SCHONEWALD ENGINEERING ASSOCIATES, INC.

**GEOTECHNICAL DESIGN REPORT**
**FENDER SYSTEM MODIFICATIONS**
**MAINE STATE FERRY TERMINAL – FRENCHBORO, MAINE**
**MAINEDOT WIN 22202.00**

**PREPARED FOR:**

HNTB Corporation
Westbrook, Maine

**PREPARED BY:**

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**May 2019**

SchonewaldEA Project No. 18-012

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## INTRODUCTION

Schonewald Engineering Associates, Inc. (SchonewaldEA) has prepared this Geotechnical Design Report for HNTB Corporation (HNTB) to present subsurface information and provide final geotechnical design and construction recommendations for the fender system modifications at the Maine State Ferry Terminal located in Frenchboro, Maine (MaineDOT WIN 22202.00).

SchonewaldEA's work on this project has been completed under a Subconsultant Task Order Agreement with HNTB that is dated May 4, 2018. This report is subject to the limitations contained in the Closure section of the report. A quality assurance review of the geotechnical analyses completed by SchonewaldEA for this project was completed by Stephen J. Rabasca, P.E. of SoilMetrics, LLC located in Cape Elizabeth, Maine.

## PROJECT DESCRIPTION

The location of the Maine State Ferry Terminal located in Frenchboro, Maine is depicted on attached Figure 1 – Locus Plan.

The fender system modifications at the Maine State Ferry Terminal in Frenchboro include the installation of one turning dolphin outboard of the footprint of the existing ferry pen, complete with fender panels and lighting. The proposed turning dolphin will be located approximately 40 feet further offshore from the existing outboard turning dolphin to increase the length of the ferry slip in order to accommodate a larger vessel. The turning dolphin will be supported by eleven post-tensioned concrete-filled steel pipe piles. The pipe piles will be post-tensioned by way of threaded-bar rock anchors that are grouted into rock. Each pile will be socketed into bedrock to serve as a seal for rock anchor construction.

The construction of the proposed turning dolphin will include the removal of two existing timber-pile dolphins that are within the footprint or in close proximity to the new dolphin. Appurtenant work includes improvements to the existing chocks and for access, as well as modifications to the electrical system and navigational aids.

## GEOLOGICAL SETTING

Surficial geology is mapped as thin glacial till deposits over shallow bedrock (thin drift) (Surficial Geologic Map of Maine). Bedrock geology is mapped as Plutonic Rocks of Devonian age, specifically granite (Bedrock Geologic Map of Maine).

## SUBSURFACE INVESTIGATION

SchonewaldEA retained New England Boring Contractors of Hermon, Maine and Prock Marine of Rockland, Maine to provide the off-shore drilling services necessary for this project and similar fender modification projects at four other Maine State Ferry Terminals around Penobscot Bay. Due to the significant effort and cost to mobilize the barge-mounted drill rig, five ferry terminals were included in the subsurface exploration program. These included Frenchboro, Swans Island, Vinalhaven, Bass Harbor, and Islesboro. As part of the overall program, two test borings were drilled at the Frenchboro terminal in the vicinity of the proposed turning dolphin to evaluate subsurface conditions. The test borings were designated MB-FBORO-101 and -102. The test borings were drilled on August 13 through 15, 2018 and were observed and logged by SchonewaldEA. The approximate locations of the test borings are depicted on Figure 2 – Boring Location Plan that is included with the Figures.

The test borings were drilled using a barge-mounted drill rig and were completed using standard cased wash boring techniques. The borings were advanced through overburden to refusal and 19.6 to 26.6 feet of rock core was obtained. Because the design of the dolphin foundation does not rely on achieving geotechnical capacity in the overburden soils, limited sampling and testing of overburden soils was completed with the objective of identifying potential constructability issues. Standard Penetration Tests (SPTs) were completed and split-spoon soil samples were obtained at select depths to define the soil stratigraphy. During SPT sampling, the sampler is driven 24 inches and the hammer blows for each 6-inch interval of penetration is recorded. The sum of the blows for the second and third intervals is the N-value, or standard penetration resistance. It is customary for the raw field N-values to be corrected based on the relative energy of the actual hammer system utilized to complete the SPTs. The corrected N-values are referred to as $N_{60}$ values, which are used in correlations and analyses to evaluate the engineering characteristics of the overburden soils. SPTs for this project were conducted using a calibrated auto-hammer; the auto-hammer efficiency factor is provided on the boring logs and was used to obtain the $N_{60}$ values that are also reported on the logs.

The logs of the subsurface explorations are included as Appendix A. Photographs of the rock core collected from the test borings are included as Appendix B.

## LABORATORY TESTING

Representative specimens of the rock core obtained in the test borings were submitted to the GeoTesting Express (GTX) geotechnical laboratory in Acton, Massachusetts for unconfined (uniaxial) compressive strength testing. The unconfined compression tests on rock were conducted in accordance with ASTM D-7012 Method D (Elastic Moduli of Rock in Uniaxial Compression) and included specimen preparation per ASTM D-4543, measuring strength, developing a stress-strain curve, reporting elastic moduli (Young's Modulus and Poisson's Ratio), and providing before and after photographs of the test specimens. The laboratory testing program is summarized in the following table.

| Boring No. | Sample No. | Specimen Depth | Sample Representative of: Tests Performed: |
|---|---|---|---|
| MB-FBORO-101 | R1 | 40.5-40.9 ft. BGS | bedrock; uniaxial compressive strength test |
| MB-FBORO-101 | R4 | 54.4-54.9 ft. BGS | bedrock; uniaxial compressive strength test |
| MB-FBORO-102 | R1 | 37.3-37.7 ft. BGS | bedrock; uniaxial compressive strength test |
| MB-FBORO-102 | R2 | 41.3-42.0 ft. BGS | bedrock; uniaxial compressive strength test |

Subsurface conditions, including the results of the rock testing, are discussed in the following section. Laboratory test results are summarized on the test boring logs included in Appendix A and the laboratory test reports are included as Appendix C.

## SUBSURFACE CONDITIONS

Subsurface conditions encountered in the two Frenchboro test borings were relatively consistent with respect to the depth and type of overburden that was encountered over granite bedrock. Overburden soils encountered in test boring MB-FBORO-101, that was located approximately 50 feet outboard from an existing timber-pile dolphin, consisted of an approximately 28-foot thickness of loose to medium dense, silty sand with varying amounts of gravel and shells (alluvium) overlying 7 feet of soft marine silt-clay that was underlain by approximately 4 feet of silty sand and gravel (glacial till). In test boring MB-FBORO-102, that was located immediately outboard of the existing timber-pile dolphin, approximately 26 feet of very loose to loose alluvium underlain by an approximately 10-foot thickness of glacial till. Bedrock underlying the overburden soils in both test borings consisted of granite. Detailed descriptions of the overburden encountered in the test borings are provided on the logs included in Appendix A.

Bedrock was encountered at approximate elevation -54.2 feet in MB-FBORO-101 and at approximate elevation -47.4 feet in MB-FBORO-102. The bedrock core obtained from the test borings consisted of very hard, typically fresh, fine- to medium-grained, pink-grey, biotite-rich granite. Granite, a plutonic igneous rock, is mapped in the area and is considered a very hard, crystalline rock.

The Rock Quality Designations (RQDs) of the rock cores obtained in the test borings ranged from 33 to 93 percent. Four specimens of the rock core were submitted for uniaxial compressive strength tests. The test results reported the uniaxial compressive strength of the granite ranged between 7.1 and 27.5 ksi.

Detailed descriptions of the rock encountered in the Frenchboro test borings are provided on the logs included in Appendix A.

Note that bedrock encountered in test borings drilled at multiple Maine State Ferry Terminal sites as part of the overall subsurface program, specifically the test borings at Vinalhaven, Frenchboro, Bass Harbor, and Swans Island, were all very hard, crystalline igneous rock.

## GEOTECHNICAL DESIGN AND CONSTRUCTION RECOMMENDATIONS

SchonewaldEA provides the following geotechnical recommendations for the design and construction of the fender system modifications at the Maine State Ferry Terminal located in Frenchboro, Maine. These recommendations are based on geotechnical provisions set forth in the AASHTO LRFD Bridge Design Specifications, 7$^{th}$ Edition, 2014 (LRFD Manual), as well as established ASD methods in the absence of definitive guidance in the LRFD Manual. The calculations included as Appendix D provide specific references.

### BEDROCK COMPRESSIVE STRENGTH AND QUALITY

Because the bedrock underlying the Maine State Ferry Terminal sites in Frenchboro, Swans Island, Vinalhaven, and Bass Harbor were all similar very hard, crystalline igneous rock, we considered the rock core descriptions, RQDs, and laboratory uniaxial compressive strength test results of the four sites in aggregate. A summary of the rock quality and compressive strength test results is provided with the calculations provided as Appendix D.

By taking the laboratory test data in aggregate, SchonewaldEA recommends using a peak uniaxial compressive strength of 15.6 ksi for the granite underlying the Frenchboro site.

### TURNING DOLPHINS - GENERAL

The structural analysis of the turning dolphin was completed by HNTB using FB Multipier software. HNTB identified the critical design scenario to be the greatest elevation difference between the top of the dolphin and bedrock. For the four Maine State Ferry Terminal sites underlain by very hard, crystalline igneous rock, boring MB-BASS-101 at Bass Harbor was the critical case. HNTB and SchonewaldEA concurred that the surficial Organic Muck layer encountered in several of the borings completed at the Maine State Ferry Terminal sites could be neglected as it would not be expected to contribute resistance. The following geotechnical input was recommended by SchonewaldEA for use in HNTB's turning dolphin model and is based on the overburden and bedrock conditions encountered in the Bass Harbor boring:

| LAYER | DESCRIPTION | LATERAL SOIL MODEL | PARAMETERS (assign top and bottom) |
|---|---|---|---|
| 1 | 4-ft thick layer FINE SANDY SILT<br>Top elevation -36.8 ft | Sand (Reese) | Unit weight = 110 pcf<br>Friction angle = 29 deg |
| 2 | 18-ft thick layer MARINE SILT-CLAY<br>Top elevation -40.8 ft | Clay (Soft, Matlock) | Unit weight = 110 pcf<br>Undrained shear strength = 400 psf |
| 3 | 3.5-ft thick layer GLACIAL TILL<br>Top elevation -58.5 ft | Sand (Reese) | Unit weight = 120 pcf<br>Friction angle = 32 deg |
| 4 | HARD ROCK (GRANITE)<br>Top elevation -62 ft | GRANITE | Effective unit weight = 170 pcf<br>Uniaxial compressive strength = 2,160,000 psf |

TURNING DOLPHINS – PIPE PILES

The turning dolphin will be supported by eleven post-tensioned concrete-filled steel pipe piles. Each pile will be socketed into competent bedrock to serve as a seal for rock anchor construction. The nominal resistance of piles bearing on hard rock, like the granite underlying Frenchboro, is controlled by the structural limit state (structural capacity), not geotechnical capacity, per the LRFD Manual, Article 10.7.3.2.3.

Regardless, the geotechnical axial capacity of the pipe piles was checked. The analysis is included with the geotechnical calculations provided as Appendix D. In the absence of definitive guidance in the LRFD Manual, the geotechnical end-bearing capacity of the pipe piles was evaluated using well-established ASD methods after Duncan C. Wyllie's "Foundations on Rock" - Section 8.3.2 "Rock Socketed Piers - End-Bearing Capacity." Per Wyllie, the allowable end-bearing capacity of rock socketed piles, which includes a Factor of Safety of 2 to 3, is a function of the uniaxial compressive strength of the rock at the base of the pile. Working through Wyllie's analysis and assuming a Factor of Safety of 2 to be consistent with recommended resistance factors for end-bearing piles on rock per the LRFD Manual ($\varphi_{stat} = 0.5$), the ultimate end-bearing capacity can be considered roughly equivalent to the nominal resistance of the pile. The geotechnical capacity of the pipe piles was found to be well in excess of the required capacity determined by HNTB, which we understand includes an additional axial load associated with prestressing the rock anchors. Additional capacity achieved via skin friction on the rock socket side wall was conservatively neglected.

The pipe piles that will support the turning dolphin should be socketed into bedrock a minimum of 2 feet to seal the pipe pile into the bedrock and allow the construction of the rock anchors. The rock socket length should be measured in the field from the low side of the rock, which is the depth at which the pile is seated into bedrock around its entire circumference.

The rock sockets should be drilled using rotary down hole hammers, rotary percussive methods, or solid rock coring techniques. No attempt should be made to drive the pipe piles into bedrock. The completed rock sockets should be cleaned of all debris to the extent practicable. A grout plug should be installed in the bottom of the pipe pile prior to commencing coring for the rock anchor.

TURNING DOLPHINS – ROCK ANCHORS

The pipe piles will be post-tensioned using threaded bar grouted into holes drilled into the underlying rock. The design calls for 1-3/8-inch diameter threaded bar installed in a 4-1/2-inch diameter core hole and grouted using 5,000 psi cement grout. A minimum 15-foot long bonded length is specified, which we understand was based in part on HNTB's design analyses that evaluated the pull-out resistance of the cone of rock that would be "dislodged" or "lifted" by the tensioned rock anchor. HNTB's rock anchor design calls for corrosion protection and for sheathing to serve as a grout-threaded bar bond break above the bonded length to ensure adequate free stressing (unbonded) length.

The core hole should be advanced into rock using methods that do not "polish" the rock wall. The core hole should extend a minimum of 12 inches below the required bottom of the bonded length to provide for drilling debris that may not be removed during flushing operations.

The geotechnical tensile capacity of the rock anchors was evaluated by SchonewaldEA using established ASD methods in the absence of definitive guidance in the LRFD Manual, Article 11.9.4.2 - "Anchor Pullout Capacity" that is related to the design of anchored retaining walls. Specific ASD guidance included Duncan C. Wyllie's "Foundations on Rock" - Section 9.3.2 "Tension Foundations - Allowable bond stresses and anchor design" and the Post-Tensioning Institute's "Recommendations for Prestressed Rock and Soil Anchors" - Article 6.0 "Design." We note that a resistance factor equal to 1.0 is allowed for per the LRFD Manual since all the rock anchors will be tested. Per Wyllie, the design against failure of the anchor at the grout interfaces requires that the length of the bond zone, and the diameter of the threadbar and drill hole are proportioned such that the average bond stress is less than the working bond strength.

Based on the analysis, the geotechnical capacity of the rock anchors was calculated to be 517 kips. Because of the high strength of the very hard, crystalline igneous rock (granite) that underlies the Frenchboro terminal, the stated geotechnical capacity of the rock anchors is limited by the strength of the cement grout. Rock anchor capacity calculations are included with the geotechnical calculations attached as Appendix D.

Selecting the location (depth into rock) of the top of the bonded length must recognize that stresses are concentrated near the top of the bonded length. Therefore, the quality of the rock core obtained from the Frenchboro test borings, together with the rock core from the borings drilled at Swans Island, Bass Harbor, and Vinalhaven, was assessed to determine an appropriate depth below the bedrock surface for the top of the bonded length of the rock anchors. Based on the evaluation, which is included with the calculations in Appendix D, SchonewaldEA recommends that the top of the bonded length of the Frenchboro rock anchors be a minimum of 5 feet below the top of the bedrock surface at the pipe pile / rock anchor location.

All the rock anchors should be tested in accordance with industry standards. One rock anchor per dolphin should be performance tested and the remaining rock anchors should be proof tested. Testing should be in accordance with the recommendations set forth in the Post-Tensioning Institute's "Recommendations for Prestressed Rock and Soil Anchors." Those requirements have been incorporated into the project specifications as Special Provision 504 – Structural Steel (Rock Anchors). It is recommended that the performance and proof tests be conducted prior to concreting the steel pipe piles. Testing should also include a lift off test conducted on one rock anchor (per dolphin) as part of the final post-tensioning process.

WAVE SCREENS

The fender system modifications at the Frenchboro terminal do not call for the installation of any timber wave screens.

**CLOSURE**

This report has been prepared for the use of HNTB Corporation for specific application to the fender modifications at the Maine State Ferry Terminal located in Frenchboro, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is expressed or implied.

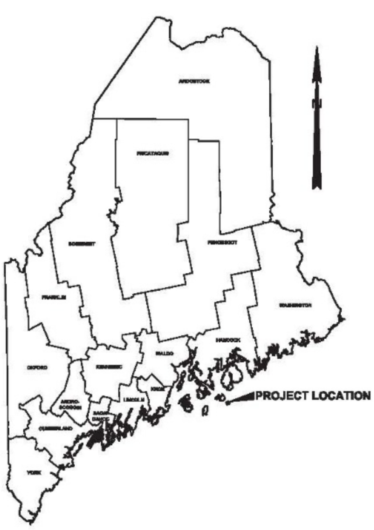
In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes in design. These analyses and recommendations are based in part upon a limited subsurface investigation at discrete exploratory locations completed at the site. If variations from the conditions encountered during the investigation appear evident during construction, it may also become necessary to re-evaluate the recommendations made in this report.

It is recommended that a geotechnical engineer be provided the opportunity for a review of the design and specifications in order that the design recommendations and construction considerations presented in this report are properly interpreted and implemented in the design and specifications.

Schonewald Engineering Associates, Inc.

