SCHONEWALD ENGINEERING ASSOCIATES, INC.

GEOTECHNICAL DESIGN REPORT
FENDER SYSTEM MODIFICATIONS
MAINE STATE FERRY TERMINAL – BASS HARBOR, MAINE
MAINEDOT WIN 23476.00

PREPARED FOR:

HNTB Corporation
Westbrook, Maine

PREPARED BY:

Isabel V. (Be) Schonewald, P.E.
Schonewald Engineering Associates, Inc. (SchonewaldEA)
129 Middle Road
Cumberland, Maine 04021
Be@SchonewaldEngineering.com

STATE OF MAINE
ISABEL V. SCHONEWALD
No. 7528
LICENSED
PROFESSIONAL ENGINEER

6/14/19

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SchonewaldEA Project No. 18-013

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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 Bass Harbor, Maine (MaineDOT WIN 23476.00).

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 Bass Harbor, Maine is depicted on attached Figure 1 – Locus Plan.

The fender system modifications at the Maine State Ferry Terminal in Bass Harbor 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 southerly arm of 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.

Associated with this work is the installation of one timber wave screen between the existing outboard (turning) dolphin and the proposed dolphin that will form the southerly side of the pen. The wave screen will be supported by an A-frame of timber piles that will be driven to rock.

Appurtenant work includes the installation of aluminum gangways, removal and storage of fender systems, and modifications to the electrical system.

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 Silurian to lower Devonian mafic to felsic volcanic rocks of the Castine Formation (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 Bass Harbor, Swans Island, Vinalhaven, Frenchboro, and Islesboro. As part of the overall program, one test boring was drilled at the Bass Harbor terminal in the vicinity of the proposed turning dolphin to evaluate subsurface conditions. The test boring was designated MB-BASS-101. The test boring was drilled on August 8 and 9, 2018 and was observed and logged by SchonewaldEA. The approximate location of the test boring is depicted on Figure 2 – Boring Location Plan that is included with the Figures.

The test boring was drilled using a barge-mounted drill rig and was completed using standard cased wash boring techniques. The boring was advanced through overburden to refusal and 30.3 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 log and was used to obtain the N_{60} values that are also reported on the log.

The log of the subsurface exploration is included as Appendix A. Photographs of the rock core collected from the test boring are included as Appendix B.

LABORATORY TESTING

Representative specimens of the rock core obtained in the test boring 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-BASS-101	R2	46.2-46.7 ft. BGS	bedrock; uniaxial compressive strength test (specimen chipped during sample preparation and was not viable for testing)
MB-BASS-101	R3	50.8-51.2 ft. BGS	bedrock; uniaxial compressive strength test
MB-BASS-101	R3	52.3-52.8 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 log included in Appendix A and the laboratory test reports are included as Appendix C.

SUBSURFACE CONDITIONS

Subsurface conditions encountered in the Bass Harbor test boring consisted of approximately 42 feet of overburden soils overlying basalt bedrock. Specifically, the overburden soils encountered in the test boring consisted of approximately 19 feet of very loose, organic silt "muck" (recent sediment) over an approximately 2-foot thickness of loose, fine sandy silt (alluvium) that was underlain by an approximately 21-foot thickness of very soft to soft marine silty clay. Bedrock underlying the overburden soils in the test boring consisted of basalt. Detailed descriptions of the overburden encountered in the test boring are provided on the logs included in Appendix A.

Bedrock was encountered at approximate elevation -62.0 feet in MB-BASS-101. The bedrock core obtained from the test boring consisted of very hard, typically fresh, aphanitic to fine-grained, dark grey basalt with quartzite veins; evidence of healed fractures was observed in most of the core runs. Basalt, a mafic volcanic rock, is mapped in the area and is considered a very hard, crystalline igneous rock.

The Rock Quality Designations (RQDs) of the rock cores obtained in the test boring ranged from 0 to 78 percent. Three specimens of the rock core were submitted for uniaxial compressive strength tests. One specimen chipped during sample preparation and was not viable for testing. The test results for the remaining two specimens reported the uniaxial compressive strength of the basalt ranged between 14.1 and 15.9 ksi.

Detailed descriptions of the rock encountered in the Bass Harbor test boring are provided on the log 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 similar very hard, crystalline igneous rock.

Test boring programs were completed for past work at the Bass Harbor terminal, including in 1971 and 2000. Two of the 1971 test borings were drilled in the general area of the existing turning dolphin /proposed wave screen at the outboard end of the southerly arm of the current ferry pen. The test borings, designated 9-71 and 10-71, encountered a relatively thick layer of organic silt "muck" over marine silt-clay. In test boring 10-71, located to the northwest of the existing turning dolphin, a layer of gravelly sand was encountered under the marine silt-clay. Split-spoon refusal was reported at depths of 24.7 and 26.0 feet below the mudline in the two test borings. Rock was not cored in either boring. This historic subsurface information is relevant since it corroborates the recent subsurface information with respect to the design of the proposed wave screen.

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 Bass Harbor, Maine. These recommendations are based on geotechnical provisions set forth in the AASHTO LRFD Bridge Design Specifications, 7th 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 Bass Harbor, Swans Island, Vinalhaven, and Frenchboro 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 basalt underlying the Bass Harbor site.

