reference 4
Length (L)	25.0 ft	Approximate
Effective Width (B')	$B' = B - 2e_B$	
Effective Length (L')	$L' = L - 2e_L$	
Bedrock Slope Angle (β)	0 degs	

Table Key

e_B = Base Eccentricity
e_L = Length Eccentricity
q_R = Factored Bearing Resistance
q_n = Nominal Bearing Resistance
ϕ_b = Resistance Factor

Figure 10.4.6.4-1 - Determination of GSI for Jointed Rock Mass (Hoek and Marinos, 2000), Reference 1.

Core Sample	GSI Value	GSI Classification
BB-WMR-104, R-1 (12'-17')	**50**	FAIR: Smooth, moderately weathered and altered surfaces

Check - Table 10.4.6.4-1 - Geomechanics Classification of Rock Masses, Reference 2. See Note 1.

Core Sample	Relative Rating (Table 10.4.6.4-1)						RMR
	1. Strength of Intact Rock	2. RQD (%)	3. Spacing of Joints	4. Condition of Joints	5. Groundwater Conditions	6. Orientation of Joints	
BB-WMR-104, R-1 (12'-17')	7	17	10	20	4	-7	**51**
						Use RMR =	**51**

Nominal Bearing Resistance Calculation

$q_n = (\sqrt{s} + (m\sqrt{s} + s)^{0.5})q_u$ — Eq. 82b from Reference 7, presented in NCHRP Report 651.

$m = m_i * \exp[(RMR-100)/14]$ — Eq. 18, Reference 6, for disturbed rock masses.

$m_i =$ **10** — Table 10.4.6.4-1, Reference 1.

$s = \exp[(RMR-100)/6]$ — Eq. 19, Reference 6, for disturbed rock masses.

Where RMR = 51,
$m =$ **0.302**
$s =$ **2.84E-04**

$q_u =$ 12268.0 (psi) — Use minimum of UCS values from laboratory test results
1766.6 (ksf)

$q_n = (\sqrt{s} + (m\sqrt{s} + s)^{0.5})q_u =$ $q_n =$ **159.3** (ksf)

Notes:
1) Both the Geological Strength Index (GSI), Reference 1, and the Rock Mass Rating (RMR), Reference 2, were used for comparison. In accordance with Section C.10.4.6.4 (AASHTO 2014), the RMR system was used to evaluate bearing resistance of the spread footing on rock.
2) Bearing resistance values assume cantilever abutments supported by spread footings/seals constructed on bedrock.

<u>Bearing Resistance Calculation for Proposed North and South Abutment</u>

Objective: Evaluate bearing resistance of proposed North and South Abutments supported by bedrock.

<u>References:</u>

1) AASHTO LRFD Bridge Design Specifications: 2014 Edition.
2) AASHTO LRFD Bridge Design Specifications: 2012 Edition.
3) Boring BB-SHEBNR-101 and -104 observed by Nobis in 2017.
4) Design plans and profile provided by MaineDOT on May 18, 2018.
5) MaineDOT Bridge Design Guide: 2003 Edition; with updates through 2014.
6) Hoek and Brown, 1988.
7) Carter and Kulhawy, 1989.

<u>Bearing Properties/Subsurface Information</u>

Bearing Condition	Bedrock
Footing El.	Footing Approximate El. 386 to 389
Groundwater El.	Approximate El. 395 to 397
Proposed Ground Surface El.	Approximate El. 402

Nominal Bearing Resistance of Bedrock @ South Abutment q_n= 78.4 ksf (lesser of q_n used for further calculation)
North Abutment q_n= 159.3 ksf

<u>Factored Bearing Resistance Calculation for North and South Abutments and Bridge Pier</u>

$q_R = q_n * \phi_b$ $\phi_b = 0.45$ (Table 5-2, Reference 5, Resistance Factors for Shallow Foundations at the Strength Limit State)

q_R =	**35.3**	**(ksf)**

(Strength Limit State Bearing Resistance of Bedrock)

$q_R = q_n * \phi_b$ $\phi_b = 1.0$ (10.5.5.3.3 and 10.6.4.1, Resistance Factors for Shallow Foundations at the Extreme Limit State)

q_R =	**78.4**	**(ksf)**

(Extreme Limit State Bearing Resistance of Bedrock)

Settlement Calculation for South Abutment

Reference :
AASHTO LRFD Bridge Design Specifications: 6th Edition - 2012

$$\rho = \frac{(q_o(1-v^2)B'I_p)}{144*E_m}$$ Eq. 10.6.2.4.4-3

q_o= applied vertical stress (ksf)
v = Poisson's Ratio
B'= effective footing width (ft)
E_m= rock mass modulus (ksi)

where $I_p = \frac{(L'/B')^{1/2}}{\beta_z}$ Eq. 10.6.2.4.4-4

L'= effective footing length (ft)
β_z= shape factor

where $E_m = (E_m/E_i)E_i$ Eq. 10.4.6.5-2

(E_m/E_i)= reduction factor from Tb. 10.4.6.5-1 (dim.)
E_i= elastic modulus of intact rock (ksi)