# FIGURES






(BASE PLANS TAKEN FROM SHEET 1 OF PROJECT PLAN SET PREPARED BY HNTB)

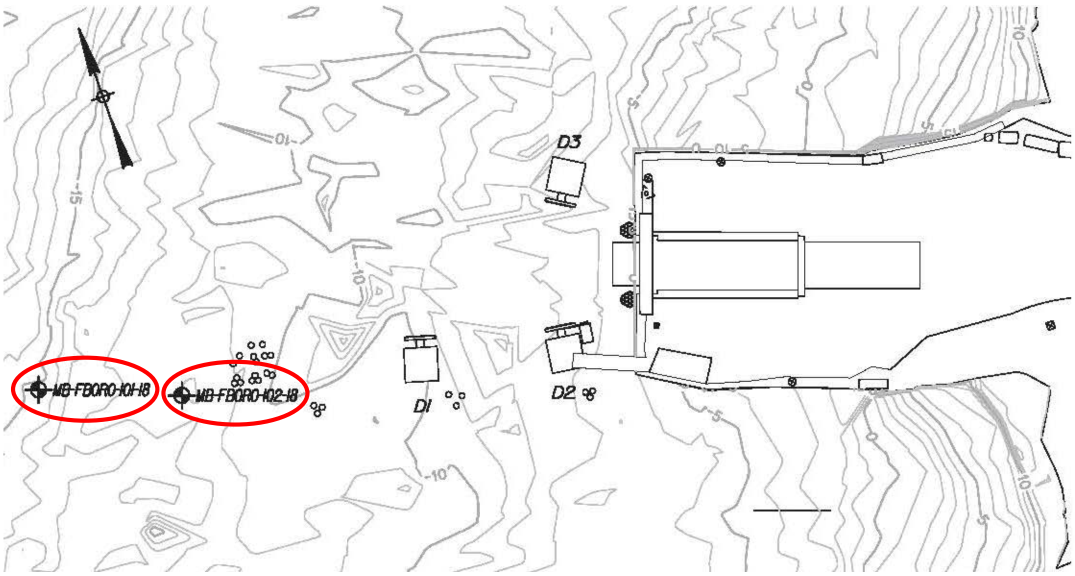
SCHONEWALD ENGINEERING ASSOCIATES, INC.

| | |
|---|---|
| PROJECT NO.: | 18-012 |
| DATE: | MAY 2019 |
| DRAWN BY: | IVS |
| APPROX. SCALE: | AS NOTED |

LOCUS PLAN
FENDER SYSTEM MODIFICATIONS
MAINE STATE FERRY TERMINAL
FRENCHBORO, MAINE
MAINEDOT WIN 22202.00

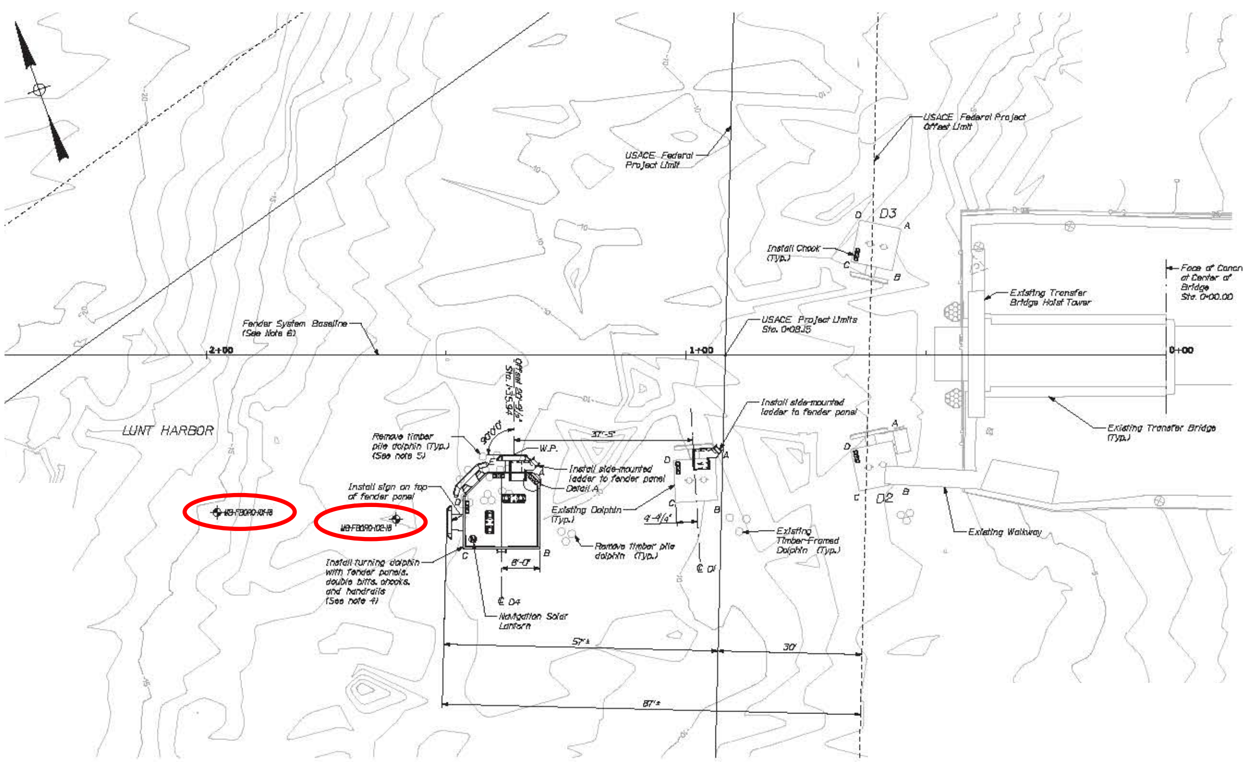
Figure No.:

1




SCALE
1" ≈ 50'

BORING LOCATION PLAN
RELATIVE TO EXISTING CONDITIONS
(BASE PLAN TAKEN FROM SHEET 4 OF PROJECT PLAN SET PREPARED BY HNTB)




SCALE
1" ≈ 40'

BORING LOCATION PLAN
RELATIVE TO PROPOSED CONDITIONS
(BASE PLAN TAKEN FROM SHEET 6 OF PROJECT PLAN SET PREPARED BY HNTB)

SCHONEWALD ENGINEERING ASSOCIATES, INC.

| | |
|---|---|
| PROJECT NO.: | 18-012 |
| DATE: | MAY 2019 |
| DRAWN BY: | IVS |
| APPROX. SCALE: | AS NOTED |

BORING LOCATION PLAN
FENDER SYSTEM MODIFICATIONS
MAINE STATE FERRY TERMINAL
FRENCHBORO, MAINE
MAINEDOT WIN 22202.00

Figure No.:

2

SCHONEWALD ENGINEERING ASSOCIATES, INC.

# APPENDIX A

## SUBSURFACE EXPLORATION LOGS

| Schonewald Engineering Associates, Inc. | PROJECT: Fender System Modifications<br>Maine State Ferry Terminal<br>LOCATION: Frenchboro, Maine | Boring No.: MB-FBORO-101-18<br>WIN: 22202.00 |
|---|---|---|

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | New England Boring Contractors | **Elevation (ft.)** | -15.4 ft (mudline) | **Core Barrel:** | NQ2 (wireline) |
| **Operator:** | Enos / Share | **Datum:** | MLLW | **Sampler:** | standard split spoon |
| **Logged By:** | Schonewald | **Rig Type:** | Mobile Drill B-53 | **Hammer Wt./Fall:** | 140 lbs / 30 inches |
| **Date Start/Finish:** | 8/14/18; 1450- 8/15/18; 1330 | **Drilling Method:** | cased wash boring | **Hammer Type:** | auto hammer |
| **Boring Location:** | per plan (B5A) | **Casing ID/OD:** | HW to 38.8 ft; NW to 38.8 ft | **Hammer Efficiency:** | 0.906 |
| | | **Auger ID/OD:** | n/a | **Water Level*:** | n/a |

**IN-SITU SAMPLING AND TESTING:**
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt

**ADDITIONAL DEFINITIONS:**
N-uncorrected = N value
$N_{60}$ = N value corrected for hammer efficiency
hammer efficiency = calculated hammer efficiency
$S_u$ = Insitu Field Vane Shear Strength (psf)
R = Rock Core Sample
RQD = Rock Quality Designation (%)

**ADDITIONAL DEFINITIONS:**
WOH = weight of 140lb. hammer
WOR = weight of rods
.
.
.
-- = not recorded

**BOREHOLE ADVANCEMENT METHOD:**
SSA=solid stem auger/HSA=hollow stem auger/RC=roller cone

**LABORATORY TEST RESULTS:**
LL=Liquid Limit / PL=Plastic Limit / PI=Plasticity Index
WC = water content, percent
-#200 = percent fines from grain size analysis
UCT qp = peak compressive strength of rock

| 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 | Lab. Testing Results |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | | | SPIN | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | 16 | | | | |
| | | | | | | | 24 | | | | |
| 5 | | | | | | | RC | | | | |
| | MD | 24/0 | 6.0 - 8.0 | 13-14-8-5 | 3"dia | | 17 | | | MD: No recovery; sampler sheared off. | |
| | | | | | | | 25 | | | | |
| | | | | | | | 26 | | | | |
| | | | | | | | 11 | | | | |
| 10 | 1D | 24/5 | 10.0 - 12.0 | 13-7-5-8 | 12 | 18 | 5 | | | 1D: Grey, fine to medium Sandy ORGANIC SILT with shells and occasional gravel. | |
| | | | | | | | 8 | | | | |
| | | | | | | | 51 | | | | |
| | | | | | | | 41 | | | | |
| | | | | | | | 73 | | | | |
| 15 | | | | | | | 92 | | | | |
| | | | | | | | 42 | | | | |
| | | | | | | | 24 | | | | |
| | | | | | | | 46 | | | | |
| | | | | | | | 57 | | | | |
| 20 | 2D | 24/18 | 20.0 - 22.0 | 2-4-1-2 | 5 | 8 | -- | | | 2D: Grey, Silty fine to medium SAND, trace fine gravel, trace coarse sand with shells throughout. | |
| | | | | | | | 26 | | | | |
| | | | | | | | 48 | | | | |
| | | | | | | | 40 | | | | |
| | | | | | | | 51 | | | | |
| 25 | | | | | | | | | | | |

**Remarks:**

8/14/18; 1418 hrs: 4.4' top outboard (westerly) dolphin to water; 7.7' barge deck to water; 35.3' barge deck to mudline
top of outboard dolphin: elev. 16.60 ft MLLW
100' cleat of outboard dolphin to borehole; approx. 3' northerly of line through cleats on two southerly dolphins.

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.

| SCHONEWALD ENGINEERING ASSOCIATES, INC. | **PROJECT:** Fender System Modifications<br>Maine State Ferry Terminal<br>**LOCATION:** Frenchboro, Maine | **Boring No.:** MB-FBORO-101-18<br>**WIN:** 22202.00 |
|---|---|---|

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | New England Boring Contractors | **Elevation (ft.)** | -15.4 ft (mudline) | **Core Barrel:** | NQ2 (wireline) |
| **Operator:** | Enos / Share | **Datum:** | MLLW | **Sampler:** | standard split spoon |
| **Logged By:** | Schonewald | **Rig Type:** | Mobile Drill B-53 | **Hammer Wt./Fall:** | 140 lbs / 30 inches |
| **Date Start/Finish:** | 8/14/18; 1450- 8/15/18; 1330 | **Drilling Method:** | cased wash boring | **Hammer Type:** | auto hammer |
| **Boring Location:** | per plan (B5A) | **Casing ID/OD:** | HW to 38.8 ft; NW to 38.8 ft | **Hammer Efficiency:** | 0.906 |
| | | **Auger ID/OD:** | n/a | **Water Level*:** | n/a |

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UCT qp = peak compressive strength of rock

| 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 | Lab. Testing Results |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 25 | | | | | | | 79 | | | | |
| | | | | | | | 97 | | | | |
| | | | | | | | 121 | | | | |
| | | | | | | | 61 | -43.4 | | 28.0 | |
| | | | | | | | 66 | | | | |
| 30 | 3D | 24/18 | 30.0 - 32.0 | 4-2-2-3 | 4 | 6 | RC | | | 3D: Dark grey, soft, Silty CLAY, trace very fine sand. | |
| 35 | | | | | | | | -50.4 | | 35.0<br>35 ft: Gravel noted; possibly till interface. | |
| | R1 | 60/60 | 38.8 - 43.8 | RQD: 38"=63% | | | NQ2 | -54.2 | | 38.8<br>Top of bedrock at Elev. -54.2 ft.<br>R1: Bedrock: Very hard, fresh, fine to medium grained, pink grey biotite-rich GRANITE with occasional cracks. Close, low angle to moderately dipping breaks; undulating, rough, fresh to discolored, and open with occasional infilling.<br>(Devonian Granite Pluton)<br>Core times: 2:35/ 3:10/ 3:00/ 2:55/ 2:35 min:sec/ft<br>ROCK QUALITY = FAIR | UCT qp= 16.78 ksi |
| 40 | | | | sample 40.5-40.9 | | | | | | | |
| | R2 | 60/59 | 43.8 - 48.8 | RQD: 44"=73% | | | | | | R2: Similar to R1, except close to moderately spaced breaks.<br>3:05/ 3:35/ 3:45/ 3:50/ 4:00 min:sec/ft<br>ROCK QUALITY = FAIR TO GOOD | |
| 45 | | | | | | | | | | | |
| | R3 | 55/55 | 48.8 - 53.4 | RQD: 44"=80% | | | | | | R3: Similar to R1, except one high angle 1/4-inch thick quartzite vein and moderately spaced breaks. | |
| 50 | | | | | | | | | | | |

**Remarks:**

8/14/18; 1418 hrs: 4.4' top outboard (westerly) dolphin to water; 7.7' barge deck to water; 35.3' barge deck to mudline
top of outboard dolphin: elev. 16.60 ft MLLW
100' cleat of outboard dolphin to borehole; approx. 3' northerly of line through cleats on two southerly dolphins.

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.

| Schonewald Engineering Associates, Inc. | **PROJECT:** Fender System Modifications, Maine State Ferry Terminal<br>**LOCATION:** Frenchboro, Maine | **Boring No.:** MB-FBORO-101-18<br>**WIN:** 22202.00 |
|---|---|---|

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | New England Boring Contractors | **Elevation (ft.)** | -15.4 ft (mudline) | **Core Barrel:** | NQ2 (wireline) |
| **Operator:** | Enos / Share | **Datum:** | MLLW | **Sampler:** | standard split spoon |
| **Logged By:** | Schonewald | **Rig Type:** | Mobile Drill B-53 | **Hammer Wt./Fall:** | 140 lbs / 30 inches |
| **Date Start/Finish:** | 8/14/18; 1450- 8/15/18; 1330 | **Drilling Method:** | cased wash boring | **Hammer Type:** | auto hammer |
| **Boring Location:** | per plan (B5A) | **Casing ID/OD:** | HW to 38.8 ft; NW to 38.8 ft | **Hammer Efficiency:** | 0.906 |
| | | **Auger ID/OD:** | n/a | **Water Level*:** | n/a |

**IN-SITU SAMPLING AND TESTING:**
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
MU = Unsuccessful Thin Wall Tube Sample attempt
V = Insitu Vane Shear Test
MV = Unsuccessful Insitu Vane Shear Test attempt

**ADDITIONAL DEFINITIONS:**
N-uncorrected = N value
$N_{60}$ = N value corrected for hammer efficiency
hammer efficiency = calculated hammer efficiency
$S_u$ = Insitu Field Vane Shear Strength (psf)
R = Rock Core Sample
RQD = Rock Quality Designation (%)

**ADDITIONAL DEFINITIONS:**
WOH = weight of 140lb. hammer
WOR = weight of rods
.
.
.
-- = not recorded

**BOREHOLE ADVANCEMENT METHOD:**
SSA=solid stem auger/HSA=hollow stem auger/RC=roller cone

**LABORATORY TEST RESULTS:**
LL=Liquid Limit / PL=Plastic Limit / PI=Plasticity Index
WC = water content, percent
-#200 = percent fines from grain size analysis
UCT qp = peak compressive strength of rock

| 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 | Lab. Testing Results |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 50 | | | | | | | | | | 5:55/ 7:10/ 10:55/ 8:45/ - min:sec/ft<br>ROCK QUALITY = GOOD | |
| | R4 | 60/60 | 53.4 - 58.4 | RQD: 49"=82% | | | | | | | |
| | | | | sample 54.4-54.9 | | | | | | R4: Similar to R1, except one calcsilicate- filled crack (parent rock altered along crack margins) and close to moderately spaced breaks.<br>4:45/ 4:45/ 5:05/ 4:25/ 5:10 min:sec/ft<br>ROCK QUALITY = GOOD | |
| 55 | | | | | | | | | | | UCT qp= 7.11 ksi |
| | | | | | | | | -73.8 | | 58.4<br>**Bottom of Exploration at 58.4 feet below ground surface.** | |
| 60 | | | | | | | | | | | |
| 65 | | | | | | | | | | | |
| 70 | | | | | | | | | | | |
| 75 | | | | | | | | | | | |

**Remarks:**

8/14/18; 1418 hrs: 4.4' top outboard (westerly) dolphin to water; 7.7' barge deck to water; 35.3' barge deck to mudline
top of outboard dolphin: elev. 16.60 ft MLLW
100' cleat of outboard dolphin to borehole; approx. 3' northerly of line through cleats on two southerly dolphins.

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.

| SCHONEWALD ENGINEERING ASSOCIATES, INC. | **PROJECT:** Fender System Modifications<br>Maine State Ferry Terminal<br>**LOCATION:** Frenchboro, Maine | **Boring No.:** MB-FBORO-102-18<br>**WIN:** 22202.00 |
|---|---|---|

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | New England Boring Contractors | **Elevation (ft.)** | -11.8 ft (mudline) | **Core Barrel:** | NQ2 (wireline) |
| **Operator:** | Enos / Share | **Datum:** | MLLW | **Sampler:** | standard split spoon |
| **Logged By:** | Schonewald | **Rig Type:** | Mobile Drill B-53 | **Hammer Wt./Fall:** | 140 lbs / 30 inches |
| **Date Start/Finish:** | 8/13/18; 1515- 8/14/18; 1355 | **Drilling Method:** | cased wash boring | **Hammer Type:** | auto hammer |
| **Boring Location:** | per plan (B5B) | **Casing ID/OD:** | HW to 35 ft; NW to 35.6 ft | **Hammer Efficiency:** | 0.906 |
| | | **Auger ID/OD:** | n/a | **Water Level*:** | n/a |

**IN-SITU SAMPLING AND TESTING:**
D = Split Spoon Sample
MD = Unsuccessful Split Spoon Sample attempt
U = Thin Wall Tube Sample
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WOH = weight of 140lb. hammer
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.
.
.
-- = not recorded

**BOREHOLE ADVANCEMENT METHOD:**
SSA=solid stem auger/HSA=hollow stem auger/RC=roller cone
**LABORATORY TEST RESULTS:**
LL=Liquid Limit / PL=Plastic Limit / PI=Plasticity Index
WC = water content, percent
-#200 = percent fines from grain size analysis
UCT qp = peak compressive strength of rock

| 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 | Lab. Testing Results |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | 1D | 24/5 | 0.0 - 2.0 | 38-20-17-14 | 3"dia | | 9 | | | 1D: Brown grey, GRAVEL, trace to little fine to coarse sand, trace silt. RECENT MARINE SEDIMENT | |
| | | | | | | | 47 | | | | |
| | | | | | | | 73 | | | | |
| | | | | | | | 49 | | | | |
| | | | | | | | 26 | | | | |
| 5 | 2D | 24/2 | 5.5 - 7.5 | 11-3-2-2 | 5 | 8 | -- | | | 2D: Grey. loose, Silty fine to coarse SAND, trace fine gravel; appears to be mostly wash. | |
| | | | | | | | 27 | | | | |
| | | | | | | | 40 | | | | |
| | | | | | | | 36 | | | | |
| | | | | | | | 28 | | | | |
| 10 | 3D | 24/24 | 10.0 - 12.0 | 7-5-1-2 | 6 | 9 | RC | | | 3D: Brown grey, fine to medium Sandy ORGANIC SILT, little gravel, trace coarse sand with numerous shells. | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| 15 | 4D | 24/10 | 15.0 - 17.0 | WOH/12"-2-2 | 2 | 3 | PUSH | | | 4D: Grey, soft, Silty CLAY, little fine to medium sand with numerous shell fragments and occasional fine gravel and coarse sand. | |
| | | | | | | | | | | | |
| | | | | | | | | | | | |
| | | | | | | | 46 | | | | |
| | | | | | | | 44 | | | | |
| 20 | 5D | 24/20 | 20.0 - 22.0 | 4-1-1/12" | 2 | 3 | 24 | | | 5D: Grey, v. loose, interbedded, Silty fine to medium SAND with shells; Silty CLAY with shells and random pieces of fine gravel; and fine to medium SAND, little to some silt with shells. | |
| | | | | | | | 26 | | | | |
| | | | | | | | 33 | | | | |
| | | | | | | | 36 | | | | |
| | | | | | | | 74 | | | | |
| 25 | | | | | | | | | | | |

**Remarks:**

8/13/18; 1505 hrs: 8.0' top outboard (westerly) dolphin to water; 7.7' barge deck to water; 28.1' barge deck to mudline
top of outboard dolphin: elev. 16.60 ft MLLW
63' cleat of outboard dolphin to borehole; approx. 2' southerly of line through cleats on two southerly dolphins.