TURNING DOLPHINS - GENERAL

The structural analysis of the turning dolphins 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 Top elevation -36.8 ft	Sand (Reese)	Unit weight = 110 pcf Friction angle = 29 deg
2	18-ft thick layer MARINE SILT-CLAY Top elevation -40.8 ft	Clay (Soft, Matlock)	Unit weight = 110 pcf Undrained shear strength = 400 psf
3	3.5-ft thick layer GLACIAL TILL Top elevation -58.5 ft	Sand (Reese)	Unit weight = 120 pcf Friction angle = 32 deg
4	HARD ROCK (GRANITE) Top elevation -62 ft	GRANITE	Effective unit weight = 170 pcf Uniaxial compressive strength = 2,160,000 psf

TURNING DOLPHINS – PIPE PILES

The proposed 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 basalt underlying Swans Island, 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 (φ_{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 (basalt) that underlies the Swans Island 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 Swans Island test borings, together with the rock core from the borings drilled at Frenchboro, 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 Swans Island 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 Bass Harbor terminal call for the installation of one timber wave screen between the existing outboard (turning) dolphin and the proposed dolphin that will form the southerly side of the pen. The proposed wave screen will be supported by an A-frame of timber piles that will be driven to rock. As part of the design process, SchonewaldEA evaluated the uplift resistance provided by a single timber pile. The analysis is included in the geotechnical calculations provided as Appendix D. The analysis indicates that the factored uplift resistance of a single timber pile used to support the Bass Harbor wave screen can be taken as approximately 4.8 kips.

The timber piles should be driven to bedrock and should be fitted with a pile tip to protect the tip from damage. The contractor should establish driving criteria based upon the hammer system proposed for use and should evaluate drivability so that the piles are not damaged when driven to bedrock.

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 Bass Harbor, 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)

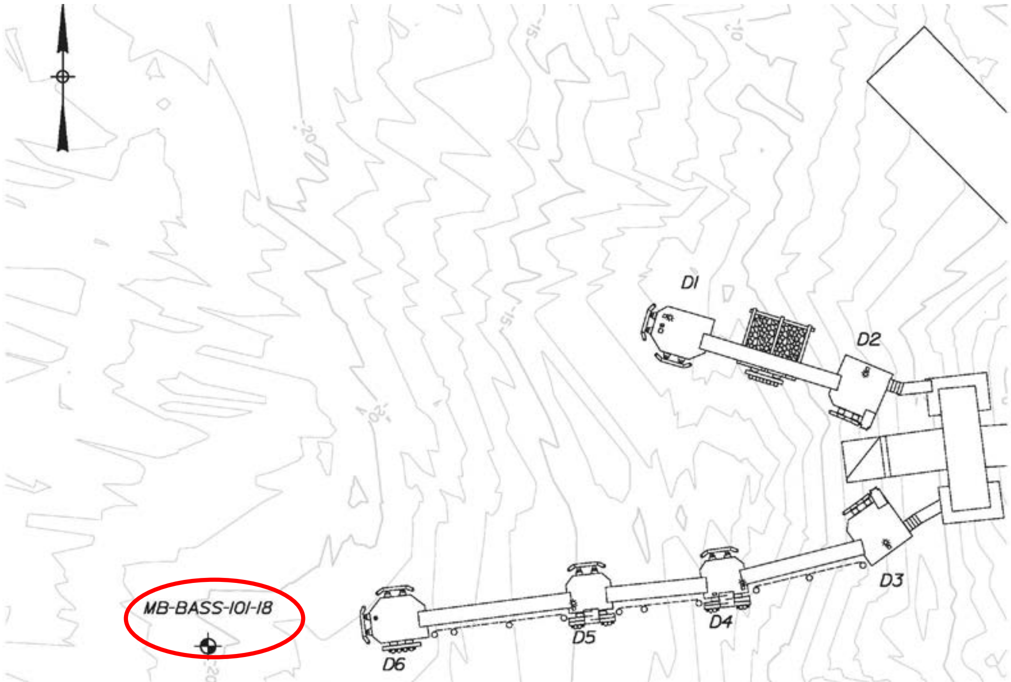
SCHONEWALD
ENGINEERING
ASSOCIATES, INC.