Calculations:

B' (ft)=	10	Assumed
L' (ft)=	25	Assumed
L'/B'=	2.5	
β_z=	1.12	Tb. 10.6.2.4.2-1

v =	0.39	From BB-SHEBNR-101, C-2 Lab Results
(E_m/E_i)=	0.150	Tb. 10.4.6.5-1 interpolated from RQD
E_i=	3400 ksi	From Lab Data Results
E_m=	510 ksi	

Assumed Settlement, S_e (in)	Applied Vertical Stress, q_o (ksf)	
0.15	76.7	**Strength Load Case Controls**

Notes:
1. Settlement estimate assume that the abutment footing/seal is constructed on bedrock.

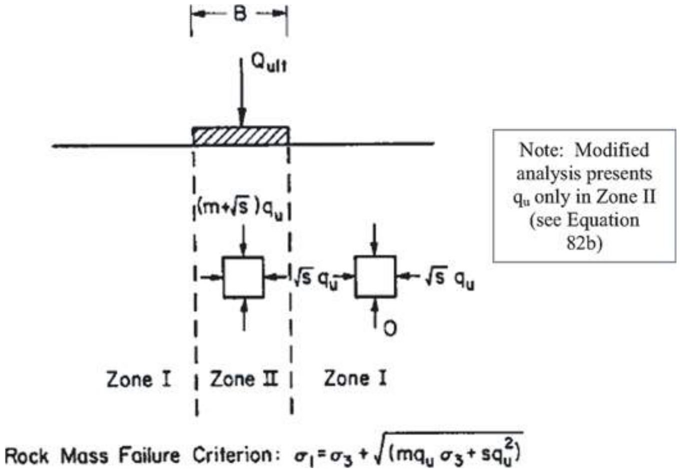


Figure 39. Lower bound solution for bearing capacity (Carter and Kulhawy, 1988).

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

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

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

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

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 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 :

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

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

Undisturbed or interlocking rock masses:

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

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

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) :

$$\mathrm{RMR} = 9\,\mathrm{Log}_e\,\mathrm{Q} + 44 \tag{22}$$

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.

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*	m	7.00	10.00	15.00	17.00	25.00
	s	1.00	1.00	1.00	1.00	1.00
CSIR rating: RMR = 100	*m*	*7.00*	*10.00*	*15.00*	*17.00*	*25.00*
NGI rating: Q = 500	*s*	*1.00*	*1.00*	*1.00*	*1.00*	*1.00*
VERY GOOD QUALITY ROCK MASS *Tightly interlocking undisturbed rock with unweathered joints at 1 to 3m.*	m	2.40	3.43	5.14	5.82	8.56
	s	0.082	0.082	0.082	0.082	0.082
CSIR rating: RMR = 85	*m*	*4.10*	*5.85*	*8.78*	*9.95*	*14.63*
NGI rating: Q = 100	*s*	*0.189*	*0.189*	*0.189*	*0.189*	*0.189*
GOOD QUALITY ROCK MASS *Fresh to slightly weathered rock, slightly disturbed with joints at 1 to 3m.*	m	0.575	0.821	1.231	1.395	2.052
	s	0.00293	0.00293	0.00293	0.00293	0.00293
CSIR rating: RMR = 65	*m*	*2.006*	*2.865*	*4.298*	*4.871*	*7.163*
NGI rating: Q = 10	*s*	*0.0205*	*0.0205*	*0.0205*	*0.0205*	*0.0205*
FAIR QUALITY ROCK MASS *Several sets of moderately weathered joints spaced at 0.3 to 1m.*	m	0.128	0.183	0.275	0.311	0.458
	s	0.00009	0.00009	0.00009	0.00009	0.00009
CSIR rating: RMR = 44	*m*	*0.947*	*1.353*	*2.030*	*2.301*	*3.383*
NGI rating: Q = 1	*s*	*0.00198*	*0.00198*	*0.00198*	*0.00198*	*0.00198*
POOR QUALITY ROCK MASS *Numerous weathered joints at 30-500mm, some gouge. Clean compacted waste rock*	m	0.029	0.041	0.061	0.069	0.102
	s	0.000003	0.000003	0.000003	0.000003	0.000003
CSIR rating: RMR = 23	*m*	*0.447*	*0.639*	*0.959*	*1.087*	*1.598*
NGI rating: Q = 0.1	*s*	*0.00019*	*0.00019*	*0.00019*	*0.00019*	*0.00019*
VERY POOR QUALITY ROCK MASS *Numerous heavily weathered joints spaced <50mm with gouge. Waste rock with fines.*	m	0.007	0.010	0.015	0.017	0.025
	s	0.0000001	0.0000001	0.0000001	0.0000001	0.0000001
CSIR rating: RMR = 3	*m*	*0.219*	*0.313*	*0.469*	*0.532*	*0.782*
NGI rating: Q = 0.01	*s*	*0.00002*	*0.00002*	*0.00002*	*0.00002*	*0.00002*