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.

| SCHONEWALD ENGINEERING ASSOCIATES, INC. | **PROJECT:** Fender System Modifications Maine State Ferry Terminal **LOCATION:** Frenchboro, Maine | **Boring No.:** MB-FBORO-102-18 **WIN:** 22202.00 |
|---|---|---|

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | New England Boring Contractors | **Elevation (ft.)** | -11.8 ft (mudline) | **Core Barrel:** | NQ2 (wireline) |
| **Operator:** | Enos / Share | **Datum:** | MLLW | **Sampler:** | standard split spoon |
| **Logged By:** | Schonewald | **Rig Type:** | Mobile Drill B-53 | **Hammer Wt./Fall:** | 140 lbs / 30 inches |
| **Date Start/Finish:** | 8/13/18; 1515- 8/14/18; 1355 | **Drilling Method:** | cased wash boring | **Hammer Type:** | auto hammer |
| **Boring Location:** | per plan (B5B) | **Casing ID/OD:** | HW to 35 ft; NW to 35.6 ft | **Hammer Efficiency:** | 0.906 |
| | | **Auger ID/OD:** | n/a | **Water Level*:** | n/a |

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.
.
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LL=Liquid Limit / PL=Plastic Limit / PI=Plasticity Index
WC = water content, percent
-#200 = percent fines from grain size analysis
UCT qp = peak compressive strength of rock

| 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 | Lab. Testing Results |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 25 | | | | | | | RC | -37.8 | | 26.0 | |
| 30 | 6D | 18/7 | 30.0 - 31.5 | 7-18-50/6" | 68 | 103 | RC | | | 6D: Grey, Silty fine to medium SAND, some gravel, trace coarse sand. TILL | |
| 35 | 7D | 6/6 | 35.0 - 35.5 | 9-30/0" | | | NQ2 | | | 7D: Grey, Silty GRAVEL, some fine to coarse sand. | |
| | R1 | 60/60 | 35.6 - 40.6 | RQD: 20"=33% | | | | -47.4 | | 35.6 Top of bedrock at Elev. -47.4 ft. R1: Bedrock: Very hard, fresh, fine to medium grained, pink grey biotite-rich GRANITE. Close, typically low angle breaks; undulating, rough, fresh to discolored, and open with occasional lime infilling. (Devonian Granite Pluton) Core times: 3:10/ 1:45/ 1:45/ 2:10/ 2:05 min:sec/ft ROCK QUALITY = POOR | |
| | | | | sample 37.3-37.7 | | | | | | | UCT qp= 26.19 ksi |
| 40 | R2 | 60/60 | 40.6 - 45.6 | RQD: 37"=62% | | | | | | R2: Similar to R1, except one near vertical infilled crack from 43.1 to 44. 6 ft (parent rock altered along crack margins); moderately spaced breaks; and open fracture from 40.6 to 40.8 ft. --/ 1:35/ 1:45/ 2:00/ 1:55 min:sec/ft ROCK QUALITY = FAIR | |
| | | | | sample 41.3-42.0 | | | | | | | UCT qp= 27.47 ksi |
| 45 | R3 | 25/24 | 45.6 - 47.7 | RQD: 16"=64% | | | | | | R3: Similar to R1, except close breaks below 47.0 ft. 2:10/ 1:50/ - min:sec/ft ROCK QUALITY = FAIR | |
| | R4 | 54/53 | 47.7 - 52.2 | RQD: 50"=93% | | | | | | R4: Similar to R1, except widely spaced breaks. One near vertical infilled crack from 47.7 to 48.9 ft and one inclusion. -/ 2:40/ 2:55/ 2:55/ - min:sec/ft. ROCK QUALITY = EXCELLENT | |
| 50 | | | | | | | | | | | |

**Remarks:**

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63' cleat of outboard dolphin to borehole; approx. 2' southerly of line through cleats on two southerly dolphins.

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| SCHONEWALD ENGINEERING ASSOCIATES, INC. | PROJECT: Fender System Modifications<br>Maine State Ferry Terminal<br>LOCATION: Frenchboro, Maine | Boring No.: MB-FBORO-102-18<br>WIN: 22202.00 |
|---|---|---|

| | | | | | |
|---|---|---|---|---|---|
| **Driller:** | New England Boring Contractors | **Elevation (ft.)** | -11.8 ft (mudline) | **Core Barrel:** | NQ2 (wireline) |
| **Operator:** | Enos / Share | **Datum:** | MLLW | **Sampler:** | standard split spoon |
| **Logged By:** | Schonewald | **Rig Type:** | Mobile Drill B-53 | **Hammer Wt./Fall:** | 140 lbs / 30 inches |
| **Date Start/Finish:** | 8/13/18; 1515- 8/14/18; 1355 | **Drilling Method:** | cased wash boring | **Hammer Type:** | auto hammer |
| **Boring Location:** | per plan (B5B) | **Casing ID/OD:** | HW to 35 ft; NW to 35.6 ft | **Hammer Efficiency:** | 0.906 |
| | | **Auger ID/OD:** | n/a | **Water Level*:** | n/a |

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LL=Liquid Limit / PL=Plastic Limit / PI=Plasticity Index
WC = water content, percent
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UCT qp = peak compressive strength of rock

| 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 | Lab. Testing Results |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 50 | | | | | | | | | | | |
| | R5 | 60/60 | 52.2 - 57.2 | RQD: 50"=83% | | | | | | R5: Similar to R4. One fracture zone at 52. 2 ft.<br>4:35/ 2:45/ 2:25/ 2:45/ 2:55 min:sec/ft<br>ROCK QUALITY = GOOD | |
| 55 | | | | | | | | | | | |
| | R6 | 60/60 | 57.2 - 62.2 | RQD: 52"=87% | | | | | | R6: Similar to R4.<br>2:15/ 2:25/ 2:40/ 2:40/ 2:45 min:sec/ft<br>ROCK QUALITY = GOOD | |
| 60 | | | | | | | | | | | |
| | | | | | | | | -74.0 | | 62.2<br>**Bottom of Exploration at 62.2 feet below ground surface.** | |
| 65 | | | | | | | | | | | |
| 70 | | | | | | | | | | | |
| 75 | | | | | | | | | | | |

**Remarks:**

8/13/18; 1505 hrs: 8.0' top outboard (westerly) dolphin to water; 7.7' barge deck to water; 28.1' barge deck to mudline
top of outboard dolphin: elev. 16.60 ft MLLW
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SCHONEWALD ENGINEERING ASSOCIATES, INC.

# APPENDIX B

# ROCK CORE PHOTOGRAPHS

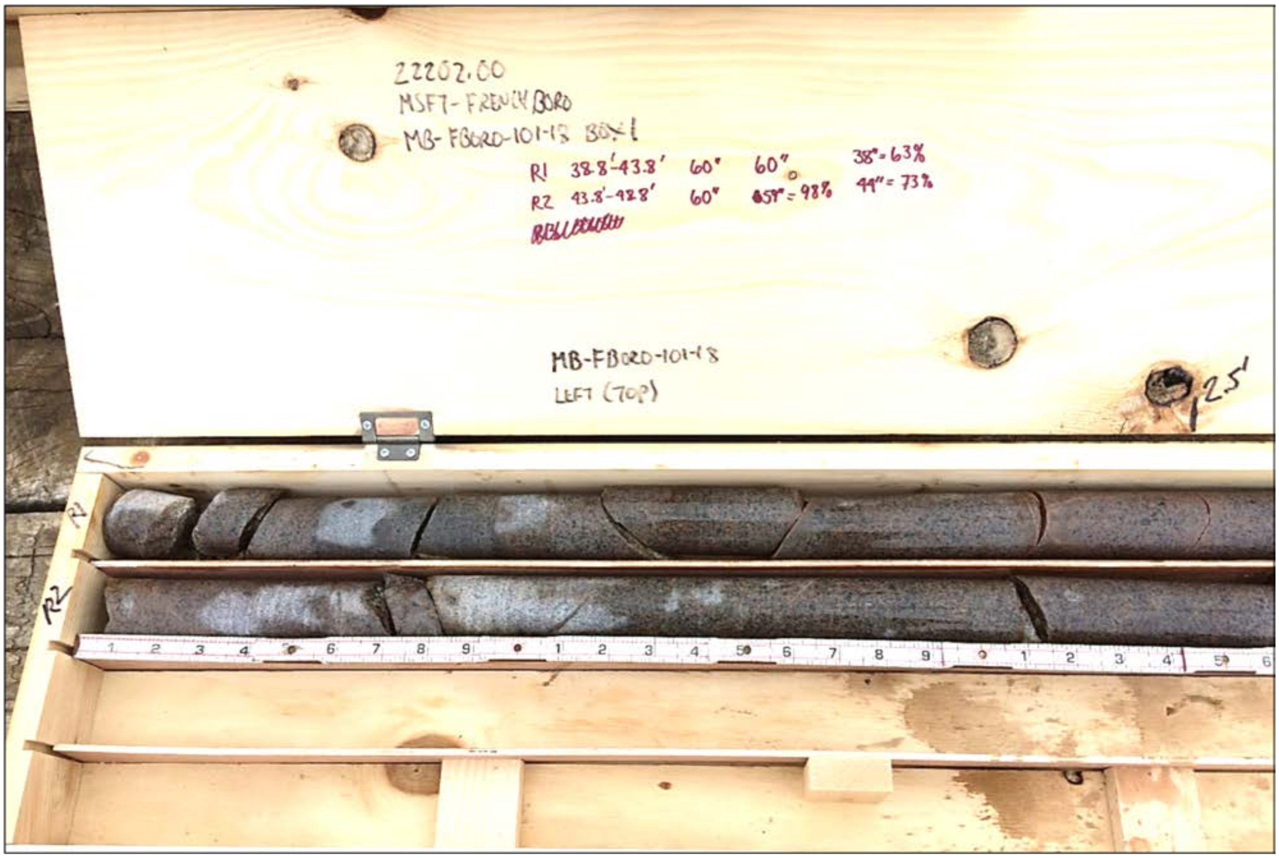



Photo 1: Core box containing wetted rock core from test boring MB-FBORO-101; left side of core box (top portion of cores). Slots from top to bottom:

1) MB-FBORO-101, R1;
2) MB-FBORO-101, R2;
3) Empty;
4) Empty.


Photo 2: Core box containing wetted rock core from test boring MB-FBORO-101; right side of core box (bottom portion of cores). Slots from top to bottom:

1) MB-FBORO-101, R1;
2) MB-FBORO-101, R2;
3) Empty;
4) Empty.

SCHONEWALD ENGINEERING ASSOCIATES, INC.

ROCK CORE PHOTOGRAPHS
FENDER SYSTEM MODIFICATIONS
MAINE STATE FERRY TERMINAL
FRENCHBORO, MAINE
MAINEDOT WIN 22202.00

Sheet No.:

**1 of 4**


Photo 3: Core box containing wetted rock core from test boring MB-FBORO-101; left side of core box (top portion of cores). Slots from top to bottom:

1) MB-FBORO-101, R3;
2) MB-FBORO-101, R4;
3) Empty;
4) Empty.


Photo 4: Core box containing wetted rock core from test boring MB-FBORO-101; right side of core box (bottom portion of cores). Slots from top to bottom:

1) MB-FBORO-101, R3;
2) MB-FBORO-101, R4;
3) Empty;
4) Empty.

SCHONEWALD ENGINEERING ASSOCIATES, INC.

ROCK CORE PHOTOGRAPHS
FENDER SYSTEM MODIFICATIONS
MAINE STATE FERRY TERMINAL
FRENCHBORO, MAINE
MAINEDOT WIN 22202.00

Sheet No.:

**2 of 4**


Photo 5: Core box containing wetted rock core from test boring MB-FBORO-102; left side of core box (top portion of cores). Slots from top to bottom:

1) MB-FBORO-102, R1;
2) MB-FBORO-102, R2;
3) MB-FBORO-102, R3;
4) MB-FBORO-102, R4.


Photo 6: Core box containing wetted rock core from test boring MB-FBORO-102; right side of core box (bottom portion of cores). Slots from top to bottom:

1) MB-FBORO-102, R1;
2) MB-FBORO-102, R2;
3) Empty (MB-FBORO-102, R3);
4) MB-FBORO-102, R4.

SCHONEWALD ENGINEERING ASSOCIATES, INC.

ROCK CORE PHOTOGRAPHS
FENDER SYSTEM MODIFICATIONS
MAINE STATE FERRY TERMINAL
FRENCHBORO, MAINE
MAINEDOT WIN 22202.00

Sheet No.:

**3 of 4**

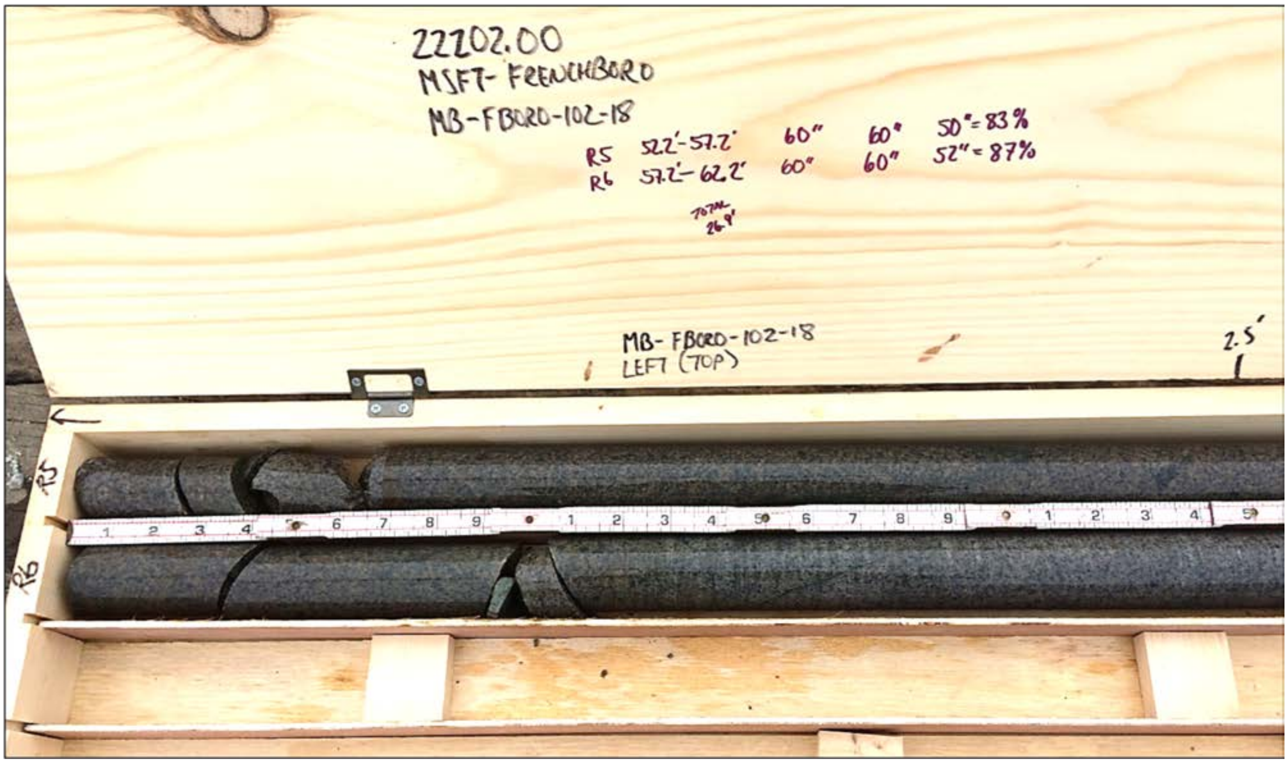



Photo 7: Core box containing wetted rock core from test boring MB-FBORO-102; left side of core box (top portion of cores). Slots from top to bottom:
1) MB-FBORO-102, R5;
2) MB-FBORO-102, R6;
3) Empty;
4) Empty.


Photo 8: Core box containing wetted rock core from test boring MB-FBORO-102; right side of core box (bottom portion of cores). Slots from top to bottom:
1) MB-FBORO-102, R5;
2) MB-FBORO-102, R6;
3) Empty;
4) Empty.

SCHONEWALD ENGINEERING ASSOCIATES, INC.