PROJECT NO.:	18-013
DATE:	MAY 2019
DRAWN BY:	IVS
APPROX. SCALE:	AS NOTED

LOCUS PLAN
FENDER SYSTEM MODIFICATIONS
MAINE STATE FERRY TERMINAL
BASS HARBOR, MAINE
MAINEDOT WIN 23476.00

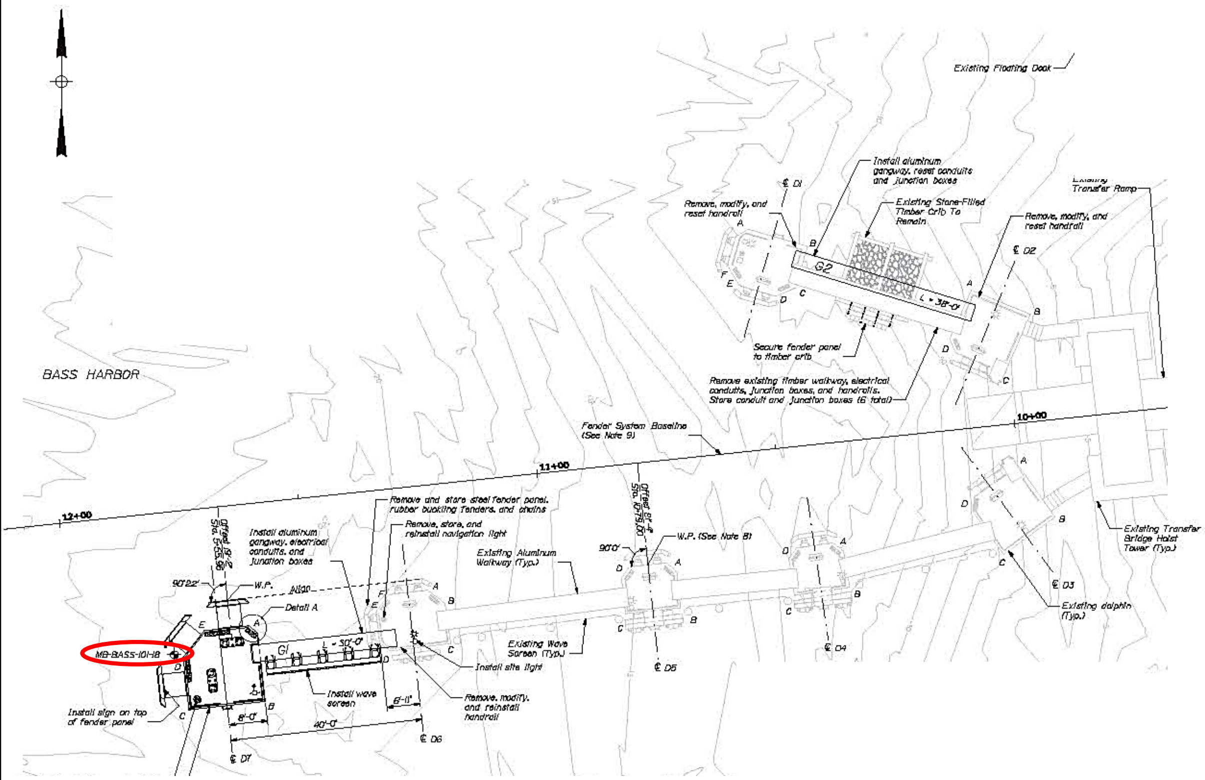
Figure No.:

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 5 OF PROJECT PLAN SET PREPARED BY HNTB)

SCHONEWALD ENGINEERING ASSOCIATES, INC.

PROJECT NO.:	**18-013**
DATE:	**MAY 2019**
DRAWN BY:	**IVS**
APPROX. SCALE:	**AS NOTED**

BORING LOCATION PLAN
FENDER SYSTEM MODIFICATIONS
MAINE STATE FERRY TERMINAL
BASS HARBOR, MAINE
MAINEDOT WIN 23476.00

Figure No.:

2

SCHONEWALD ENGINEERING ASSOCIATES, INC.

APPENDIX A

SUBSURFACE EXPLORATION LOG

Schonewald Engineering Associates, Inc.	**PROJECT:** Fender System Modifications Maine State Ferry Terminal **LOCATION:** Base Harbor, Maine	**Boring No.:** MB-BASS-101-18 **WIN:** 23476.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	-19.8 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:	8/8/18; 1230 - 8/9/18; 1145	**Drilling Method:**	cased wash borings	**Hammer Type:**	Automatic
Boring Location:	per plan (B2A)	**Casing ID/OD:**	HW to 42.0 ft / NW to 42.2 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	1D	24/3	0.0 - 2.0	WOR/18"-WOH	3"dia		PUSH			1D: SHELL FRAGMENTS; (live) crab in tip of spoon (not jarred).	
5	2D	24/24	5.0 - 7.0	WOR/24"	--		PUSH			2D: Grey brown, v. soft, ORGANIC SILT, some fine sand with occasional shells and wood. RECENT MARINE SEDIMENTS	
10	3D	24/24	10.0 - 12.0	WOR/24"	--		PUSH			3D: Grey brown, v. soft, ORGANIC SILT, some fine sand with occasional shells and wood.	
15											
							37	-36.8		17.0 17.0 ft: Casing fetches up; probable stratum change based on drilling behavior and wash.	
							43				
							42				
20	4D	24/24	20.0 - 22.0	5-5-2-1	7	11	25			4D: Dark grey, loose, SILT, some fine sand, trace to little gravel; possible transition to marine silt-clay.	
							25	-40.8		21.0	
							24				
							27				
							32				
25											

Remarks:

8/8/18; 1224 hrs: 15.0' top turn dolphin #2 to water; 7.7' barge deck to water; 31.8' barge deck to mudline
top of turn dolphin #2: elev. 19.28 ft MLLW
50' approx. center of turn dolphin #2 to borehole; approx. on line with center of walkway between turn dolphin #2 and side dolphin #2.

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.: MB-BASS-101-18

SCHONEWALD ENGINEERING ASSOCIATES, INC.	**PROJECT:** Fender System Modifications Maine State Ferry Terminal **LOCATION:** Base Harbor, Maine	**Boring No.:** MB-BASS-101-18 **WIN:** 23476.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	-19.8 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:	8/8/18; 1230 - 8/9/18; 1145	**Drilling Method:**	cased wash borings	**Hammer Type:**	Automatic
Boring Location:	per plan (B2A)	**Casing ID/OD:**	HW to 42.0 ft / NW to 42.2 ft	**Hammer Efficiency:**	0.906
		Auger ID/OD:	n/a	**Water Level*:**	n/a

IN-SITU SAMPLING AND TESTING:
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LL=Liquid Limit / PL=Plastic Limit / PI=Plasticity Index
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-#200 = percent fines from grain size analysis
UCT qp = peak compressive strength of rock