The modulus of elasticity for normally consolidated granular soils tends to increase with depth. An alternative method of defining the soil modulus for granular soils is to assume that it increases linearly with depth starting at zero at the ground surface in accordance with the following equation:

$$E_s = nh \times z \qquad \text{(C10.4.6.3-1)}$$

where:

E_s = the soil modulus at depth z (ksi)
n_h = rate of increase of soil modulus with depth as defined in Table C10.4.6.3-2 (ksi/ft)
z = depth below the ground surface (ft)

Table C10.4.6.3-2—Rate of Increase of Soil Modulus with Depth n_h (ksi/ft) for Sand

Consistency	Dry or Moist	Submerged
Loose	0.417	0.208
Medium	1.11	0.556
Dense	2.78	1.39

The formulation provided in Eq. C10.4.6.3-1 is used primarily for analysis of lateral response or buckling of deep foundations.

The potential for soil swell that may result in uplift on deep foundations or heave of shallow foundations should be evaluated based on Table 10.4.6.3-1.

Table 10.4.6.3-1—Method for Identifying Potentially Expansive Soils (Reese and O'Neill, 1988)

Liquid Limit *LL* (%)	Plastic Limit *PL* (%)	Soil Suction (ksf)	Potential Swell (%)	Potential Swell Classification
>60	>35	>8	>1.5	High
50–60	25–35	3–8	0.5–1.5	Marginal
<50	<25	<3	<0.5	Low

10.4.6.4—Rock Mass Strength

The strength of intact rock material should be determined using the results of unconfined compression tests on intact rock cores, splitting tensile tests on intact rock cores, or point load strength tests on intact specimens of rock.

The rock should be classified using the rock mass rating system (RMR) as described in Table 10.4.6.4-1. For each of the five parameters in the Table, the relative rating based on the ranges of values provided should be evaluated. The rock mass rating (RMR) should be determined as the sum of all five relative ratings. The RMR should be adjusted in accordance with the criteria in Table 10.4.6.4-2. The rock classification should be determined in accordance with Table 10.4.6.4-3.

C10.4.6.4

Because of the importance of the discontinuities in rock, and the fact that most rock is much more discontinuous than soil, emphasis is placed on visual assessment of the rock and the rock mass.

Other methods for assessing rock mass strength, including in-situ tests or other visual systems that have proven to yield accurate results may be used in lieu of the specified method.

Table 10.4.6.4-1—Geomechanics Classification of Rock Masses

	Parameter		Ranges of Values						
1	Strength of intact rock material	Point load strength index	>175 ksf	85–175 ksf	45–85 ksf	20–45 ksf	For this low range, uniaxial compressive test is preferred		
		Uniaxial compressive strength	>4320 ksf	2160–4320 ksf	1080–2160 ksf	520–1080 ksf	215–520 ksf	70–215 ksf	20–70 ksf
	Relative Rating		15	12	7	4	2	1	0
2	Drill core quality RQD		90% to 100%	75% to 90%	50% to 75%	25% to 50%	<25%		
	Relative Rating		20	17	13	8	3		
3	Spacing of joints		>10 ft	3–10 ft	1–3 ft	2 in.–1 ft	<2 in.		
	Relative 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 <0.05 in. • Hard joint wall rock	• Slightly rough surfaces • Separation <0.05 in. • Soft joint wall rock	• Slicken-sided surfaces or • Gouge <0.2 in. thick or • Joints open 0.05–0.2 in. • Continuous joints	• Soft gouge >0.2 in. thick or • Joints open >0.2 in. • Continuous joints		
	Relative Rating		25	20	12	6	0		
5	Groundwater conditions (use one of the three evaluation criteria as appropriate to the method of exploration)	Inflow per 30 ft tunnel length	None	<400 gal./hr.	400–2000 gal./hr.	>2000 gal./hr.			
		Ratio = joint water pressure/ major principal stress	0	0.0–0.2	0.2–0.5	>0.5			
		General Conditions	Completely Dry	Moist only (interstitial water)	Water under moderate pressure	Severe water problems			
	Relative Rating		10	7	4	0			

Table 10.4.6.4-2—Geomechanics Rating Adjustment for Joint Orientations

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

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

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

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

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

$$\tau = \left(\cot \phi_i' - \cos \phi_i'\right) m \frac{q_u}{8} \qquad (10.4.6.4\text{-}1)$$

in which:

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

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

where:

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

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

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

σ'_n = effective normal stress (ksf)

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

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

$$c_i = \tau - \sigma_n' \tan \phi_i' \qquad (C10.4.6.4\text{-}1)$$

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

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

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

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

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

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

Table C10.4.6.4-1—Typical Ranges of Friction Angles for Smooth Joints in a Variety of Rock Types (modified after Barton, 1976; Jaeger and Cook, 1976)

Rock Class	Friction Angle Range	Typical Rock Types
Low Friction	20–27°	Schists (high mica content), shale, marl
Medium Friction	27–34°	Sandstone, siltstone, chalk, gneiss, slate
High Friction	34–40°	Basalt, granite, limestone, conglomerate

Note: Values assume no infilling and little relative movement between joint faces.