ROCK CORE PHOTOGRAPHS
FENDER SYSTEM MODIFICATIONS
MAINE STATE FERRY TERMINAL
FRENCHBORO, MAINE
MAINEDOT WIN 22202.00

Sheet No.:

**4 of 4**

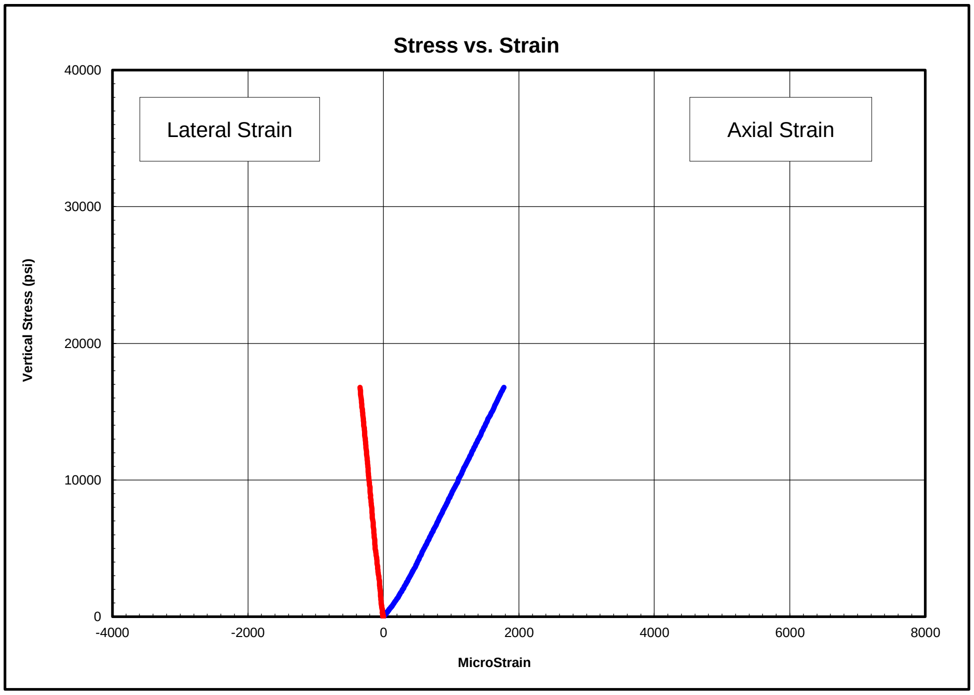
SCHONEWALD ENGINEERING ASSOCIATES, INC.

# APPENDIX C

# LABORATORY TEST REPORTS

| | |
|---|---|
| Client: | Schonewald Engineering Associates, Inc. |
| Project Name: | MeDOT WIN 22202.00 MSFT Frenchboro, ME |
| Project Location: | Frenchboro, ME |
| GTX #: | 308804 |
| Test Date: | 9/25/2018 |
| Tested By: | crs |
| Checked By: | jsc |
| Boring ID: | MB-FBORO-101 |
| Sample ID: | R1 |
| Depth, ft: | 40.5-40.9 |
| Sample Type: | rock core |
| Sample Description: | See photographs<br>Intact material failure |

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




Peak Compressive Stress: 16,779 psi

| Stress Range, psi | Young's Modulus, psi | Poisson's Ratio |
|---|---|---|
| 1700-6200 | 9,570,000 | 0.22 |
| 6200-10600 | 9,970,000 | 0.19 |
| 10600-15100 | 10,000,000 | 0.20 |

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: | Shonewald Engineering Associates, Inc. | Test Date: | 9/20/2018 |
| Project Name: | MeDOT WIN 22202.00 MSFT Frenchboro, ME | Tested By: | crs |
| Project Location: | Frenchboro, ME | Checked By: | jsc |
| GTX #: | 308804 | | |
| Boring ID: | MB-FBORO-101 | | |
| Sample ID: | R1 | | |
| Depth: | 40.5-40.9 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: | 4.39 | 4.39 | 4.39 | | |
| Specimen Diameter, in: | 1.99 | 1.99 | 1.99 | | |
| Specimen Mass, g: | 590.29 | | | | |
| Bulk Density, lb/ft$^3$ | 164 | | | **Minimum Diameter Tolerence Met?** | YES |
| Length to Diameter Ratio: | 2.2 | | | **Length to Diameter Ratio Tolerance Met?** | YES |

**DEVIATION FROM STRAIGHTNESS (Procedure S1)**

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

*Maximum difference must be < 0.020 in.*

**Straightness Tolerance Met?** YES

**END FLATNESS AND PARALLELISM (Procedure FP1)**

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

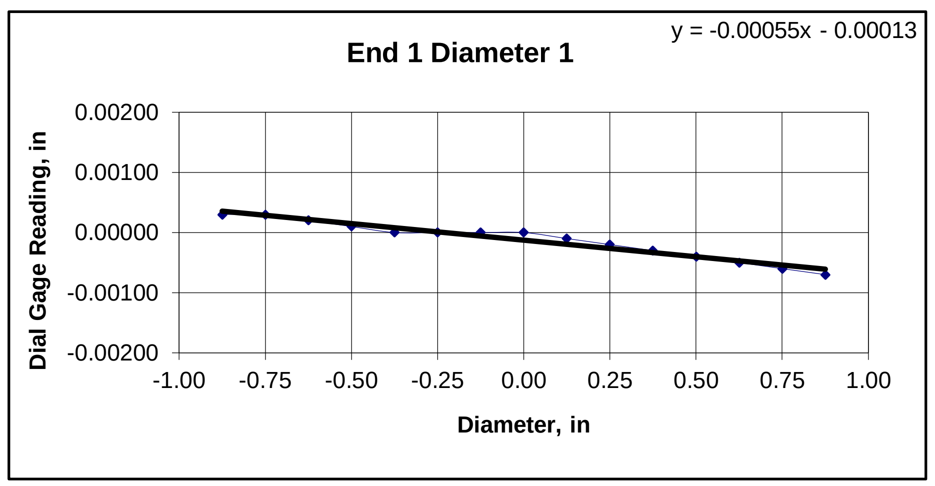
Difference between max and min readings, in:
0° = 0.00100 90° = 0.00120

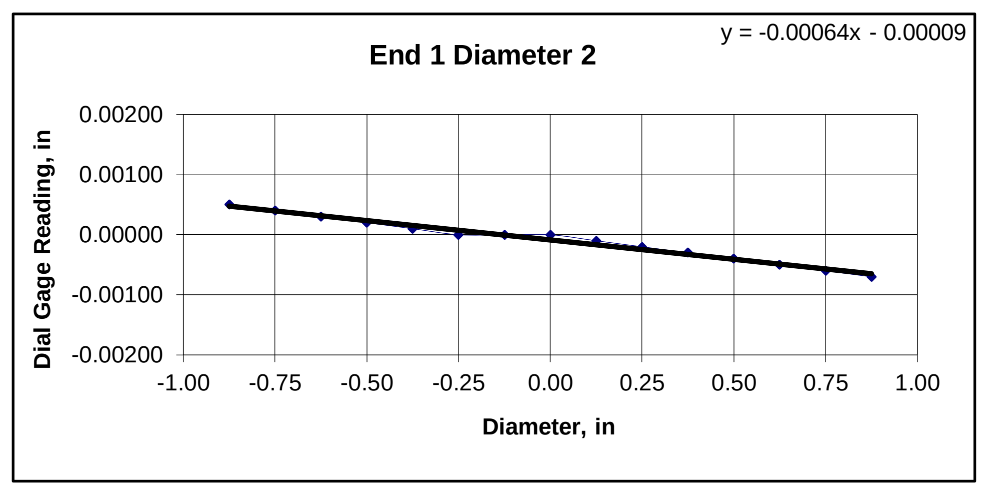
| END 2 | -0.875 | -0.750 | -0.625 | -0.500 | -0.375 | -0.250 | -0.125 | 0.000 | 0.125 | 0.250 | 0.375 | 0.500 | 0.625 | 0.750 | 0.875 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Diameter 1, in | 0.00040 | 0.00030 | 0.00020 | 0.00020 | 0.00010 | 0.00000 | 0.00000 | 0.00000 |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

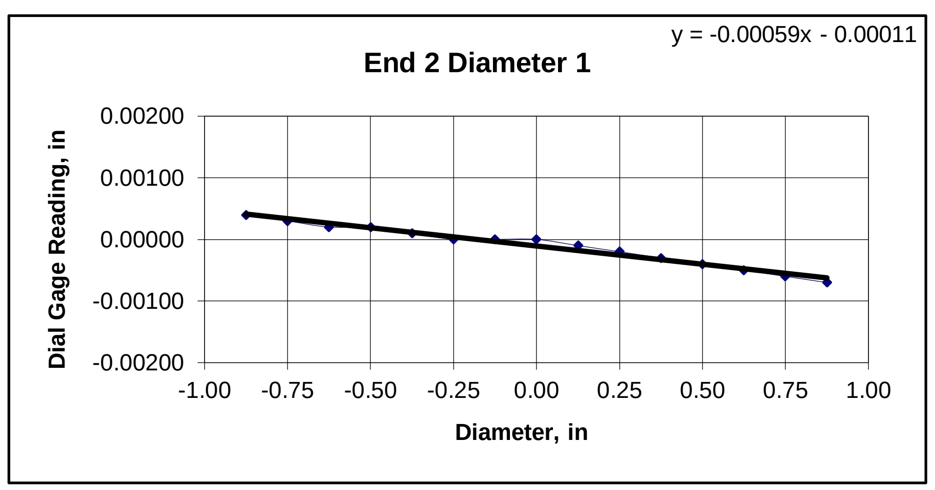
Difference between max and min readings, in:
0° = 0.0011 90° = 0.0012

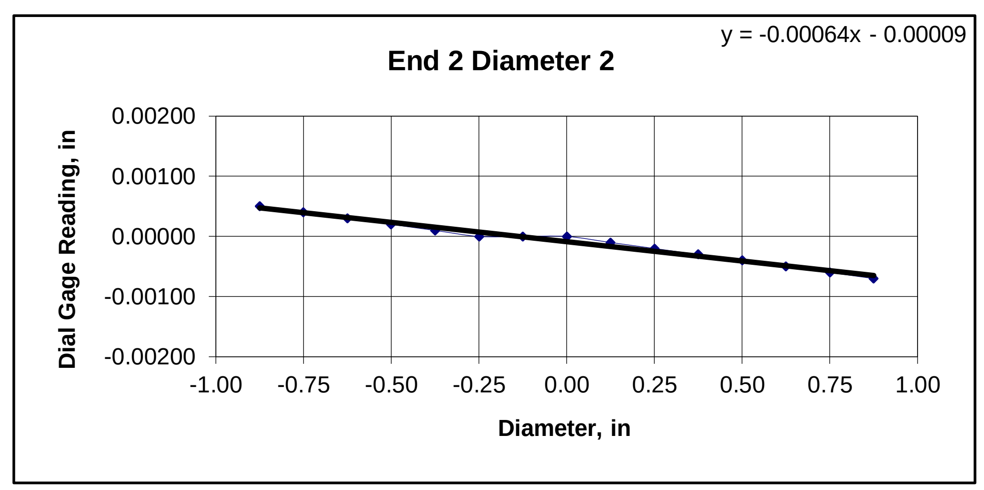
*Maximum difference must be < 0.0020 in.* Difference = $\pm$ 0.00060

**Flatness Tolerance Met?** YES




DIAMETER 1

| | |
|---|---|
| End 1: | |
| Slope of Best Fit Line | 0.00055 |
| Angle of Best Fit Line: | 0.03159 |
| End 2: | |
| Slope of Best Fit Line | 0.00059 |
| Angle of Best Fit Line: | 0.03389 |
| Maximum Angular Difference: | 0.00229 |

**Parallelism Tolerance Met?** YES
Spherically Seated

DIAMETER 2

| | |
|---|---|
| End 1: | |
| Slope of Best Fit Line | 0.00064 |
| Angle of Best Fit Line: | 0.03683 |
| End 2: | |
| Slope of Best Fit Line | 0.00064 |
| Angle of Best Fit Line: | 0.03683 |
| Maximum Angular Difference: | 0.00000 |

**Parallelism Tolerance Met?** YES
Spherically Seated

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

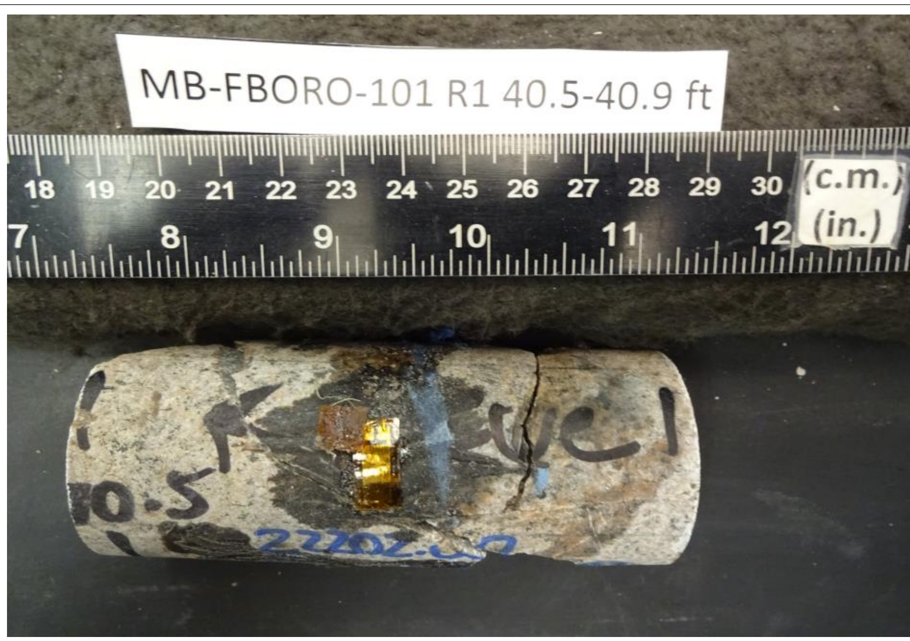
| END 1 | Difference, Maximum and Minimum (in.) | Diameter (in.) | Slope | Angle° | Perpendicularity Tolerance Met? |
|---|---|---|---|---|---|
| Diameter 1, in | 0.00100 | 1.990 | 0.00050 | 0.029 | YES |
| Diameter 2, in (rotated 90°) | 0.00120 | 1.990 | 0.00060 | 0.035 | YES |
| END 2 | | | | | |
| Diameter 1, in | 0.00110 | 1.990 | 0.00055 | 0.032 | YES |
| Diameter 2, in (rotated 90°) | 0.00120 | 1.990 | 0.00060 | 0.035 | YES |

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

**Perpendicularity Tolerance Met?** YES

| | |
|---|---|
| Client: | Schonewald Engineering Associates, Inc. |
| Project Name: | MeDOT WIN 22202.00 MSFT Frenchboro, ME |
| Project Location: | Frenchboro, ME |
| GTX #: | 308804 |
| Test Date: | 9/21/2018 |
| Tested By: | tlm |
| Checked By: | jsc |
| Boring ID: | MB-FBORO-101 |
| Sample ID: | R1 |
| Depth, ft: | 40.5-40.9 |

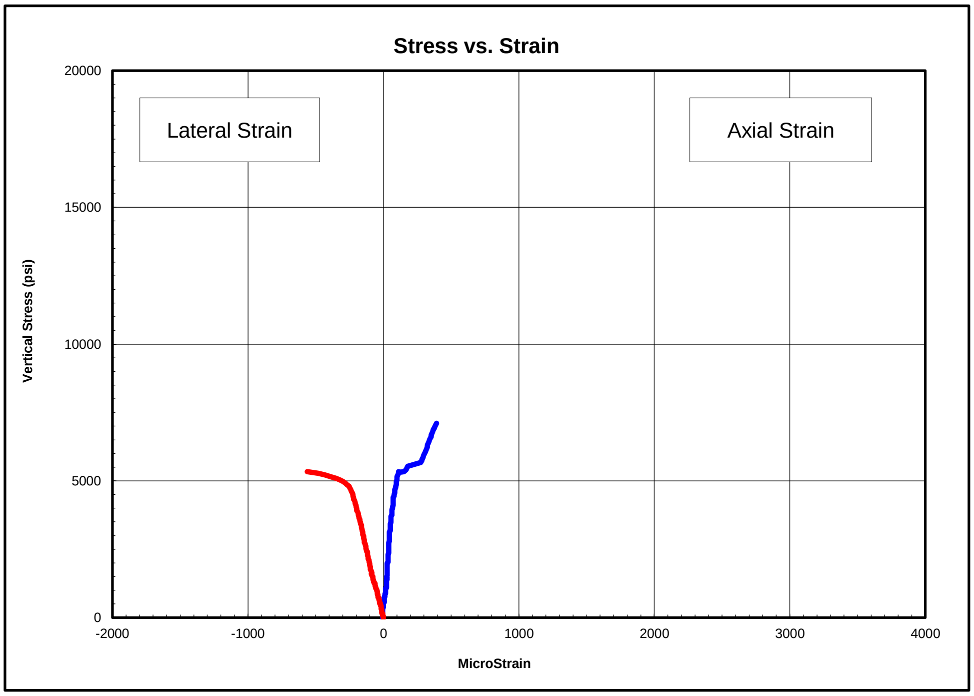

After cutting and grinding




After break

| | |
|---|---|
| Client: | Schonewald Engineering Associates, Inc. |
| Project Name: | MeDOT WIN 22202.00 MSFT Frenchboro, ME |
| Project Location: | Frenchboro, ME |
| GTX #: | 308804 |
| Test Date: | 9/25/2018 |
| Tested By: | crs |
| Checked By: | jsc |
| Boring ID: | MB-FBORO-101 |
| Sample ID: | R4 |
| Depth, ft: | 54.4-54.9 |
| Sample Type: | rock core |
| Sample Description: | See photographs<br>Discontinuity failure |

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




Peak Compressive Stress: 7,107 psi

The lateral strain gauges failed before the peak value was attained. The axial strain gauges picked up an initial failure within the specimen and then continued reading until total failure occured.

| Stress Range, psi | Young's Modulus, psi | Poisson's Ratio |
|---|---|---|
| 700-2600 | 58,700,000 | --- |
| 2600-4500 | 46,400,000 | --- |
| 4500-6400 | --- | --- |

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: | Shonewald Engineering Associates, Inc. | Test Date: | 9/20/2018 |
| Project Name: | MeDOT WIN 22202.00 MSFT Frenchboro, ME | Tested By: | crs |
| Project Location: | Frenchboro, ME | Checked By: | jsc |
| GTX #: | 308804 | | |
| Boring ID: | MB-FBORO-101 | | |
| Sample ID: | R4 | | |
| Depth: | 54.4-54.9 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 | 4.26 | 4.26 |
| Specimen Diameter, in: | 1.98 | 1.99 | 1.99 |
| Specimen Mass, g: | 559.72 | | |
| Bulk Density, lb/ft$^3$ | 161 | **Minimum Diameter Tolerence Met?** | YES |
| Length to Diameter Ratio: | 2.1 | **Length to Diameter Ratio Tolerance Met?** | YES |

**DEVIATION FROM STRAIGHTNESS (Procedure S1)**

Maximum gap between side of core and reference surface plate:
Is the maximum gap $\leq$ 0.02 in.? NO