Depth (ft.)	Sample Information: 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	5D	24/24	25.0 - 27.0	WOR/24"	--		27			5D: Dark grey with occasional black, v. soft, Silty CLAY. MARINE SILT-CLAY	
							32				
							29				
							27				
							31				
30							27				
							31				
							23				
							22				
							25				
35	6D	24/16	35.0 - 37.0	WOR/24"	--		19			6D: Dark grey with occasional black, v. soft, Silty CLAY.	
							19				
							15				
							41	-58.5		38.7	
							55			38.7 ft: Possible stratum change based on sudden increase in casing blows.	
40	7D	24/3	40.0 - 42.0	8-3-2/12"	5	8	--			7D: Dark grey, Silty GRAVEL, some fine to coarse sand. TILL	
							--	-62.0		42.2	
	R1	17/14	42.2 - 43.6	RQD: 0"=0%			NQ2			Top of bedrock at Elev. -62.0 ft. R1: Bedrock: Very hard, typically fresh, aphanitic to fine grained, dark grey BASALT with thin quartzite veins. Very close, typically low angle breaks; undulating, rough, typically fresh, and open with mud infilling typical. (Castine Formation) Core times: 3:35/ - min:sec/ft ROCK QUALITY = VERY POOR	
	R2	52/52	43.6 - 47.9	RQD: 30"=58%						R2: Similar to R1, except with numerous low angle healed fractures and close to widely spaced breaks. -/ 2:40/ 2:50/ 2:35/ - min:sec/ft ROCK QUALITY = FAIR	
45											
				sample 46.2-46.7							UCT qp: specimen chipped; not viable for testing
	R3	60/60	47.9 - 52.9	RQD: 47"=78%						R3: Similar to R1, except with occasional low angle healed fractures and close to widely spaced, low angle to moderately dipping breaks with occasional mud infilling. 4:15/ 3:10/ 3:55/ 3:30/ 3:50 min:sec/ft	
50											

Remarks:

8/8/18; 1224 hrs: 15.0' top turn dolphin #2 to water; 7.7' barge deck to water; 31.8' barge deck to mudline
top of turn dolphin #2: elev. 19.28 ft MLLW
50' approx. center of turn dolphin #2 to borehole; approx. on line with center of walkway between turn dolphin #2 and side dolphin #2.

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: Base Harbor, Maine

Boring No.: MB-BASS-101-18
WIN: 23476.00

Driller:	New England Boring Contractors	**Elevation (ft.)**	-19.8 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:	8/8/18; 1230 - 8/9/18; 1145	**Drilling Method:**	cased wash borings	**Hammer Type:**	Automatic
Boring Location:	per plan (B2A)	**Casing ID/OD:**	HW to 42.0 ft / NW to 42.2 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										ROCK QUALITY = FAIR TO GOOD	
				sample 50.8-51.2							UCT qp = 15.89 ksi
	R4	46/46	52.9 - 56.7	sample 52.3-52.8 RQD: 10"=22%						R4: Similar to R1, except with occasional moderately dipping healed fractures and close to moderately spaced, typically moderately dipping breaks with mud infilling typical. 3:00 / 3:35/ 4:15/ - min:sec/ft ROCK QUALITY = VERY POOR TO POOR	UCT qp = 14.05 ksi
55											
	R5	38/38	56.7 - 59.9	RQD: 10"=26%						R5: Similar to R1, except with occasional healed fractures and typically close, typically moderately dipping breaks; typically fresh with occasional mud infilling. 2:40/ 2:10/ 2:15/ 2:05/ - min:sec/ft ROCK QUALITY = VERY POOR TO POOR	
	R6	55/55	59.9 - 64.5	RQD: 18"=33%						R6: Similar to R1, except with occasional moderately dipping healed fractures and typically close, typically moderately dipping breaks; typically fresh with occasional mud infilling. -/ 4:25/ 3:55/ 3:40/ - min:sec ft ROCK QUALITY = POOR	
60											
	R7	49/49	64.5 - 68.6	RQD: 27"=55%						R7: Similar to R1, except with occasional low angle to moderately dipping healed fractures and typically close to moderately spaced, typically moderately dipping breaks; fresh with occasional mud infilling. 3:25/ 4:00/ 4:05/ 4:35/ - min:sec ft ROCK QUALITY = FAIR	
65											
	R8	47/47	68.6 - 72.5	RQD: 25"=53%						R8: Similar to R1, except with occasional moderately dipping healed fractures and typically close to moderately spaced, typically moderately dipping breaks; fresh with occasional mud infilling. 3:50/ 4:10/ 5:10/ - min:sec ft ROCK QUALITY = POOR TO FAIR	
70											
								-92.3		72.5 **Bottom of Exploration at 72.5 feet below ground surface.**	
75											

Remarks:

8/8/18; 1224 hrs: 15.0' top turn dolphin #2 to water; 7.7' barge deck to water; 31.8' barge deck to mudline
top of turn dolphin #2: elev. 19.28 ft MLLW
50' approx. center of turn dolphin #2 to borehole; approx. on line with center of walkway between turn dolphin #2 and side dolphin #2.

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.

APPENDIX B

ROCK CORE PHOTOGRAPHS

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

1) MB-BASS-101, R1 and R2;
2) MB-BASS-101, R2 and R3;
3) MB-BASS-101, R3 and R4;
4) MB-BASS-101, R4.

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

1) MB-BASS-101, R2;
2) MB-BASS-101, R3;
3) MB-BASS-101, R3 and R4;
4) Empty.

SCHONEWALD ENGINEERING ASSOCIATES, INC.

ROCK CORE PHOTOGRAPHS
FENDER SYSTEM MODIFICATIONS
MAINE STATE FERRY TERMINAL
BASS HARBOR, MAINE
MAINEDOT WIN 23476.00

Sheet No.:

1 of 2

Photo 3: Core box containing wetted rock core from test boring MB-BASS-101; left side of core box (top portion of cores). Slots from top to bottom:
1) MB-BASS-101, R5;
2) MB-BASS-101, R6;
3) MB-BASS-101, R7 and R8;
4) MB-BASS-101, R8.