When a major discontinuity with a significant thickness of infilling is to be investigated, the shear strength will be governed by the strength of the infilling material and the past and expected future displacement of the discontinuity. Refer to Sabatini et al. (2002) for detailed procedures to evaluate infilled discontinuities.

10.4.6.5—Rock Mass Deformation

The elastic modulus of a rock mass (E_m) shall be taken as the lesser of the intact modulus of a sample of rock core (E_i) or the modulus determined from one of the following equations:

$$E_m = 145\left(10^{\frac{RMR-10}{40}}\right) \qquad (10.4.6.5\text{-}1)$$

where:

E_m = Elastic modulus of the rock mass (ksi)

E_m $\leq$ E_i

E_i = Elastic modulus of intact rock (ksi)

RMR = Rock mass rating specified in Article 10.4.6.4.

or

$$E_m = \left(\frac{E_m}{E_i}\right)E_i \qquad (10.4.6.5\text{-}2)$$

C10.4.6.5

Table 10.4.6.5-1 was developed by O'Neill and Reese (1999) based on a reanalysis of the data presented by Carter and Kulhawy (1988) for the purposes of estimating side resistance of shafts in rock.

Preliminary estimates of the elastic modulus of intact rock may be made from Table C10.4.6.5-1. Note that some of the rock types identified in the Table are not present in the U.S.

It is extremely important to use the elastic modulus of the rock mass for computation of displacements of rock materials under applied loads. Use of the intact modulus will result in unrealistic and unconservative estimates.

where:

E_m = Elastic modulus of the rock mass (ksi)

E_m/E_i = Reduction factor determined from Table 10.4.6.5-1 (dim)

E_i = Elastic modulus of intact rock from tests (ksi)

For critical or large structures, determination of rock mass modulus (E_m) using in-situ tests may be warranted. Refer to Sabatini et al. (2002) for descriptions of suitable in-situ tests.

Table 10.4.6.5-1—Estimation of E_m Based on *RQD* (after O'Neill and Reese, 1999)

RQD (percent)	E_m/E_i	
	Closed Joints	Open Joints
100	1.00	0.60
70	0.70	0.10
50	0.15	0.10
20	0.05	0.05

Table C10.4.6.5-1—Summary of Elastic Moduli for Intact Rock (modified after Kulhawy, 1978)

Rock Type	No. of Values	No. of Rock Types	Elastic Modulus, E_i (ksi ×10^3)			Standard Deviation (ksi × 10^3)
			Maximum	Minimum	Mean	
Granite	26	26	14.5	0.93	7.64	3.55
Diorite	3	3	16.2	2.48	7.45	6.19
Gabbro	3	3	12.2	9.8	11.0	0.97
Diabase	7	7	15.1	10.0	12.8	1.78
Basalt	12	12	12.2	4.20	8.14	2.60
Quartzite	7	7	12.8	5.29	9.59	2.32
Marble	14	13	10.7	0.58	6.18	2.49
Gneiss	13	13	11.9	4.13	8.86	2.31
Slate	11	2	3.79	0.35	1.39	0.96
Schist	13	12	10.0	0.86	4.97	3.18
Phyllite	3	3	2.51	1.25	1.71	0.57
Sandstone	27	19	5.68	0.09	2.13	1.19
Siltstone	5	5	4.76	0.38	2.39	1.65
Shale	30	14	5.60	0.001	1.42	1.45
Limestone	30	30	13.0	0.65	5.7	3.73
Dolostone	17	16	11.4	0.83	4.22	3.44

Poisson's ratio for rock should be determined from tests on intact rock core.

Where tests on rock core are not practical, Poisson's ratio may be estimated from Table C10.4.6.5-2.

Table C10.4.6.5-2—Summary of Poisson's Ratio for Intact Rock (modified after Kulhawy, 1978)

Rock Type	No. of Values	No. of Rock Types	Poisson's Ratio, ν			Standard Deviation
			Maximum	Minimum	Mean	
Granite	22	22	0.39	0.09	0.20	0.08
Gabbro	3	3	0.20	0.16	0.18	0.02
Diabase	6	6	0.38	0.20	0.29	0.06
Basalt	11	11	0.32	0.16	0.23	0.05
Quartzite	6	6	0.22	0.08	0.14	0.05
Marble	5	5	0.40	0.17	0.28	0.08
Gneiss	11	11	0.40	0.09	0.22	0.09
Schist	12	11	0.31	0.02	0.12	0.08
Sandstone	12	9	0.46	0.08	0.20	0.11
Siltstone	3	3	0.23	0.09	0.18	0.06
Shale	3	3	0.18	0.03	0.09	0.06
Limestone	19	19	0.33	0.12	0.23	0.06
Dolostone	5	5	0.35	0.14	0.29	0.08

10.4.6.6—Erodibility of Rock

Consideration should be given to the physical characteristics of the rock and the condition of the rock mass when determining a rock's susceptibility to erosion in the vicinity of bridge foundations. Physical characteristics that should be considered in the assessment of erodibility include cementing agents, mineralogy, joint spacing, and weathering.