*Maximum difference must be < 0.020 in.*

**Straightness Tolerance Met?** NO

**END FLATNESS AND PARALLELISM (Procedure FP1)**

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

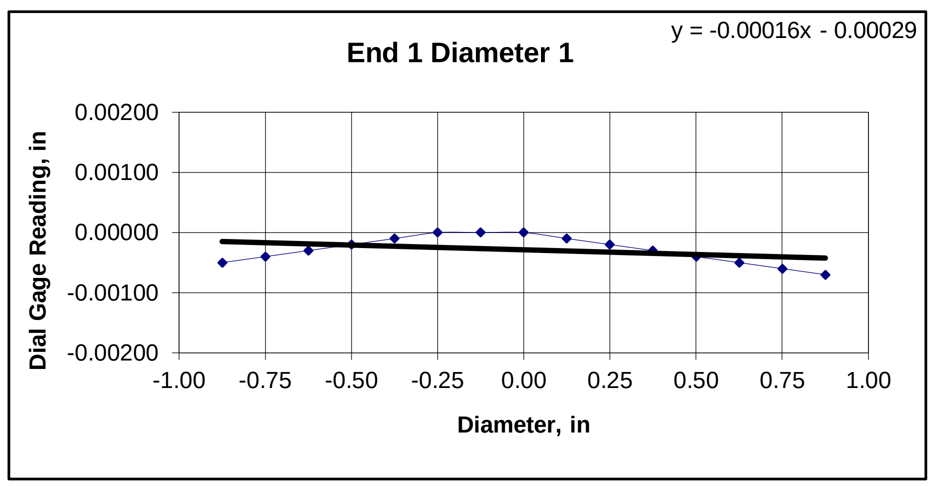
Difference between max and min readings, in:
0° = 0.00070 90° = 0.00090

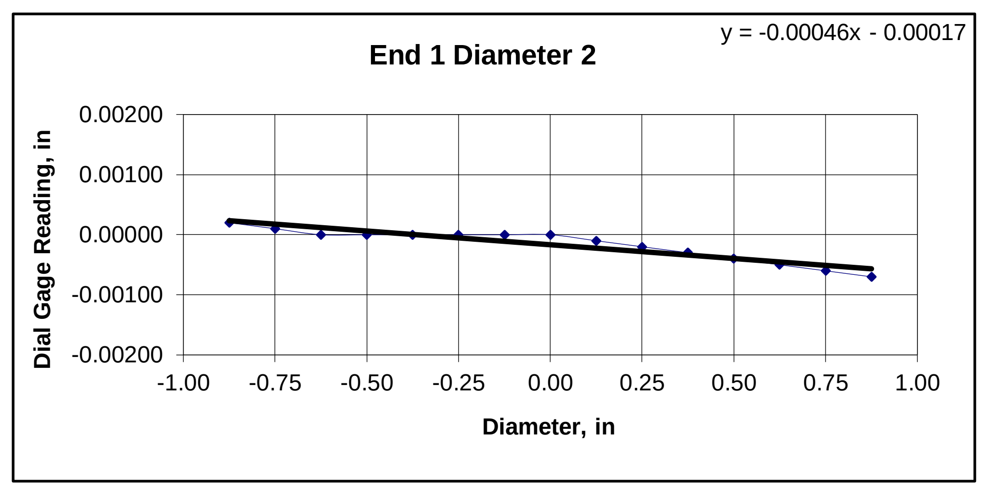
| END 2 | -0.875 | -0.750 | -0.625 | -0.500 | -0.375 | -0.250 | -0.125 | 0.000 | 0.125 | 0.250 | 0.375 | 0.500 | 0.625 | 0.750 | 0.875 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Diameter 1, in | -0.00050 | -0.00040 | -0.00030 | -0.00020 | -0.00010 | 0.00000 | 0.00000 | 0.00000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

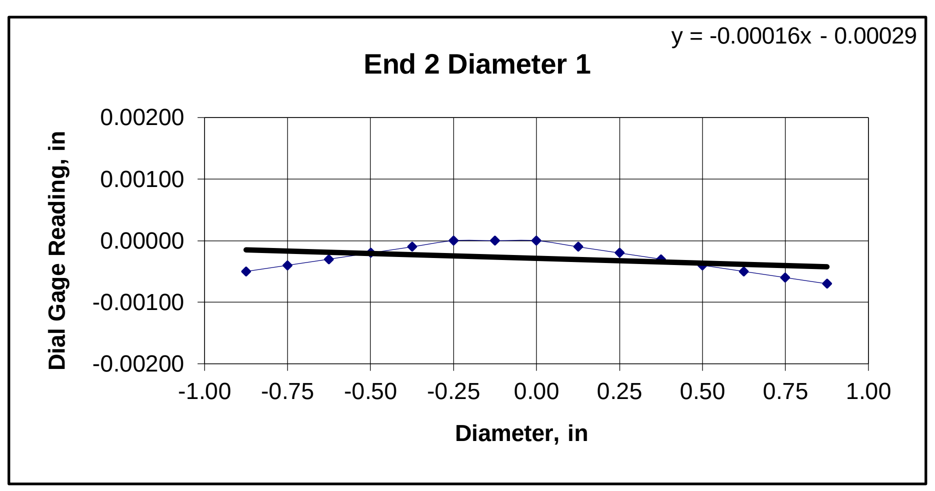
Difference between max and min readings, in:
0° = 0.0007 90° = 0.0009

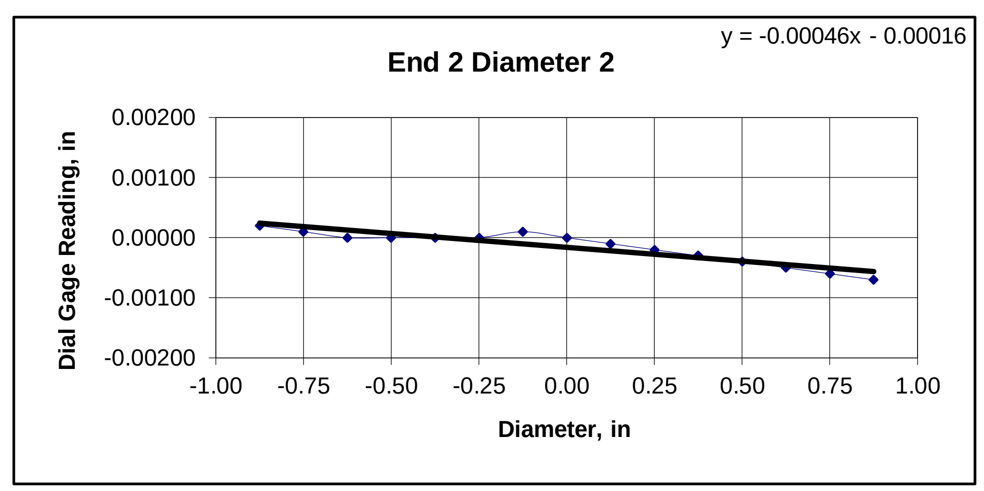
*Maximum difference must be < 0.0020 in.* Difference = $\pm$ 0.00045

**Flatness Tolerance Met?** YES




DIAMETER 1

End 1:

| | |
|---|---|
| Slope of Best Fit Line | 0.00016 |
| Angle of Best Fit Line: | 0.00900 |

End 2:

| | |
|---|---|
| Slope of Best Fit Line | 0.00016 |
| Angle of Best Fit Line: | 0.00900 |

Maximum Angular Difference: 0.00000

**Parallelism Tolerance Met?** YES
Spherically Seated

DIAMETER 2

End 1:

| | |
|---|---|
| Slope of Best Fit Line | 0.00046 |
| Angle of Best Fit Line: | 0.02619 |

End 2:

| | |
|---|---|
| Slope of Best Fit Line | 0.00046 |
| Angle of Best Fit Line: | 0.02636 |

Maximum Angular Difference: 0.00016

**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 | 1.985 | 0.00035 | 0.020 | YES |
| Diameter 2, in (rotated 90°) | 0.00090 | 1.985 | 0.00045 | 0.026 | YES |
| END 2 | | | | | |
| Diameter 1, in | 0.00070 | 1.985 | 0.00035 | 0.020 | YES |
| Diameter 2, in (rotated 90°) | 0.00090 | 1.985 | 0.00045 | 0.026 | YES |

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

**Perpendicularity Tolerance Met?** YES

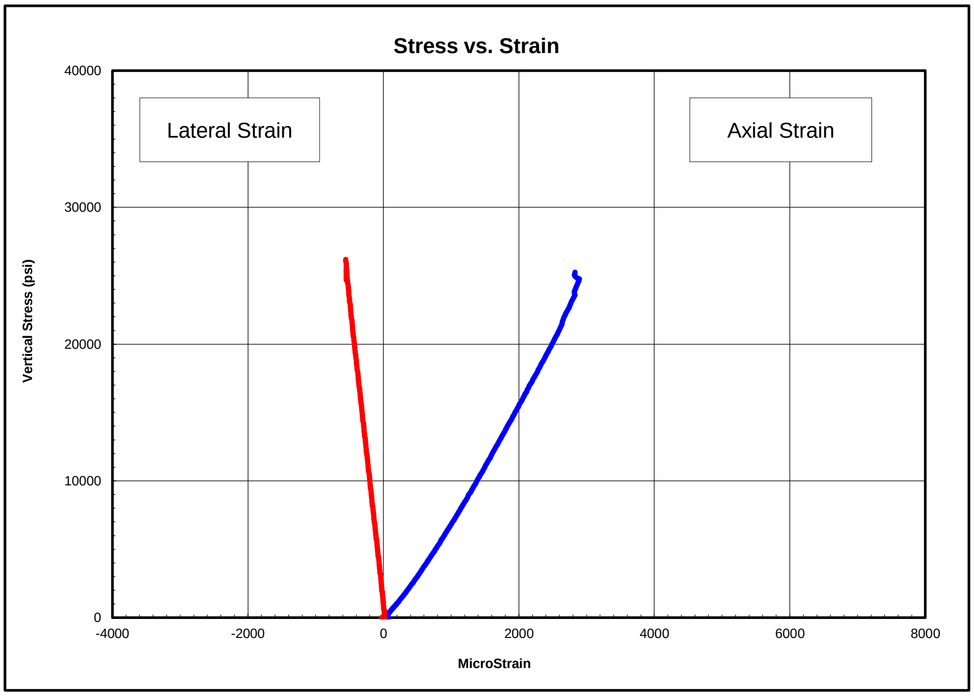
| | |
|---|---|
| Client: | Schonewald Engineering Associates, Inc. |
| Project Name: | MeDOT WIN 22202.00 MSFT Frenchboro, ME |
| Project Location: | Frenchboro, ME |
| GTX #: | 308804 |
| Test Date: | 9/21/2018 |
| Tested By: | tlm |
| Checked By: | jsc |
| Boring ID: | MB-FBORO-101 |
| Sample ID: | R4 |
| Depth, ft: | 54.4-54.9 |


After cutting and grinding


After break

| | |
|---|---|
| Client: | Schonewald Engineering Associates, Inc. |
| Project Name: | MeDOT WIN 22202.00 MSFT Frenchboro, ME |
| Project Location: | Frenchboro, ME |
| GTX #: | 308804 |
| Test Date: | 9/25/2018 |
| Tested By: | crs |
| Checked By: | jsc |
| Boring ID: | MB-FBORO-102 |
| Sample ID: | R1 |
| Depth, ft: | 37.3-37.7 |
| Sample Type: | rock core |
| Sample Description: | See photographs<br>Intact material failure |

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




Peak Compressive Stress: 26,194 psi

| Stress Range, psi | Young's Modulus, psi | Poisson's Ratio |
|---|---|---|
| 2600-9600 | 7,830,000 | 0.18 |
| 9600-16600 | 8,910,000 | 0.20 |
| 16600-23600 | 9,900,000 | 0.22 |

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: | Shonewald Engineering Associates, Inc. | | Test Date: | 9/20/2018 |
| Project Name: | MeDOT WIN 22202.00 MSFT Frenchboro, ME | | Tested By: | crs |
| Project Location: | Frenchboro, ME | | Checked By: | jsc |
| GTX #: | 308804 | | | |
| Boring ID: | MB-FBORO-102 | | | |
| Sample ID: | R1 | | | |
| Depth: | 37.3-37.7 | 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: | 4.31 | 4.31 | 4.31 | | |
| Specimen Diameter, in: | 1.98 | 1.98 | 1.98 | | |
| Specimen Mass, g: | 577.51 | | | | |
| Bulk Density, lb/ft$^3$ | 165 | | | **Minimum Diameter Tolerence Met?** | **YES** |
| Length to Diameter Ratio: | 2.2 | | | **Length to Diameter Ratio Tolerance Met?** | **YES** |

**DEVIATION FROM STRAIGHTNESS (Procedure S1)**

Maximum gap between side of core and reference surface plate:

Is the maximum gap $\leq$ 0.02 in.? YES

*Maximum difference must be < 0.020 in.*

**Straightness Tolerance Met?** **YES**

**END FLATNESS AND PARALLELISM (Procedure FP1)**

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

Difference between max and min readings, in:

0° = 0.00070 90° = 0.00130

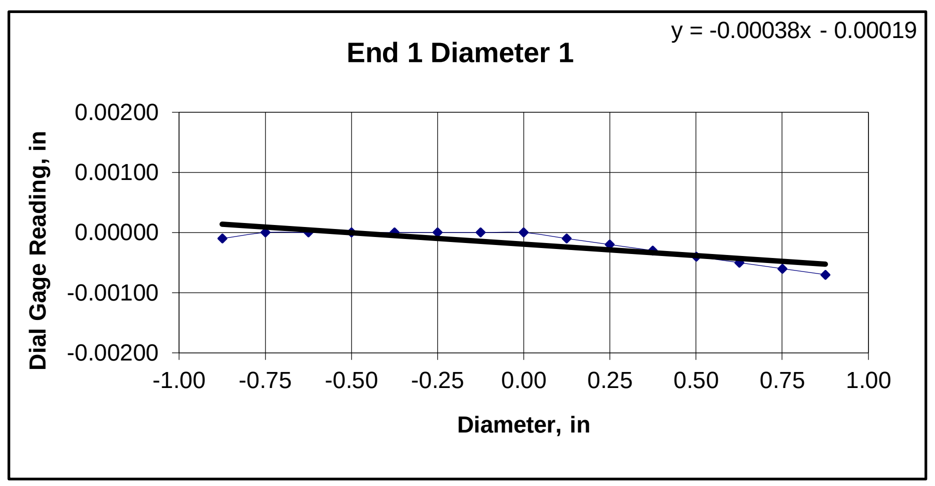
| END 2 | -0.875 | -0.750 | -0.625 | -0.500 | -0.375 | -0.250 | -0.125 | 0.000 | 0.125 | 0.250 | 0.375 | 0.500 | 0.625 | 0.750 | 0.875 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Diameter 1, in | -0.00010 | -0.00010 | -0.00010 | 0.00000 | 0.00000 | 0.00000 | 0.00000 | 0.00000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

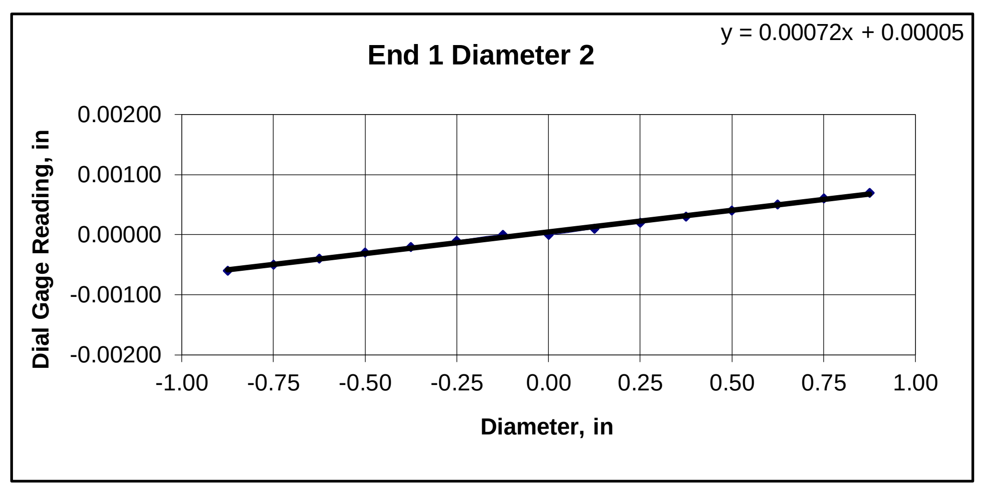
Difference between max and min readings, in:

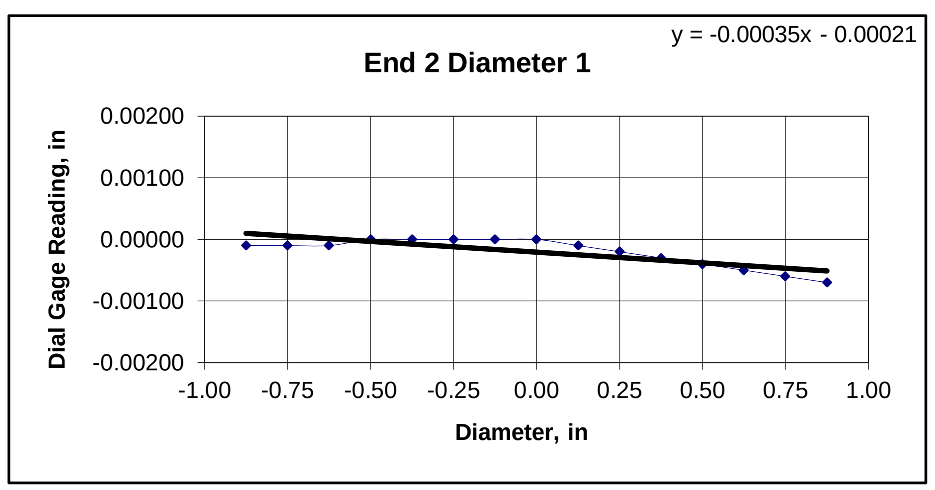
0° = 0.0007 90° = 0.0013

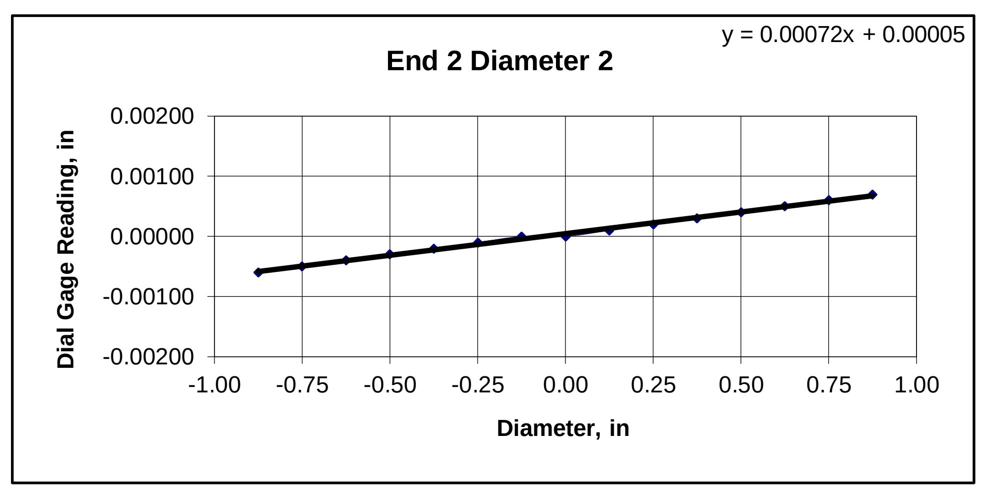
*Maximum difference must be < 0.0020 in.* Difference = $\pm$ 0.00065