Photo 4: Core box containing wetted rock core from test boring MB-BASS-101; right side of core box (bottom portion of cores). Slots from top to bottom:
1) MB-BASS-101, R5 and R6;
2) MB-BASS-101, R6 and R7;
3) MB-BASS-101, R7 and R8;
4) MB-BASS-101, R8.

SCHONEWALD ENGINEERING ASSOCIATES, INC.

ROCK CORE PHOTOGRAPHS
FENDER SYSTEM MODIFICATIONS
MAINE STATE FERRY TERMINAL
BASS HARBOR, MAINE
MAINEDOT WIN 23476.00

Sheet No.:

2 of 2

SCHONEWALD ENGINEERING ASSOCIATES, INC.

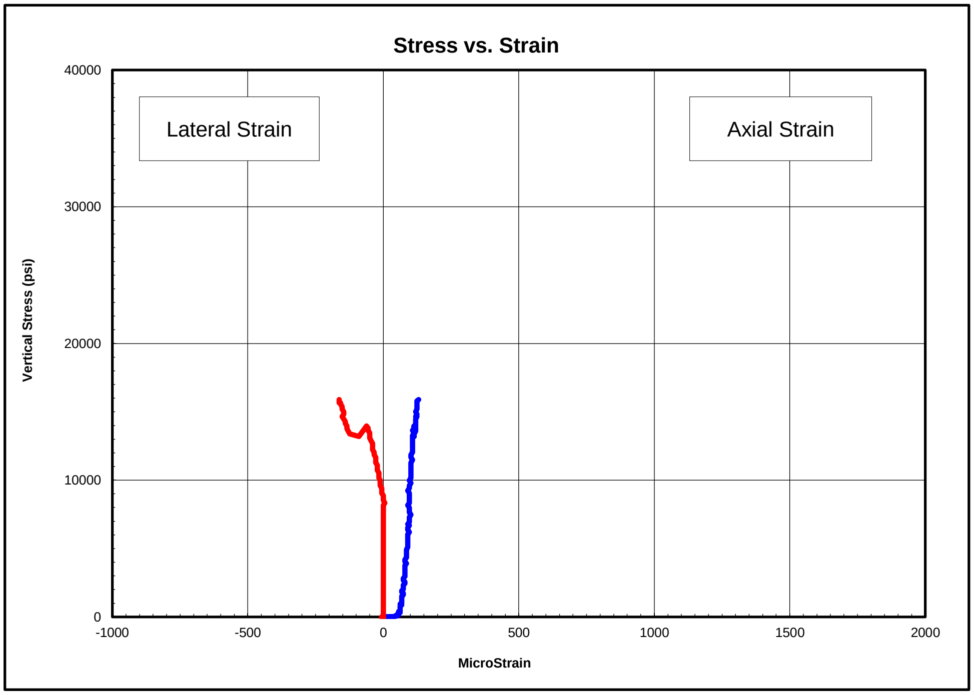
APPENDIX C

LABORATORY TEST REPORTS

GeoTesting EXPRESS

Client:	Schonewald Engineering Associates, Inc.
Project Name:	MeDOT WIN 23476.00 MSFT Bass Harbor, ME
Project Location:	Bass Harbor, ME
GTX #:	308803
Test Date:	9/20/2018
Tested By:	crs
Checked By:	jsc
Boring ID:	MB-BASS-101
Sample ID:	R3
Depth, ft:	50.8-51.2
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: 15,894 psi

The strain values recorded for this test produce values of Poisson's Ratio that exceed maximum values found in rocks. One axial and one lateral strain gauge failed to record meaningful data. Young's Modulus reported based on results of a single axial and lateral strain gauge.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1600-5800	179,000,000	---
5800-10100	166,000,000	---
10100-14300	182,000,000	---

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:	Schonewald Engineering Associates, Inc.	Test Date:	9/19/2018
Project Name:	MeDOT WIN 23476.00 MSFT Bass Harbor, ME	Tested By:	crs
Project Location:	Bass Harbor, ME	Checked By:	jsc
GTX #:	308803		
Boring ID:	MB-BASS-101		
Sample ID:	R3		
Depth:	50.8-51.2 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:	4.22	4.22	4.22
Specimen Diameter, in:	1.99	1.99	1.99
Specimen Mass, g:	578.78		
Bulk Density, lb/ft^3	168	**Minimum Diameter Tolerence Met?**	**YES**
Length to Diameter Ratio:	2.1	**Length to Diameter Ratio Tolerance Met?**	**YES**

DEVIATION FROM STRAIGHTNESS (Procedure S1)

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

Maximum difference must be < 0.020 in.

Straightness Tolerance Met? **YES**

END FLATNESS AND PARALLELISM (Procedure FP1)

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

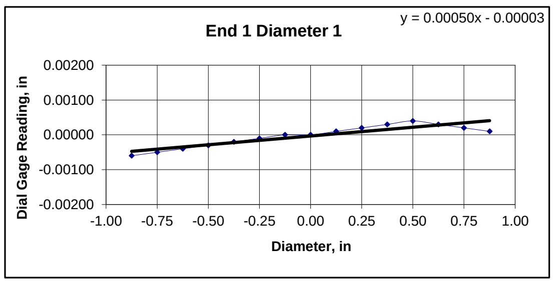
Difference between max and min readings, in:
0° = 0.00100 90° = 0.00020