C10.4.6.6

There is no consensus on how to determine erodibility of rock masses near bridge foundations. Refer to Richardson and Davis (2001) "Evaluating Scour at Bridges—Fourth Edition", Mayne et al. (2001), Appendix M for guidance on two proposed methods. The first method was proposed in an FHWA memorandum of July 1991 and consists of evaluating various rock index properties. The second method is documented in Smith (1994) "Preliminary Procedure to Evaluate Scour in Bedrock" which uses the erodibility index proposed by G. W. Annandale. The Engineer should consider the appropriateness of these two methods when determining the potential for a rock mass to scour.

10.5—LIMIT STATES AND RESISTANCE FACTORS

10.5.1—General

The limit states shall be as specified in Article 1.3.2; foundation-specific provisions are contained in this Section.

Foundations shall be proportioned so that the factored resistance is not less than the effects of the factored loads specified in Section 3.

10.5.2—Service Limit States

10.5.2.1—General

Foundation design at the service limit state shall include:

- Settlements,

C10.5.2.1

In bridges where the superstructure and substructure are not integrated, settlement corrections can be made by jacking and shimming bearings. Article 2.5.2.3 requires jacking provisions for these bridges.

- Horizontal movements,
- Overall stability, and
- Scour at the design flood.

Consideration of foundation movements shall be based upon structure tolerance to total and differential movements, rideability and economy. Foundation movements shall include all movement from settlement, horizontal movement, and rotation.

Bearing resistance estimated using the presumptive allowable bearing pressure for spread footings, if used, shall be applied only to address the service limit state.

The cost of limiting foundation movements should be compared with the cost of designing the superstructure so that it can tolerate larger movements or of correcting the consequences of movements through maintenance to determine minimum lifetime cost. The Owner may establish more stringent criteria.

The design flood for scour is defined in Article 2.6.4.4.2, and is specified in Article 3.7.5 as applicable at the service limit state.

Presumptive bearing pressures were developed for use with working stress design. These values may be used for preliminary sizing of foundations, but should generally not be used for final design. If used for final design, presumptive values are only applicable at service limit states.

10.5.2.2—Tolerable Movements and Movement Criteria

Foundation movement criteria shall be consistent with the function and type of structure, anticipated service life, and consequences of unacceptable movements on structure performance. Foundation movement shall include vertical, horizontal, and rotational movements. The tolerable movement criteria shall be established by either empirical procedures or structural analyses, or by consideration of both.

Foundation settlement shall be investigated using all applicable loads in the Service I Load Combination specified in Table 3.4.1-1. Transient loads may be omitted from settlement analyses for foundations bearing on or in cohesive soil deposits that are subject to time-dependant consolidation settlements.

All applicable service limit state load combinations in Table 3.4.1-1 shall be used for evaluating horizontal movement and rotation of foundations.

Horizontal movement criteria should be established at the top of the foundation based on the tolerance of the structure to lateral movement, with consideration of the column length and stiffness.

C10.5.2.2

Experience has shown that bridges can and often do accommodate more movement and/or rotation than traditionally allowed or anticipated in design. Creep, relaxation, and redistribution of force effects accommodate these movements. Some studies have been made to synthesize apparent response. These studies indicate that angular distortions between adjacent foundations greater than 0.008 rad. in simple spans and 0.004 rad. in continuous spans should not be permitted in settlement criteria (Moulton et al., 1985; DiMillio, 1982; Barker et al., 1991). Other angular distortion limits may be appropriate after consideration of:

- cost of mitigation through larger foundations, realignment or surcharge,
- rideability,
- aesthetics, and
- safety.

Rotation movements should be evaluated at the top of the substructure unit in plan location and at the deck elevation.

Tolerance of the superstructure to lateral movement will depend on bridge seat or joint widths, bearing type(s), structure type, and load distribution effects.

10.5.2.3—Overall Stability

The evaluation of overall stability of earth slopes with or without a foundation unit shall be investigated at the service limit state as specified in Article 11.6.2.3.