**Flatness Tolerance Met?** **YES**




DIAMETER 1

End 1:

| | |
|---|---|
| Slope of Best Fit Line | 0.00038 |
| Angle of Best Fit Line: | 0.02177 |

End 2:

| | |
|---|---|
| Slope of Best Fit Line | 0.00035 |
| Angle of Best Fit Line: | 0.01997 |

Maximum Angular Difference: 0.00180

**Parallelism Tolerance Met?** **YES**

Spherically Seated

DIAMETER 2

End 1:

| | |
|---|---|
| Slope of Best Fit Line | 0.00072 |
| Angle of Best Fit Line: | 0.04125 |

End 2:

| | |
|---|---|
| Slope of Best Fit Line | 0.00072 |
| Angle of Best Fit Line: | 0.04125 |

Maximum Angular Difference: 0.00000

**Parallelism Tolerance Met?** **YES**

Spherically Seated

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

| END 1 | Difference, Maximum and Minimum (in.) | Diameter (in.) | Slope | Angle° | Perpendicularity Tolerance Met? |
|---|---|---|---|---|---|
| Diameter 1, in | 0.00070 | 1.980 | 0.00035 | 0.020 | YES |
| Diameter 2, in (rotated 90°) | 0.00130 | 1.980 | 0.00066 | 0.038 | YES |
| END 2 | | | | | |
| Diameter 1, in | 0.00070 | 1.980 | 0.00035 | 0.020 | YES |
| Diameter 2, in (rotated 90°) | 0.00130 | 1.980 | 0.00066 | 0.038 | YES |

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

**Perpendicularity Tolerance Met?** **YES**

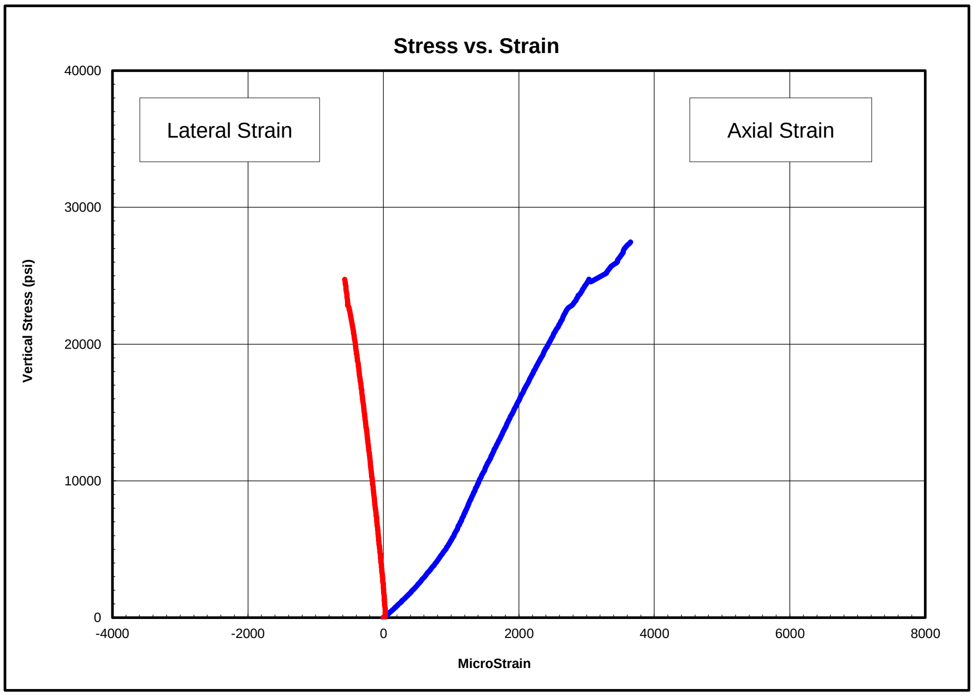
| | |
|---|---|
| Client: | Schonewald Engineering Associates, Inc. |
| Project Name: | MeDOT WIN 22202.00 MSFT Frenchboro, ME |
| Project Location: | Frenchboro, ME |
| GTX #: | 308804 |
| Test Date: | 9/21/2018 |
| Tested By: | tlm |
| Checked By: | jsc |
| Boring ID: | MB-FBORO-102 |
| Sample ID: | R1 |
| Depth, ft: | 37.3-37.7 |


After cutting and grinding


After break

| | |
|---|---|
| Client: | Schonewald Engineering Associates, Inc. |
| Project Name: | MeDOT WIN 22202.00 MSFT Frenchboro, ME |
| Project Location: | Frenchboro, ME |
| GTX #: | 308804 |
| Test Date: | 9/25/2018 |
| Tested By: | crs |
| Checked By: | jsc |
| Boring ID: | MB-FBORO-102 |
| Sample ID: | R2 |
| Depth, ft: | 41.3-42.0 |
| Sample Type: | rock core |
| Sample Description: | See photographs<br>Intact material failure |

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




Peak Compressive Stress: 27,467 psi

| Stress Range, psi | Young's Modulus, psi | Poisson's Ratio |
|---|---|---|
| 2700-10100 | 8,620,000 | 0.18 |
| 10100-17400 | 10,000,000 | 0.24 |
| 17400-24700 | 8,330,000 | 0.27 |

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: | Shonewald Engineering Associates, Inc. | Test Date: | 9/20/2018 |
| Project Name: | MeDOT WIN 22202.00 MSFT Frenchboro, ME | Tested By: | crs |
| Project Location: | Frenchboro, ME | Checked By: | jsc |
| GTX #: | 308804 | | |
| Boring ID: | MB-FBORO-102 | | |
| Sample ID: | R2 | | |
| Depth: | 41.3-42.0 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: | 4.09 | 4.09 | 4.09 |
| Specimen Diameter, in: | 1.98 | 1.98 | 1.98 |
| Specimen Mass, g: | 544.24 | | |
| Bulk Density, lb/ft$^3$ | 164 | **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 ≤ 0.02 in.? YES

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

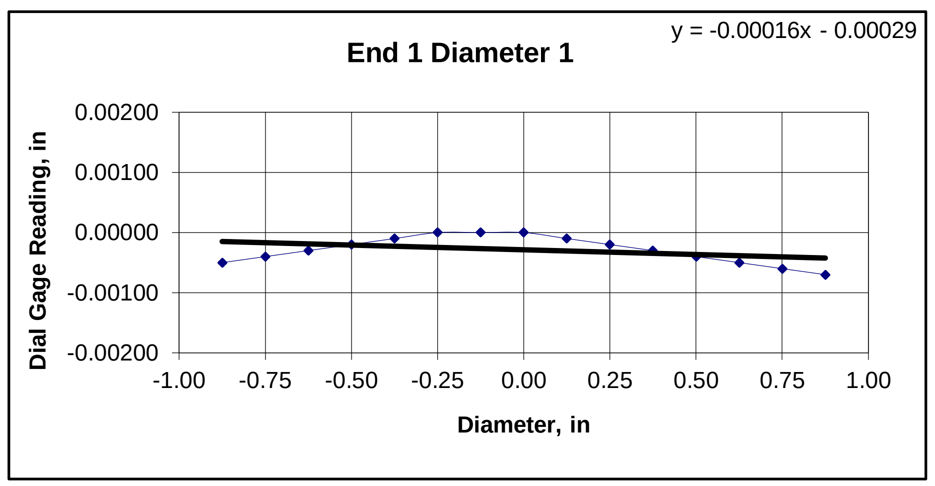
**END FLATNESS AND PARALLELISM (Procedure FP1)**

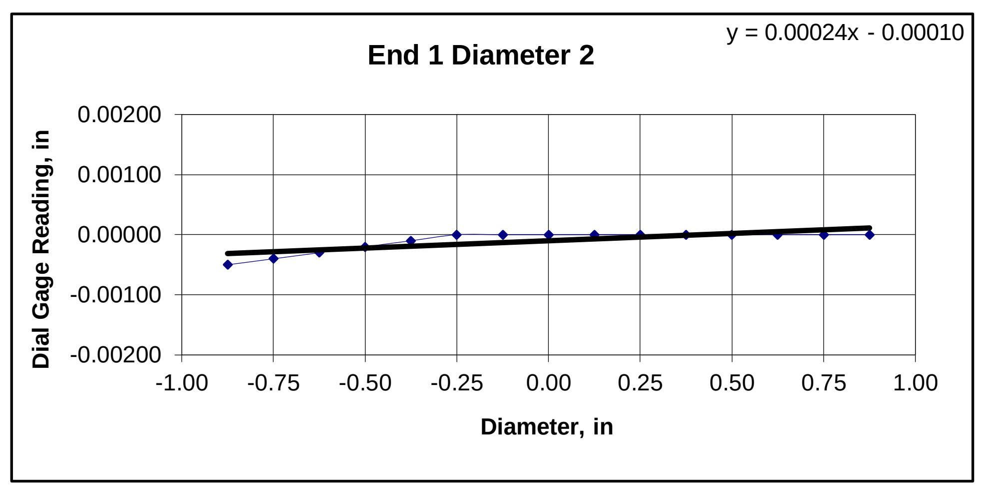
| END 1 | -0.875 | -0.750 | -0.625 | -0.500 | -0.375 | -0.250 | -0.125 | 0.000 | 0.125 | 0.250 | 0.375 | 0.500 | 0.625 | 0.750 | 0.875 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Diameter 1, in | -0.00050 | -0.00040 | -0.00030 | -0.00020 | -0.00010 | 0.00000 | 0.00000 | 0.00000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

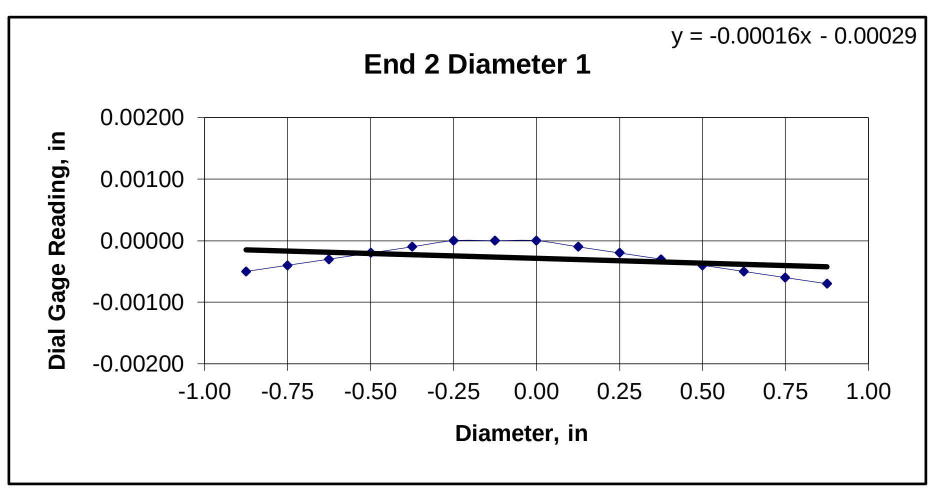
Difference between max and min readings, in:
0° = 0.00070 90° = 0.00050

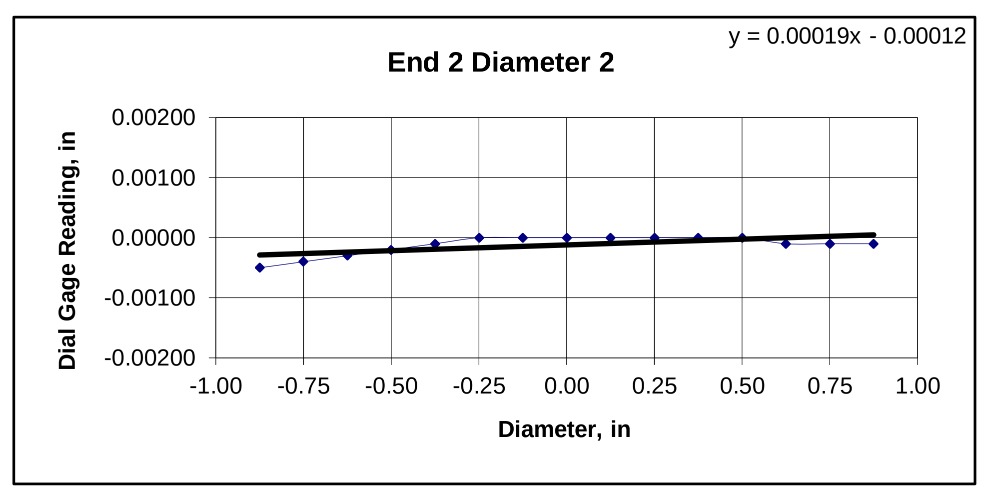
| END 2 | -0.875 | -0.750 | -0.625 | -0.500 | -0.375 | -0.250 | -0.125 | 0.000 | 0.125 | 0.250 | 0.375 | 0.500 | 0.625 | 0.750 | 0.875 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Diameter 1, in | -0.00050 | -0.00040 | -0.00030 | -0.00020 | -0.00010 | 0.00000 | 0.00000 | 0.00000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

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




DIAMETER 1

End 1:
Slope of Best Fit Line 0.00016
Angle of Best Fit Line: 0.00900

End 2:
Slope of Best Fit Line 0.00016
Angle of Best Fit Line: 0.00900

Maximum Angular Difference: 0.00000

**Parallelism Tolerance Met?** YES
Spherically Seated

DIAMETER 2

End 1:
Slope of Best Fit Line 0.00024
Angle of Best Fit Line: 0.01391

End 2:
Slope of Best Fit Line 0.00019
Angle of Best Fit Line: 0.01097

Maximum Angular Difference: 0.00295

**Parallelism Tolerance Met?** YES
Spherically Seated

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

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

*Maximum angle of departure must be ≤ 0.25°*

**Perpendicularity Tolerance Met?** YES

| | |
|---|---|
| Client: | Schonewald Engineering Associates, Inc. |
| Project Name: | MeDOT WIN 22202.00 MSFT Frenchboro, ME |
| Project Location: | Frenchboro, ME |
| GTX #: | 308804 |
| Test Date: | 9/21/2018 |
| Tested By: | tlm |
| Checked By: | jsc |
| Boring ID: | MB-FBORO-102 |
| Sample ID: | R2 |
| Depth, ft: | 41.3-42.0 |


After cutting and grinding


After break

SCHONEWALD ENGINEERING ASSOCIATES, INC.

# APPENDIX D

# CALCULATIONS

| | | | |
|---|---|---|---|
| **Project:** MaineDOT MSFT Fender System Modifications | **Proj. No.:** | 18-010 - 18-014 | |
| **Location:** Vinalhaven, Frenchboro, Bass Hbr, Swans Is, Islesboro | | | |
| **Subject:** Geotechnical Calculations | **Last updated:** | Feb. 2019 | **By** IVS |
| Rock Quality and Compressive Strength Summary | **Checked:** | May 2019 | **By** SJR |

**Objective**: Compile field and lab data to evaluate quality and strength of bedrock underlying the five Maine State Ferry Terminal (MSFT) sites.

**Data Sources**: Published bedrock geological information
Site-specific test boring logs, specifically rock core descriptions and RQDs.
Site-specific laboratory test results: unconfined compression tests on rock core samples.

**VINALHAVEN, FRENCHBORO, BASS HARBOR, AND SWANS ISLAND**

TEST BORINGS: Rock observed underlying the Vinalhaven and Frenchboro sites consists of GRANITE. : Rock observed underlying the Bass Harbor and Swans Island sites consists of BASALT. Both are very hard, fine to medium grained igneous crystalline rock and would be expected to behave the same under loading. RQDs and the results of laboratory uniaxial compressive strength tests are summarized on the test boring stick figures on Page 3 of 4.

LABORATORY TESTS: results of laboratory uniaxial compressive strength tests on the GRANITE and BASALT encountered in Vinalhaven, Frenchboro, Bass Hbr, and Swans Is are as follows:

| UCT $q_p$ (ksi) | | | | | |
|---|---|---|---|---|---|
| 17.3 | | | | | |
| ~~29.4~~ | | | | | |
| 15.9 | | | | | |
| 14.1 | average peak uniaxial compressive strength = | 16.4 | ksi = | 16,446 | psi |
| 16.4 | | | | | |
| 15.9 | throw out upper and lower results (strikethroughs), | | | | |
| 12.6 | average peak uniaxial compressive strength = | 15.6 | ksi = | 15,571 | psi |
| ~~7.9~~ | | | | | |
| ~~6.7~~ | minimum peak uniaxial compressive strength = | 6.7 | ksi = | 6,700 | psi |
| 16.8 | | | | | |
| ~~7.1~~ | maximum peak uniaxial compressive strength = | 29.4 | ksi = | 29,400 | psi |
| ~~26.2~~ | | | | | |
| ~~27.5~~ | | | | | |

**VINALHAVEN, FRENCHBORO, BASS HARBOR, AND SWANS ISLAND**

require pipe pile to penetrate 2 ft into bedrock prior to coring for rock anchors

bond stresses are concentrated at top of bonded length, therefore, require top of bonded length a minimum of 5 ft into bedrock where bedrock is GRANITE or BASALT based on rock core RQDs (see Page 3 of 4).

| | | | |
|---|---|---|---|
| **Project:** MaineDOT MSFT Fender System Modifications | **Proj. No.:** | 18-010 - 18-014 | |
| **Location:** Vinalhaven, Frenchboro, Bass Hbr, Swans Is, Islesboro | | | |
| **Subject:** Geotechnical Calculations | **Last updated:** | Feb. 2019 | **By** IVS |
| Rock Quality and Compressive Strength Summary | **Checked:** | May 2019 | **By** SJR |

**Objective**: Compile field and lab data to evaluate quality and strength of bedrock underlying the five Maine State Ferry Terminal (MSFT) sites.

**Data Sources**: Published bedrock geological information
Site-specific test boring logs, specifically rock core descriptions and RQDs.
Site-specific laboratory test results: unconfined compression tests on rock core samples.

**ISLESBORO**

<u>TEST BORINGS:</u> Rock observed underlying the Islesboro site consists of METAPELITE, which is an aphanitic to fine grained, low-grade metamorphosed mudstone. RQDs and the results of laboratory uniaxial compressive strength tests are summarized on the test boring stick figures on Page 4 of 4.