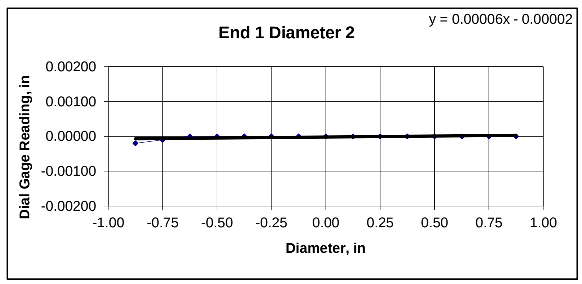
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00060	-0.00050	-0.00040	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00010	0.00020	0.00020	0.00030	0.00040	0.00030	0.00020
Diameter 2, in (rotated 90°)	-0.00020	-0.00010	0.00000	0.00000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

Difference between max and min readings, in:
0° = 0.001 90° = 0.0002

Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00050

Flatness Tolerance Met? **YES**





DIAMETER 1

End 1:

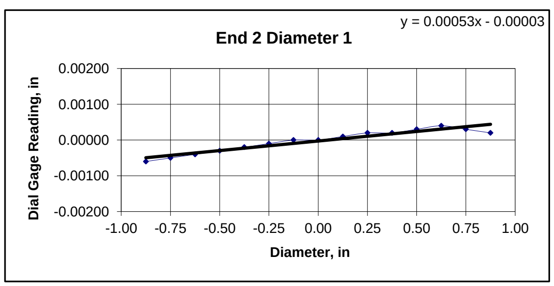
Slope of Best Fit Line	0.00050
Angle of Best Fit Line:	0.02881

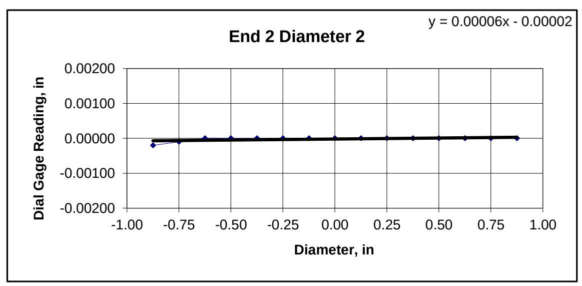
End 2:

Slope of Best Fit Line	0.00053
Angle of Best Fit Line:	0.03061

Maximum Angular Difference: 0.00180

Parallelism Tolerance Met? **YES**
Spherically Seated





DIAMETER 2

End 1:

Slope of Best Fit Line	0.00006
Angle of Best Fit Line:	0.00327

End 2:

Slope of Best Fit Line	0.00006
Angle of Best Fit Line:	0.00327

Maximum Angular Difference: 0.00000

Parallelism Tolerance Met? **YES**
Spherically Seated

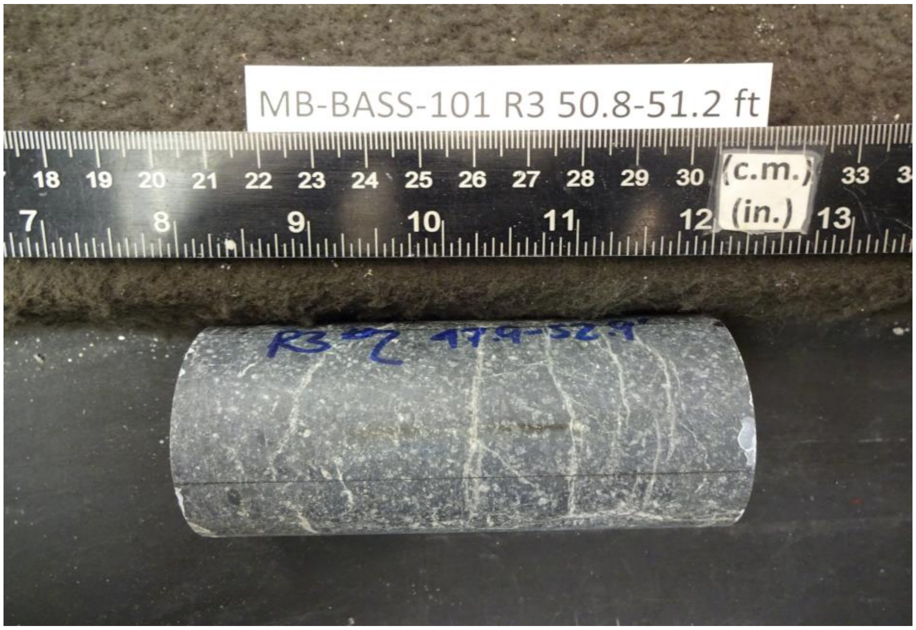
PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

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

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°)	0.00020	1.990	0.00010	0.006	YES
END 2					
Diameter 1, in	0.00100	1.990	0.00050	0.029	YES
Diameter 2, in (rotated 90°)	0.00020	1.990	0.00010	0.006	YES

Perpendicularity Tolerance Met? **YES**

Client:	Schonewald Engineering Associates, Inc.
Project Name:	MeDOT WIN 23476.00 MSFT Bass Harbor, ME
Project Location:	Bass Harbor, ME
GTX #:	308803
Test Date:	9/21/2018
Tested By:	tlm
Checked By:	jsc
Boring ID:	MB-BASS-101
Sample ID:	R3
Depth, ft:	50.8-51.2