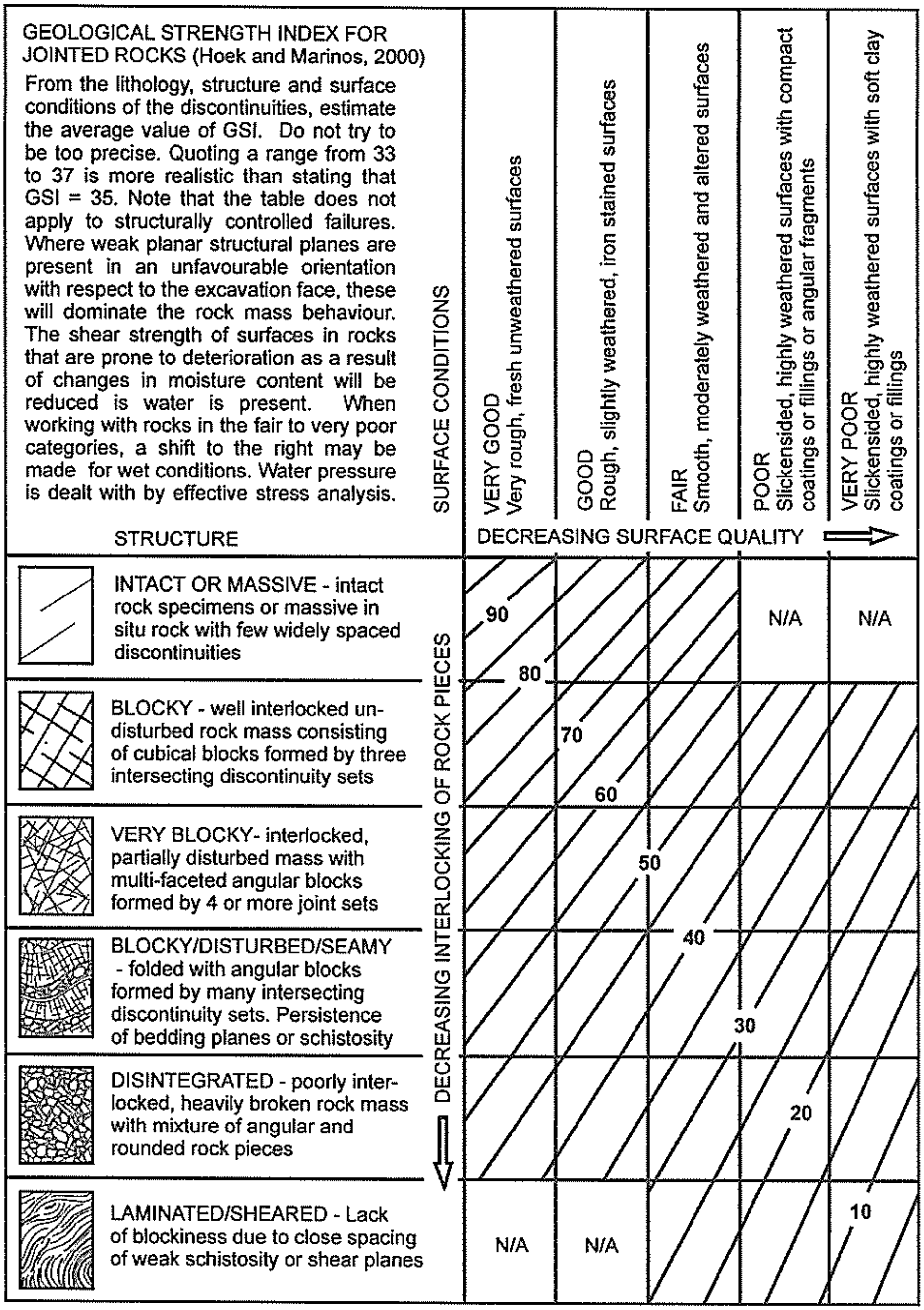


Figure 10.4.6.4-1—Determination of *GSI* for Jointed Rock Mass (Hoek and Marinos, 2000)

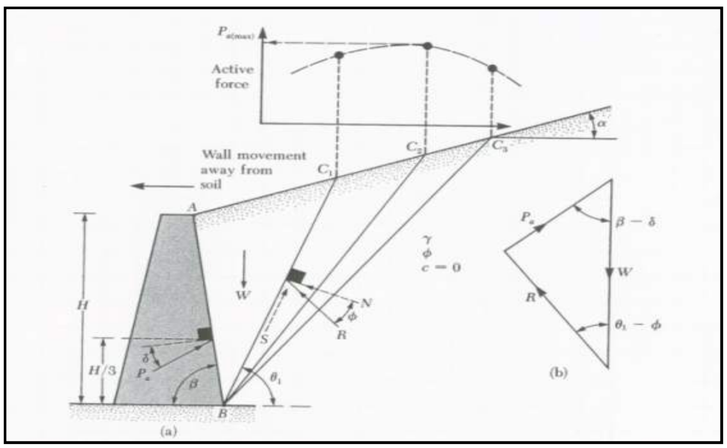
APPENDIX F.2 – Active Earth Pressure

Objective: Calculate lateral earth pressure coefficient for the proposed north and south abutments.

Approach: Use Rankine's method to determine active earth pressure coefficient in accordance with MaineDOT Bridge Design Manual.