<u>LABORATORY TESTS:</u> results of laboratory uniaxial compressive strength tests on the METAPELITE encountered in Islesboro are as follows:

| UCT $q_p$ (ksi) |
|---|
| ~~11.1~~ |
| 8.2 |
| 8.5 |
| ~~1.0~~ |
| 6.9 |
| 8.5 |
| 7.0 |
| 7.6 |
| 6.1 |
| 6.1 |
| ~~1.4~~ |
| 5.2 |

average peak uniaxial compressive strength = 6.5 ksi = 6,467 psi

throw out upper and lower results (strikethroughs),
average peak uniaxial compressive strength = 7.1 ksi = 7,122 psi

minimum peak uniaxial compressive strength = 1.0 ksi = 1,000 psi

maximum peak uniaxial compressive strength = 11.1 ksi = 11,100 psi

**ISLESBORO**

require pipe pile to penetrate 2 ft into bedrock prior to coring for rock anchors

bond stresses are concentrated at top of bonded length, therefore, require top of bonded length a minimum of 15 ft into bedrock where bedrock is METAPELITE based on rock core RQDs (see Page 4 of 4)

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| | | | |
|---|---|---|---|
| Client | HNTB | File No. | 18-010 - 18-014 |
| Project | MEDOT MSFT FENDER SYSTEM MODIFICATIONS | Prepared | FEB 2019 By IVS |
| Location | VINALHAVEN, FRENCHBORO, BASS HBR, SWANS IS, | Checked | By |
| Subject | ROCK QUALITY AND COMPRESSIVE STRENGTH SUMMARY | Revised | By |

TOP OF BEDROCK — 5' — 10' — 15' — DEPTH BELOW TOP OF BEDROCK

VERT: 1" = 5'

**FRENCHBORO MB-FBORO-102 EL -47.4 FT**

| Run | RQD | $UCT_{gp}$ |
|---|---|---|
| R1 | 33% | 26.2 KSI |
| R2 | 62% | 27.5 KSI |
| R3 | 64% | |
| R4 | 93% | |
| R5 | 83% | |
| R6 | 87% | |

GRANITE

**FRENCHBORO MB-FBORO-101 EL -54.2 FT**

| Run | RQD | $UCT_{gp}$ |
|---|---|---|
| R1 | 63% | 16.8 KSI |
| R2 | 73% | |
| R3 | 80% | |
| R4 | 82% | 7.11 KSI |

GRANITE

**SWANS IS MB-SWAN-102 EL -40.1 FT**

| Run | RQD | $UCT_{gp}$ |
|---|---|---|
| R1 | 35% | |
| R2 | 50% | 12.6 KSI |
| R3 | 44% | 7.9 KSI, 6.7 KSI |
| R4 | 77% | |
| R5 | 62% | |
| R6 | 47% | |
| R7 | 97% | |

BASALT

**SWANS IS MB-SWAN-101 EL -20.2 FT**

| Run | RQD | $UCT_{gp}$ |
|---|---|---|
| R1 | 25% | 16.4 KSI |
| R2 | 60% | 15.9 KSI |
| R3 | 80% | |
| R4 | 75% | |
| R5 | 57% | |
| R6 | 26% | |

BASALT

**BASS HBR MB-BASS-101 EL -62.0 FT**

| Run | RQD | $UCT_{gp}$ |
|---|---|---|
| R1 | 0% | |
| R2 | 58% | ~~$UCT_{gp}$~~ |
| R3 | 78% | 15.9 KSI, 14.1 KSI |
| R4 | 22% | |
| R5 | 26% | |
| R6 | 33% | |
| R7 | 53% | |
| R8 | 53% | |

BASALT

**VINALHAVEN MB-VNHN-101 EL -36.9 FT**

| Run | RQD | $UCT_{gp}$ |
|---|---|---|
| R1 | 83% | |
| R2 | 82% | 17.3 KSI, 29.4 KSI |
| R3 | 93% | |
| R4 | 60% | |
| R5 | 60% | |
| R6 | 90% | |

GRANITE (GABBRO)

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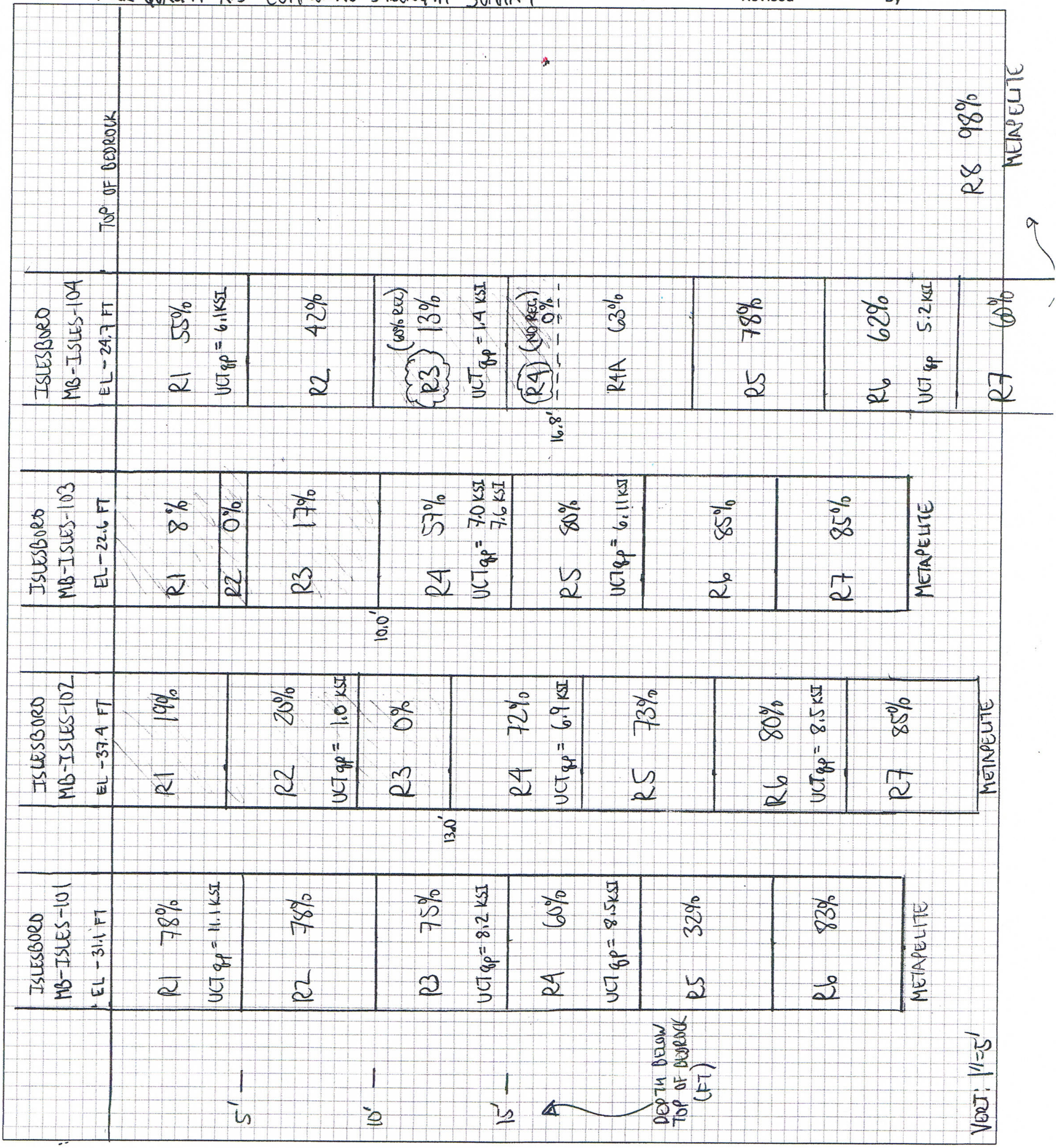
www.SchonewaldEngineering.com



| | | | |
|---|---|---|---|
| Client | HNTB | File No. | 18-010 – 18-014 |
| Project | MeDOT MSFT FENDER SYSTEM MODIFICATIONS | Prepared | FEB 2019 By IUS |
| Location | ISLESBORO | Checked | By |
| Subject | ROCK QUALITY AND COMPRESSIVE STRENGTH SUMMARY | Revised | By |



| | | | |
|---|---|---|---|
| **Project:** MaineDOT MSFT Fender System Modifications | | **Proj. No.** | 18-010 - 18-014 |
| **Location:** Vinalhaven, Frenchboro, Bass Hbr, Swans Is, Islesboro | | | |
| **Subject:** | Geotechnical Calculations | **Last updated:** | May 2019 **By** IVS |
| | Geotechnical axial capacity of pipe piles | **Checked:** | May 2019 **By** SJR |

**Objective**:
Check geotechnical axial compressive capacity of pipe piles for turning dolphins and mooring dolphin.

**References**:
2014 LRFD Manual - Article 10.5.5 "Resistance Factors"
Article 10.7.3.2.3 "Point Bearing Piles on Rock - Piles Driven to Hard Rock"
Duncan C. Wyllie's "Foundations on Rock" - Section 8.3.2 "Rock Socketed Piers - End-Bearing Capacity"

**Data Sources**:
Refer to calculations entitled "Rock Quality and Compressive Strength Summary"

**Discussion:**
Per the LRFD Manual Article 10.7.3.2.3 - The nominal resistance of piles driven to point bearing on hard rock where pile penetration is minimal is controlled by the structural limit state. Per the LRFD Manual Article 10.5.5 - The resistance factor for driven piles, drilled shafts, and micropiles in axial compression bearing on rock is 0.50.

Per Wyllie - The allowable end-bearing capacity of rock socketed piles, which includes a Factor of Safety of 2 to 3, is a function of the uniaxial compressive strength of the rock at the base of the pile. Working through Wyllie's analysis and assuming a Factor of Safety of 2, the ultimate end-bearing capacity can be determined and can be considered roughly equivalent to the nominal resistance of the pile.

**Analysis:**

Qult = Qavail * FS

Qult = (UCT qp)*(π(B/2)^2) from Wyllie Eq 8.9 and conservatively assuming a FS=2

where: Qult = ultimate axial capacity in end-bearing
UCT qp= uniaxial compressive strength of intact rock
B = pile diameter

- capacity of **turning dolphins (TDs)** proposed at Vinalhaven, Frenchboro, Bass Hbr & Swans Is, all of which are underlain by GRANITE or BASALT with a UCT qp = 15.6 ksi (refer to rock quality and compressive strength calcs)

Qult = (UCT qp)*(π(B/2)^2)
Qult = (15.6 ksi)*[(π)(8 in)^2]
Qult = 3,136 kips
Qavail = Qult/FS = Qult*$\varphi_{stat}$ where: FS = factor of safety
$\varphi_{stat}$ = resistance factor (tip resistance on rock)

3,136 kips * 0.5
1,568 kips

per HNTB TD design: Qreq'd = 345 kips Qavail = 1,568 kips >> Qreq'd = 345 kips **OKAY**

- capacity of **mooring dolphin (MDs)** proposed at Islesboro, that is underlain by METAPELITE with a UCT qp = 7.1 ksi (refer to rock quality and compressive strength calcs)

Qult = (UCT qp)*(π(B/2)^2)
Qult = (7.1 ksi)*[(π)(8 in)^2]
Qult = 1,427 kips
Qavail = Qult/FS = Qult*$\varphi_{stat}$
1,427 kips * 0.5
714 kips

per HNTB MD design: Qreq'd = 163 kips Qavail = 714 kips >> Qreq'd = 163 kips **OKAY**

| | | | | |
|---|---|---|---|---|
| **Project:** MaineDOT MSFT Fender System Modifications | | **Proj. No.** | 18-010 - 18-014 | |
| **Location:** Vinalhaven, Frenchboro, Bass Hbr, Swans Is, Islesboro | | | | |
| **Subject:** | Geotechnical Calculations | **Last updated:** | Mar 2019 | **By** IVS |
| | Rock anchor capacity | **Checked:** | May 2019 | **By** SJR |

**Objective**:
Check geotechnical capacity of rock anchors proposed for turning dolphins and mooring dolphin.
**References**:
2014 LRFD Manual - Article 11.5.7 "Resistance Factors" (pullout resistance of anchors utilized for permanent anchored retaining walls) and Article 11.9.4.2 "Anchor Pullout Capacity"
Duncan C. Wyllie "Foundations on Rock" - Section 9.3.2 "Tension Foundations - Allowable bond stresses and anchor design"
Post-Tensioning Institute "Recommendations for Prestressed Rock and Soil Anchors" - Article 6.0 "Design"
**Data Sources**:
Refer to calculations entitled "Rock Quality and Compressive Strength Summary"
**Discussion:**
The tensile capacity of rock anchors was determined using established ASD methods (Wyllie and PTI) in the absence of definitive guidance in the LRFD Manual, Article 11.9.4.2 - "Anchor Pullout Capacity." A resistance factor equal to 1.0 is allowed for per the LRFD Manual since all the rock anchors will be tested.
Per Wyllie - The design against failure of the anchor at the grout interfaces requires that the length of the bond zone, and the diameter of the threadbar and drill hole are proportioned such that the average bond stress is less than the working bond strength. Selecting the location (depth into rock) of the top of the bonded length must recognize that stresses are concentrated near the top of the bonded length.
**Analysis:**

Qavail = Qult / FS

Qavail = (Lb)(π)(d)(τa) from Wyllie Eq 9.8, which includes a FS=3 through the τa term

where: Qavail = available tensile capacity of rock anchor
Lb = bonded length = 15 ft per design
d = drill hole diameter = 4.5 in = 0.375 ft per design
τa = working bond strength of rock-grout interface
τa ≈ (UCT qp)/30 up to 1.4 Mpa or 203 psi (Wyllie, Eq 9.9)

❖ capacity of **turning dolphins (TDs)** proposed at Vinalhaven, Frenchboro, Bass Hbr & Swans Is, all of which are underlain by GRANITE or BASALT with a UCT qp = 15.6 ksi (refer to rock quality and compressive strength calcs)

τa ≈ (UCT qp)/30 = (15,600 psi/30) = 520 psi 520 psi > 203 psi, use τa = 203 psi
Qavail = (Lb)(π)(d)(τa)
Qavail = (15 ft)*(π)(0.375 ft)[(203 lb/in$^2$)(144 in$^2$/ft$^2$)]
Qavail = 516,557 lbs
Qavail = 517 kips

per HNTB TD design: Qreq'd = 112 kips Qavail = 517 kips >> Qreq'd = 112 kips **OKAY**

❖ capacity of **mooring dolphin (MDs)** proposed at Islesboro, that is underlain by METAPELITE with a UCT qp = 7.1 ksi (refer to rock quality and compressive strength calcs)

τa ≈ (UCT qp)/30 = (7,100 psi/30) = 237 psi 237 psi > 203 psi, use τa = 203 psi
Qavail = (Lb)(π)(d)(τa)
Qavail = (15 ft)*(π)(0.375 ft)[(203 lb/in$^2$)(144 in$^2$/ft$^2$)]
Qavail = 516,557 lbs
Qavail = 517 kips

per HNTB MD design: Qreq'd = 49 kips Qavail = 517 kips >> Qreq'd = 49 kips **OKAY**

| | | | |
|---|---|---|---|
| **Project:** MaineDOT MSFT Fender System Modifications | | **Proj. No.** 18-010 - 18-014 | |
| **Location:** Vinalhaven, Frenchboro, Bass Hbr, Swans Is, Islesboro | | | |
| **Subject:** | Geotechnical Calculations | **Last updated:** Apr 2019 | **By** IVS |
| | Geotechnical uplift capacity of wave screen timber piles | **Checked:** May 2019 | **By** SJR |

**Objective**:
Estimate geotechnical uplift capacity of timber piles for wave screens using static methods.

**References**:
2014 LRFD Manual - Article 10.5.5 "Resistance Factors"
Article 10.7.3.10 "Uplift Resistance of Single Piles"

**Data Sources**:
Overburden conditions encountered in recent subsurface explorations (refer to summary on Page 4) and historical subsurface data from Vinalhaven and Swans Island (refer to Pages 5 and 6);
Proposed wave screen locations (refer to Pages 5 and 6).

**Discussion:**
Per the LRFD Manual Article 10.7.3.2.3 - The nominal uplift resistance of a driven pile, Rs, is calculated using an appropriate method of determining side resistance of a single pile. This analysis uses the static Norlund method for calculating nominal side resistance in cohesionless soils and the static alpha method for calculating nominal side resistance in cohesive soils.
Per the LRFD Manual Table 10.5.5.2.3-1 - The resistance factor for the uplift resistance of a single driven pile after the Norlund method is $\varphi_{up}$ = 0.35 and after the alpha method is $\varphi_{up}$ = 0.25.