References:
1. MaineDOT Bridge Design Guide, 2003, with revisions through 2014.
2. Foundation Design, Principles and Practices, Second Edition, Coduto
3. AASHTO Bridge Design Specifications, 2014 Edition.

Active Earth Pressure using Rankine's Method (Foundation Design, Principles and Practices, Second Edition, Coduto)



$$K_a = \tan^2(45° - \phi/2)$$

α = backslope angle
β = wall batter angle
ϕ = soil friction angle

Proposed Wall

α	β	ϕ
0	90	32

The above equation is valid for a vertical backface and horizontal fill behind the wall.

K_a = 0.31 **(Use for proposed abutments)**

We recommend that the proposed abutments be designed for lateral earth pressures using backfill material properties for Soil Type 4 (MaineDOT Bridge Design Guide Section 3.6.1). In accordance with the MaineDOT Bridge Manual Soil Type 4 has a friction angle of 32 degrees.

Table 3-3 Material Classification

Soil Type	Soil Description	Internal Angle of Friction of Soil, ϕ	Soil Total Unit Weight (pcf)	Coeff. of Friction, tan δ, Concrete to Soil	Interface Friction, Angle, Concrete to Soil δ
1	Very loose to loose silty sand and gravel Very loose to loose sand Very loose to medium density sandy silt Stiff to very stiff clay or clayey silt	29°*	100	0.35	19°
2	Medium density silty sand and gravel Medium density to dense sand Dense to very dense sandy silt	33°	120	0.40	22°
3	Dense to very dense silty sand and gravel Very dense sand	36°	130	0.45	24°
4	Granular underwater backfill Granular borrow	32°	125	0.45	24°
5	Gravel Borrow	38°	135	0.50	27°

* The value given for the internal angle of friction (ϕ) for stiff to very stiff silty clay or clayey silt should be used with caution due to the large possible variation with different moisture contents.

APPENDIX F.3 – Seismic Design Considerations

Calculated by: JDV Date: 8/1/2018
Checked by: BTW Date: 8/2/2018
Revision: 0
Status: Final

Seismic site classification for the Fields Bridge - Gammon Road over East Branch Nezinscot River

Objective: Evaluate seismic site classification for the above mentioned project site in Sumner-Hartford, Maine.

References:

1) AASHTO LRFD Bridge Design Specifications, 7th Edition, 2014, with 2016 Interim Revisions.
2) Borings Observed by Nobis Engineering in June 2017 and December 2017.
3) MaineDOT Bridge Design Manual, 2003, with revisions through March, 2017.

Solution:

AASHTO Section 3.10 was used to determine the seismic site classification for the Fields Bridge project site, as follows:

Step 1: Check for the three categories of Site Class F as described in Table 3.10.3.1-1 - Site Classification Definitions, as follows:

1. Peats or highly organic clays (H>10 ft of peat or highly organic clay where H = thickness of soil).
2. Very high plasticity clays (H>25 ft with PI >75).
3. Very thick soft/medium stiff clays (H > 120 ft).

Soil conditions for Site Class F were not shown on the Nobis Boring Logs.

Step 2: Check for existence of a soft layer with total thickness >10 ft, where soft layer is defined by su<0.5 ksf, w>40%, and PI>20. If these criteria are met, classify site as Site Class E.

Soil conditions for Site Class E were not shown on the Nobis Boring Logs.

Step 3: Categorize the site using one of three methods (i.e. A, B, or C).

Method B (N-bar Method) was used to determine the average Standard Penetration Test (SPT) blow count (blows/ft) for the upper 100 ft of the soil profile using the 2017 borings. See attached Table 3.10.3.1-1. The 2016 borings were performed in general accordance with ASTM D1586. The samples were obtained using a 1-3/8" diameter sampler driven with a 140-lb safety hammer dropping a distance of 30 inches.

Conclusion:

Based on the Nobis boring logs, the average Standard Penetration Resistance, N-bar, for the site is above 50. Bedrock consisting of Metasandstone/ Metasiltstone or Quartzite/ Granite/ Mica was encountered in the test borings from approximately 11 to 18.6 feet below ground surface. Based on the bedrock cores collected, the Rock Quality Designation (RQD) ranged from 37 to 88 percent with a typical value of 70 to 80 percent. Assume that the bedrock has a shear wave velocity greater than 2,500 ft/sec. **Therefore, Site Class B - Rock**. The calculated values for the borings are summarized in the following table. See attached tables presenting the value of SPT-N vs. depth for the respective borings.

Sample Calculation: Consider Boring BB-SHEBNR-101

Determine N-bar: Use the average N value of each layer, N_i, for each layer provided in the table toward the end of this report.

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

$$\overline{N} = \frac{9' + 2' + 89'}{\frac{9'}{25} + \frac{2'}{93} + \frac{89'}{100}} = 79\ bpf$$

Determine the site class for this boring using the Site Class Definitions, attached (Table 3.10.3.1-1, Reference 1)

Below, summary tables provide general information for all of the borings considered in this analysis.

Table: Summary of Seismic Site Classification from Borings Performed by Nobis Engineering

Boring No.	N-bar (bpf)	Site Class	Comments
BB-SHEBNR-101	**79**	B	Bedrock encountered about 11 feet bgs
BB-SHEBNR-102	**65**	B	Bedrock encountered about 18.5 feet bgs
BB-SHEBNR-103	**-**	-	Refusal on concrete about 3.75 feet bgs
BB-SHEBNR-104	**73**	B	Bedrock encountered about 15.2 feet bgs

Data from Boring BB-SHEBNR-101 (performed June 2017)

Layer	Depth Range		Thickness (d_i)	N_i	d_i/N_i	Comments
	Start [ft]	End [ft]	[ft]	blows/ft		
1	0	9	9	25	0.360	Fill
2	9	11	2	93	0.022	Weathered Bedrock
3	11	100	89	100	0.890	Bedrock
SUM			**100**		**1.272**	

N-bar	**79**

Site Class B - Rock

Calculated by: JDV Date: 8/1/2018
Checked by: BTW Date: 8/2/2018
Revision: 0
Status: Final

Data from Boring BB-SHEBNR-102 (performed June 2017)

Layer	Depth Range		Thickness (d_i)	N_i	d_i/N_i	Comments
	Start [ft]	End [ft]	[ft]	blows/ft		
1	0	9	9	22	0.409	Fill
2	9	18.6	9.6	30	0.320	Sand (Outwash Deposit)
3	18.6	100	81.4	100	0.814	Bedrock
SUM			**100**		**1.543**	

N-bar	**65**	**Site Class B - Rock**

Data from Boring BB-SHEBNR-104 (performed December 2017)

Layer	Depth Range		Thickness (d_i)	N_i	d_i/N_i	Comments
	Start [ft]	End [ft]	[ft]	blows/ft		
1	0	9	9	32	0.281	Fill
2	9	15.2	6.2	25	0.248	Sand (Outwash Deposit)
3	15.2	100	84.8	100	0.848	Bedrock
SUM			**100**		**1.377**	

N-bar	**73**	**Site Class B - Rock**

USGS Design Maps Summary Report

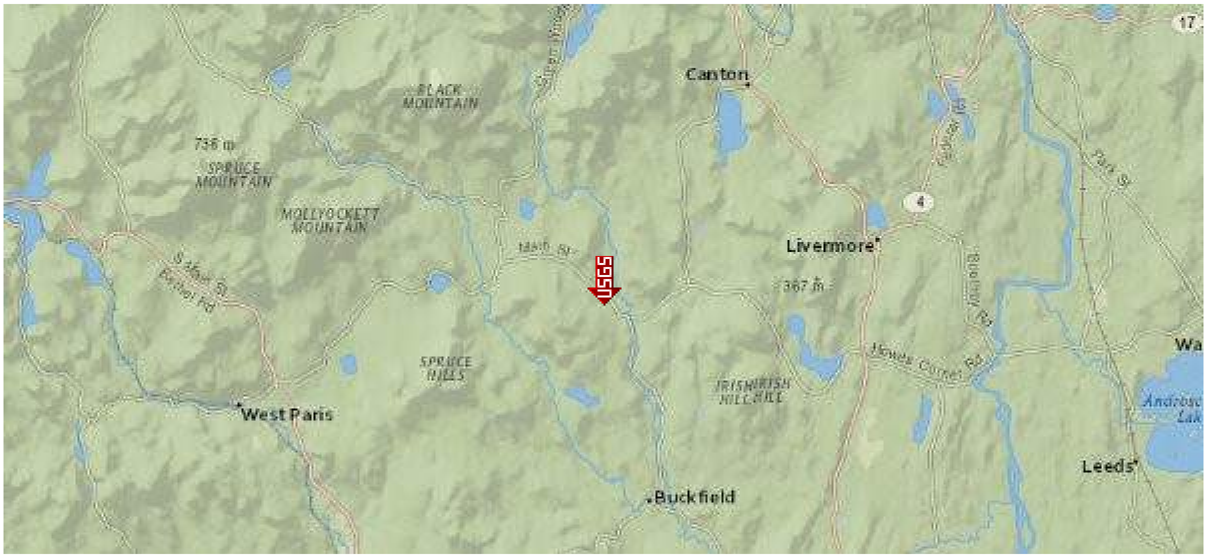
User–Specified Input

Report Title Fields Bridge
Wed August 1, 2018 20:55:34 UTC

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

Site Coordinates 44.36995°N, 70.38888°W

Site Soil Classification Site Class B – "Rock"



USGS–Provided Output

PGA =	0.089 g	**A_s =**	0.089 g
S_S =	0.180 g	**S_{DS} =**	0.180 g
S_1 =	0.048 g	**S_{D1} =**	0.048 g