$R_R = R_s\varphi_{up}$ where: $R_R$ = factored uplift resistance
$\varphi_{up}$ = uplift resistance factor (see above discussion)
$R_S$ = nominal side resistance

Rs = qs*As where: qs = unit side resistance of pile
As = surface area of pile = circumference*L = C*L
C (timber pile) = 2πr = 2π(9 in/2) = 2.36 ft

**Analysis - Norlund method for cohesionless soils:**

qs = Kδ Cf φ'v [(sin(δ+ω)) / cos ω)

from: LRFD Manual Eq 10.7.3.8.6f-1 with φf taken as 30 deg.
where: Kδ = coef. of lateral earth pressure at mid-point of layer under consideration; Fig 10.7.3.8.6f-2
Cf = correction factor for Kδ; Fig 10.7.3.8.6f-5
φ'v = effective overburden pressure at midpoint of layer
δ = friction angle between pile and soil; Fig 10.7.3.8.6f-6
ω = angle of pile taper from vertical = 0.47 deg
(assume taper from 12 in to 8 in over 40 ft)

function of soil: φf taken as 30 deg.
function of pile: Kδ = 2.55 from Fig 10.7.3.8.6f-2 with ω = 0.47 deg
δ = 16.5 from Fig 10.7.3.8.6f-6 for timber pile and Vol = 0.55 cf/ft
Cf = 0.8 from Fig 10.7.3.8.6f-5 with φf = 30 deg and δ = 16.5 deg

**Analysis - alpha method for cohesive soils:**

qs = (Su)(α) where: Su = average undrained shear strength of soil taken as 400 psf based on field observation and experience
α = function of Su; taken from LRFD Fig 10.7.3.8.6b-1 with D = 9 in = 0.75 ft and Db from soil profile where Db is the length of the pile providing resistance

**ANALYSIS (cont'd)**

**resistance is a function of soil profile:**

develop reasonable suburface profile for modeling the proposed wave screens

refer to summary of overburden conditions (Pg 4) and proposed wave screen locations (Pgs 5 and 6)

| | |
|---|---|
| ***Islesboro:*** | cohesionless; base on **MB-ISLES-103;** ISLES-101 and ISLES-102 similar |
| ***Swans Island:*** | cohesionless; westerly end base on **MB-ISLES-103**, that is similar to SWAN-101 |
| | cohesionless; easterly end base on **MB-ISLES-104**, that is similar to historic data |
| ***Vinalhaven:*** | cohesive; base on **MB-VNHN-101**, that is similar to historic Vinalhaven data |
| ***Bass Harbor:*** | cohesive; base on **MB-SWAN-102;** conservatively low, but limited data available |
| ***Frenchboro:*** | no wave screens proposed at Frenchboro |

**EVALUATE FOR LOCATIONS WITH COHESIONLESS SOILS**

❖ uplift capacity of single timber pile based on **MB-ISLES-103**

disregard resistance from "muck;" resistance only from soil layer between 2 and 6 ft BGS

φ'v = (115 pcf - 62.4 pcf)(4 ft) = 210 psf (at midpoint of layer)

qs = Kδ Cf φ'v [(sin(δ+ω)) / cos ω)

qs = (2.55)(0.8)(210)[(sin(16.5+0.47)/cos(0.47)]

qs = 125 psf

Rs = qs*As

Rs = 125 psf * 4 ft * 2.36 ft

Rs = 1,180 lbs

Rs = 1.2 kips

$R_R = R_s\varphi_{up}$

$R_R$ = 1.2 kips * 0.35

$R_R$ = 0.4 kips **CONCLUSION:** uplift capacity negligible; (structural) design of wave screen must limit uplift (tensile) load

**NOTE applies to prop. Islesboro wave screens & westerly end of Swans Is. wave screen**

❖ uplift capacity of single timber pile based on **MB-ISLES-104**

disregard resistance from "muck;" resistance only from soil layer between 2 and 13 ft BGS

φ'v = (115 pcf - 62.4 pcf)(7.5 ft) = 395 psf (at midpoint of layer)

qs = Kδ Cf φ'v [(sin(δ+ω)) / cos ω)

qs = (2.55)(0.8)(395)[(sin(16.5+0.47)/cos(0.47)]

qs = 235 psf

Rs = qs*As

Rs = 235 psf * 11 ft * 2.36 ft

Rs = 6,101 lbs

Rs = 6.1 kips

$R_R = R_s\varphi_{up}$

$R_R$ = 6.1 kips * 0.35

$R_R$ = 2.1 kips **CONCLUSION:** uplift capacity limited; (structural) design of wave screen must limit uplift (tensile) load

**NOTE applies to the easterly end of Swans Is. wave screen**

**ANALYSIS (cont'd)**

**EVALUATE FOR LOCATIONS WITH COHESIVE SOILS**

❖ uplift capacity of single timber pile based on **MB-VNHN-101**

disregard resistance from "muck;" resistance only from soil layer between 5 and 15 ft BGS

Silty Clay with a Su = 400 psf

qs = (Su)(α) with α taken from LRFD Fig 10.7.3.8.6b-1 with D = 0.75 ft and Db = 10 ft

and Db/D = 13.3

α = 0.6 (from center fig)

qs = (400)(0.6)

qs = 240 psf

Rs = qs*As

Rs = 240 psf * 10 ft * 2.36 ft

Rs = 5,664 lbs

Rs = 5.7 kips

$R_R = R_s\varphi_{up}$

$R_R$ = 5.7 kips * 0.25

$R_R$ = 1.4 kips **CONCLUSION:** uplift capacity limited; (structural) design of wave screen should limit uplift (tensile) load

**NOTE applies to the prop. Vinalhaven wave screens**

❖ uplift capacity of single timber pile based on **MB-SWAN-102**

disregard resistance from "muck;" resistance only from soil layer between 4 and 25.5 ft BGS

Silty Clay with a Su = 400 psf

qs = (Su)(α) with α taken from LRFD Fig 10.7.3.8.6b-1 with D = 0.75 ft and Db = 21.5 ft

and Db/D = 28.7

α = 0.95 (from center fig)

qs = (400)(0.95)

qs = 380 psf

Rs = qs*As

Rs = 380 psf * 21.5 ft * 2.36 ft

Rs = 19,281 lbs

Rs = 19.2 kips

$R_R = R_s\varphi_{up}$

$R_R$ = 19.2 kips * 0.25

$R_R$ = 4.8 kips **CONCLUSION:** modest uplift capacity; (structural) design of wave screen should limit uplift (tensile) load

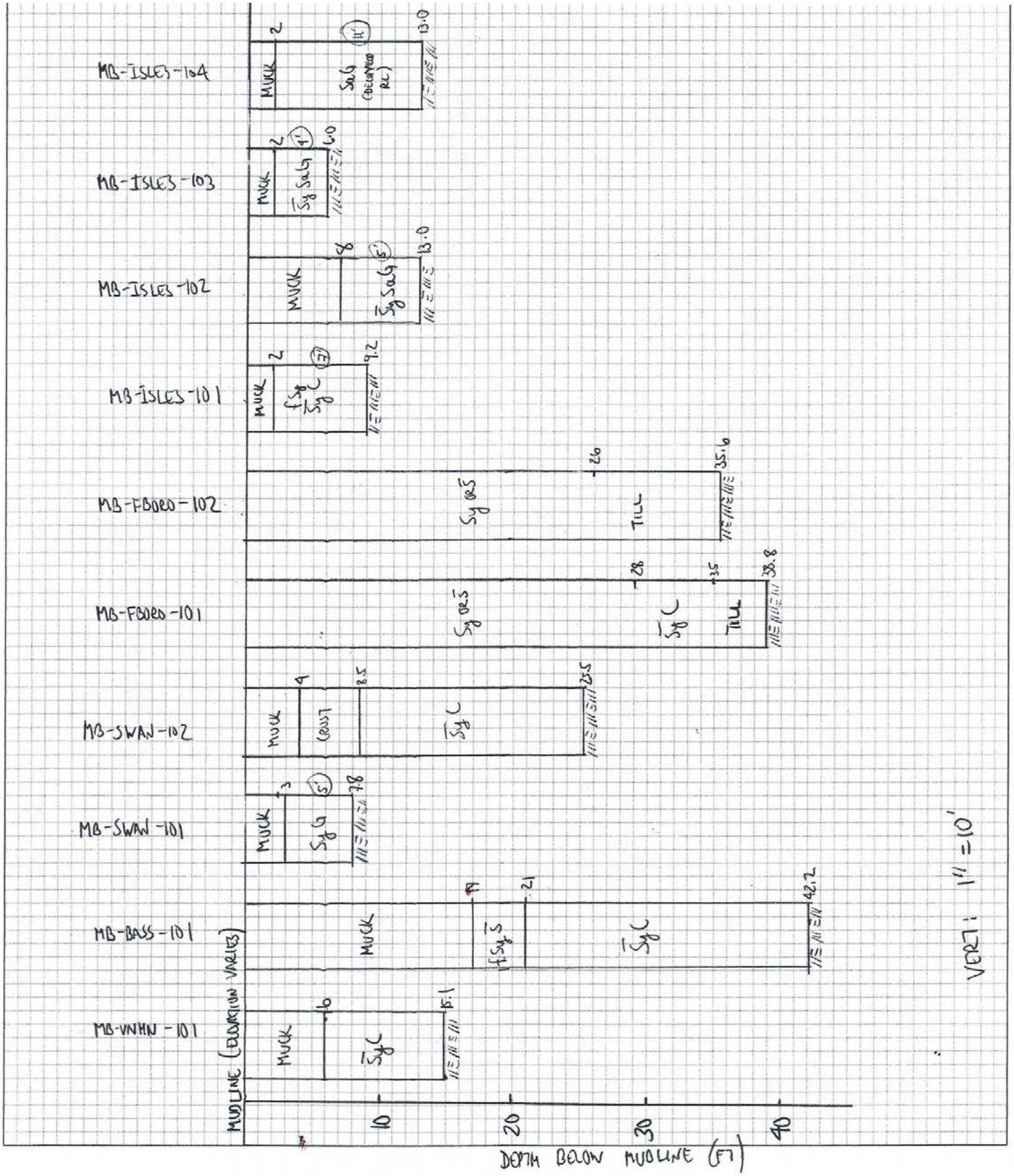
**NOTE applies to the prop. Bass Harbor wave screen; could be conservatively low, but have limited subsurface data**

**Project:** MaineDOT MSFT Fender System Modifications **Proj. No.** 18-010 - 18-014

**Location:** Vinalhaven, Frenchboro, Bass Hbr, Swans Is, Islesboro

**Subject:** Geotechnical uplift capacity of wave screen timber piles **Last updated:** Apr 2019 **By** IVS

Summary of subsurface conditions - recent test borings



SCHONEWALD ENGINEERING ASSOCIATES, INC.



**Project:** MaineDOT MSFT Fender System Modifications **Proj. No.** 18-010 - 18-014

**Location:** Vinalhaven, Frenchboro, Bass Hbr, Swans Is, Islesboro

**Subject:** Geotechnical uplift capacity of wave screen timber piles **Last updated:** Apr 2019 **By** IVS

Summary of wave screen and recent/historic boring locations and historic subsurface data

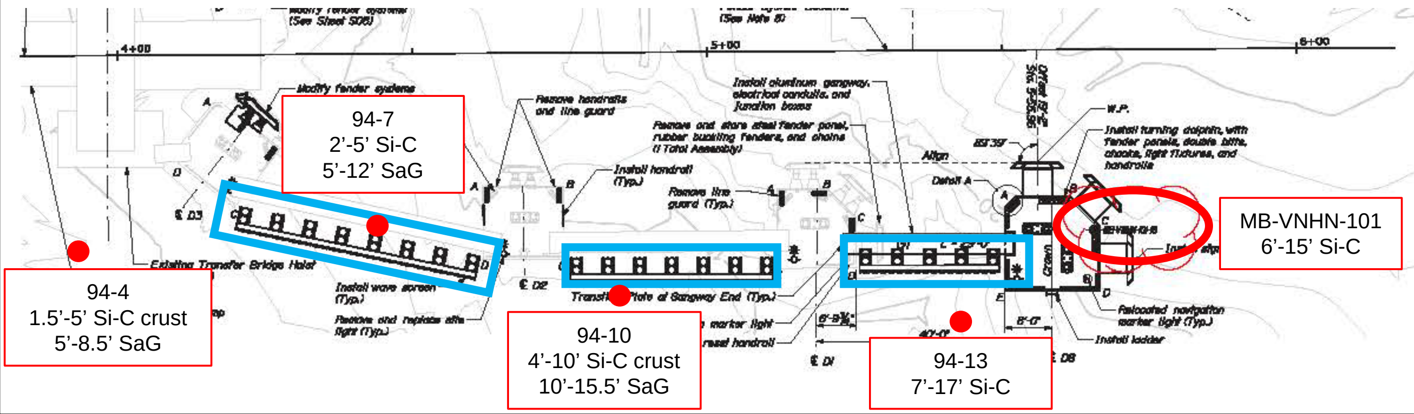
**Legend** (Pg 5 & 6):

Prop. Wave Screen

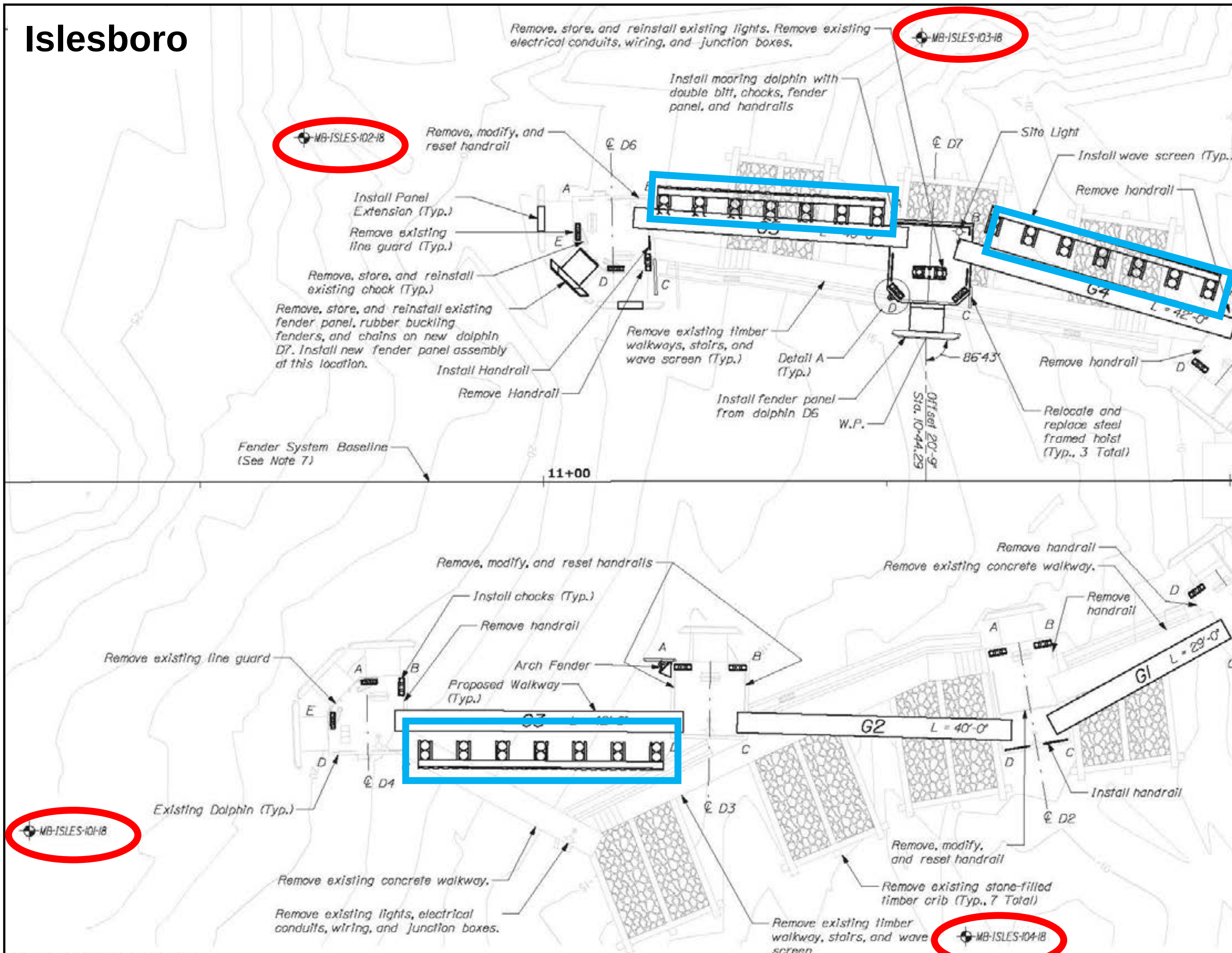
Recent Boring

94-13
7'-17' Si-C
Approx. Historic Boring w/ Summary of Conditions

## Vinalhaven




## Islesboro



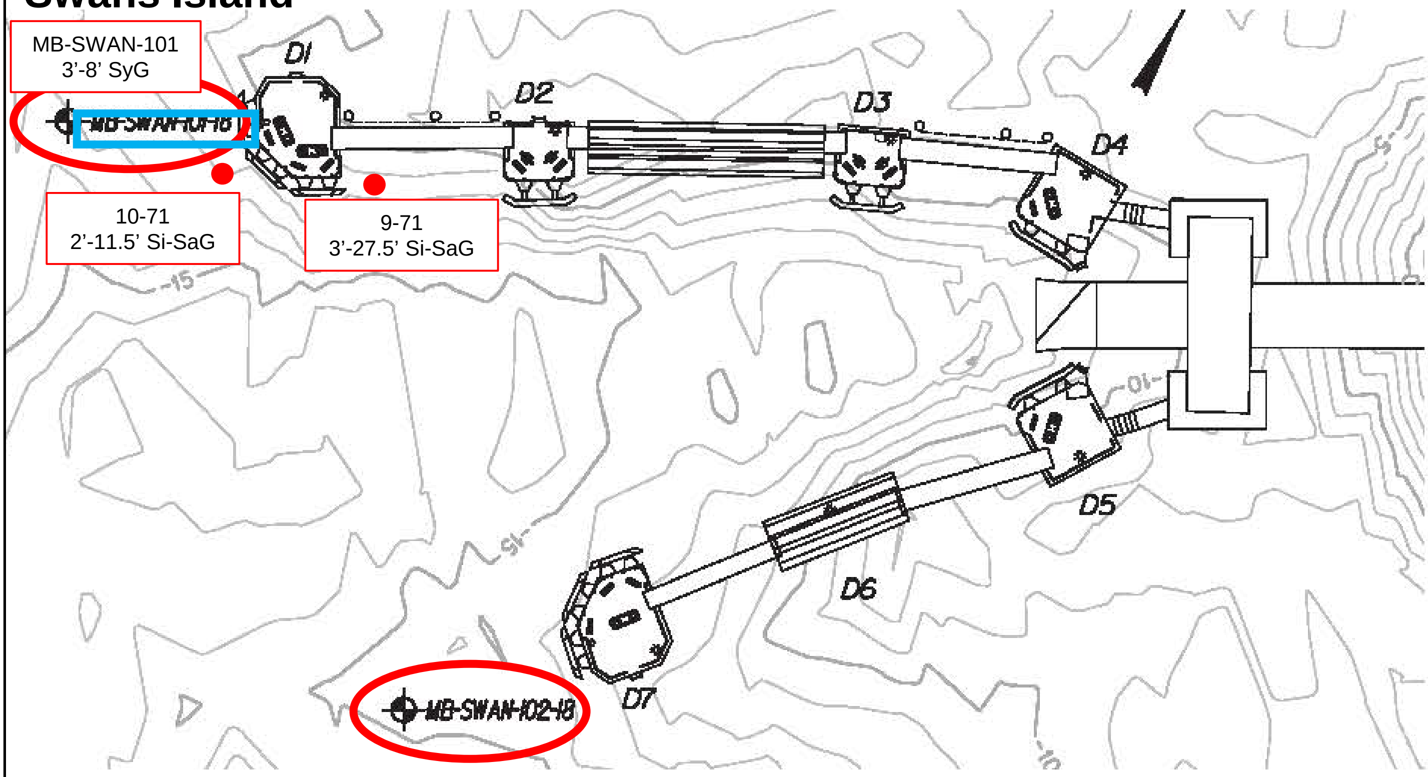
**Project:** MaineDOT MSFT Fender System Modifications **Proj. No.** 18-010 - 18-014

**Location:** Vinalhaven, Frenchboro, Bass Hbr, Swans Is, Islesboro

**Subject:** Geotechnical uplift capacity of wave screen timber piles **Last updated:** Apr 2019 **By** IVS

Summary of wave screen and recent/historic boring locations and historic subsurface data

## Swans Island




## Bass Harbor