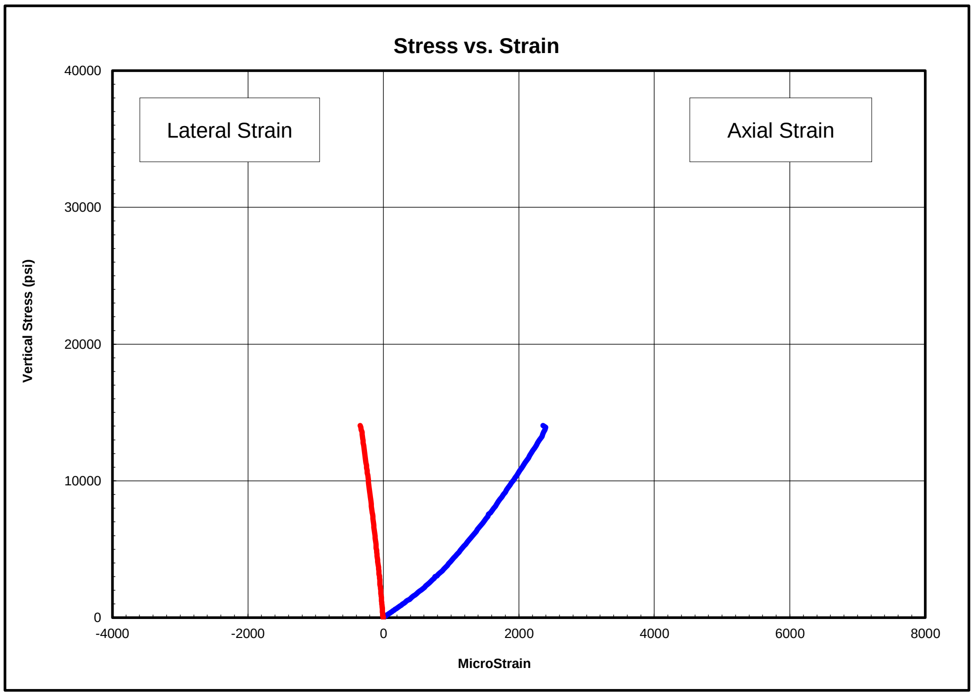


After cutting and grinding

After break

Client:	Schonewald Engineering Associates, Inc.
Project Name:	MeDOT WIN 23476.00 MSFT Bass Harbor, ME
Project Location:	Bass Harbor, ME
GTX #:	308803
Test Date:	9/20/2018
Tested By:	crs
Checked By:	jsc
Boring ID:	MB-BASS-101
Sample ID:	R3
Depth, ft:	52.3-52.8
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: 14,047 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1400-5200	4,780,000	0.10
5200-8900	6,460,000	0.16
8900-12600	7,400,000	0.19

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:	Schonewald Engineering Associates, Inc.	Test Date:	9/19/2018
Project Name:	MeDOT WIN 23476.00 MSFT Bass Harbor, ME	Tested By:	crs
Project Location:	Bass Harbor, ME	Checked By:	jsc
GTX #:	308803		
Boring ID:	MB-BASS-101		
Sample ID:	R3		
Depth:	52.3-52.8 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:	4.28	4.28	4.28
Specimen Diameter, in:	1.99	1.99	1.99
Specimen Mass, g:	591.17		
Bulk Density, lb/ft^3	169	**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.? NO

Maximum difference must be < 0.020 in.
Straightness Tolerance Met? **NO**

END FLATNESS AND PARALLELISM (Procedure FP1)

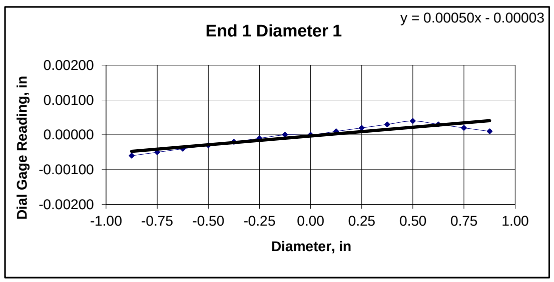
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00060	-0.00050	-0.00040	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00010	0.00020	0.00030	0.00040	0.00030	0.00020	0.00010
Diameter 2, in (rotated 90°)	0.00010	0.00000	0.00000	0.00000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

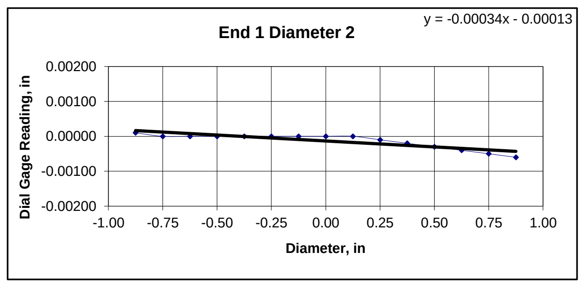
Difference between max and min readings, in:
0° = 0.00100 90° = 0.00070

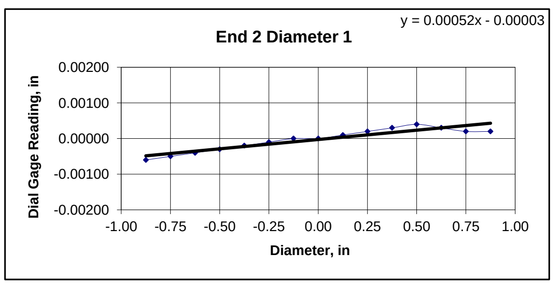
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00060	-0.00050	-0.00040	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00010	0.00020	0.00030	0.00040	0.00030	0.00020	0.00020
Diameter 2, in (rotated 90°)	0.00000	0.00000	0.00000	0.00000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

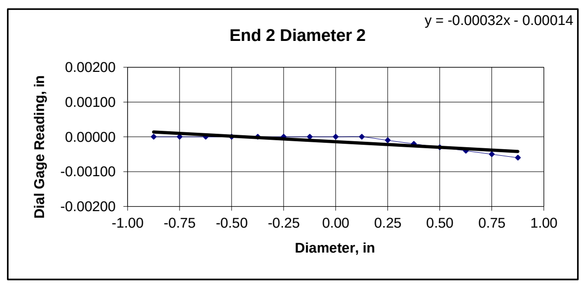
Difference between max and min readings, in:
0° = 0.001 90° = 0.0006

Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00050
Flatness Tolerance Met? **YES**









DIAMETER 1

End 1:	
Slope of Best Fit Line	0.00050
Angle of Best Fit Line:	0.02881
End 2:	
Slope of Best Fit Line	0.00052
Angle of Best Fit Line:	0.02996
Maximum Angular Difference:	0.00115

Parallelism Tolerance Met? **YES**
Spherically Seated

DIAMETER 2

End 1:	
Slope of Best Fit Line	0.00034
Angle of Best Fit Line:	0.01948
End 2:	
Slope of Best Fit Line	0.00032
Angle of Best Fit Line:	0.01833
Maximum Angular Difference:	0.00115

Parallelism Tolerance Met? **YES**
Spherically Seated

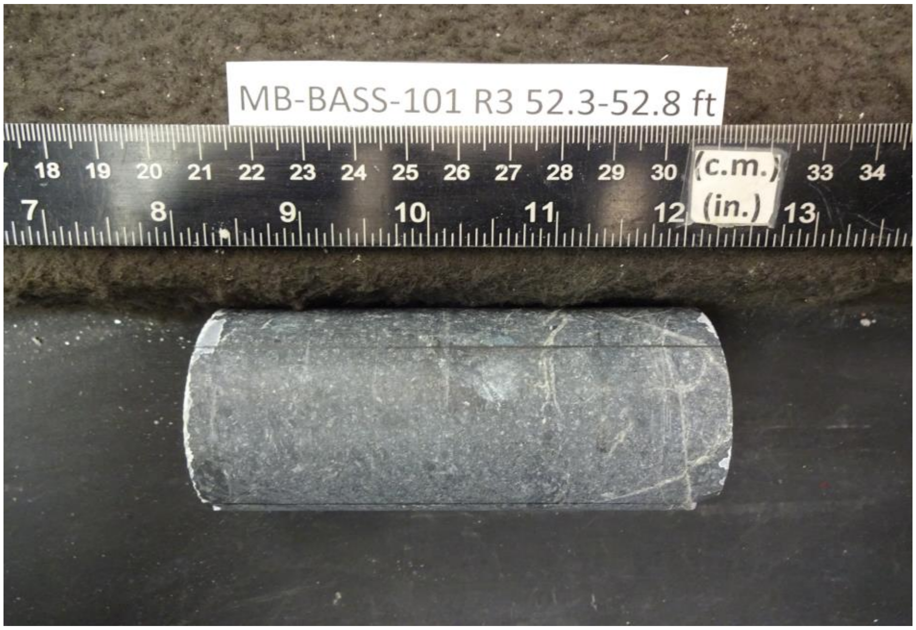
PERPENDICULARITY (Procedure P1) (Calculated from End Flatness and Parallelism measurements above)

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00100	1.990	0.00050	0.029	YES
Diameter 2, in (rotated 90°)	0.00070	1.990	0.00035	0.020	YES
END 2					
Diameter 1, in	0.00100	1.990	0.00050	0.029	YES
Diameter 2, in (rotated 90°)	0.00060	1.990	0.00030	0.017	YES

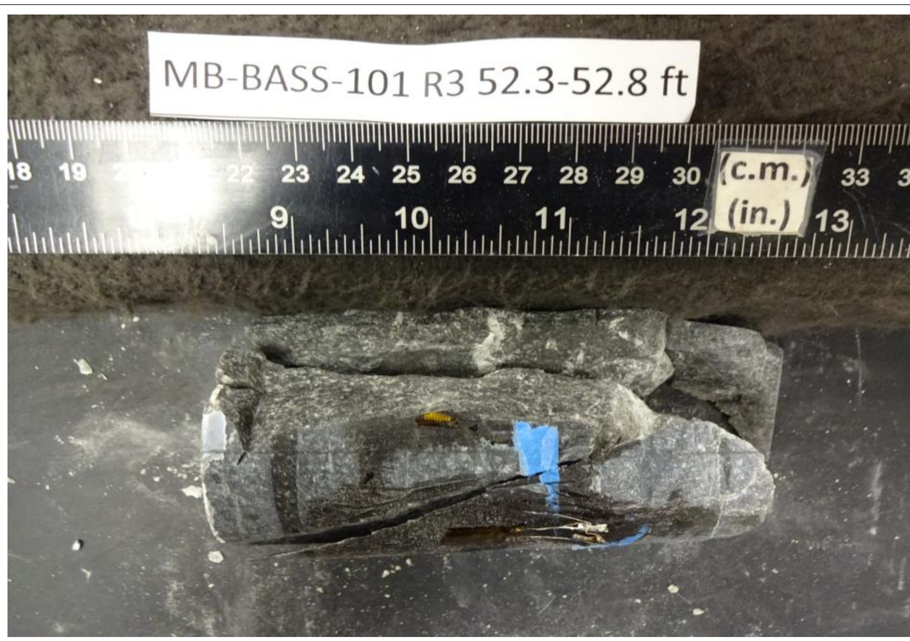
Maximum angle of departure must be $\leq$ 0.25°

Perpendicularity Tolerance Met? **YES**

Client:	Schonewald Engineering Associates, Inc.
Project Name:	MeDOT WIN 23476.00 MSFT Bass Harbor, ME
Project Location:	Bass Harbor, ME
GTX #:	308803
Test Date:	9/21/2018
Tested By:	tlm
Checked By:	jsc
Boring ID:	MB-BASS-101
Sample ID:	R3
Depth, ft:	52.3-52.8

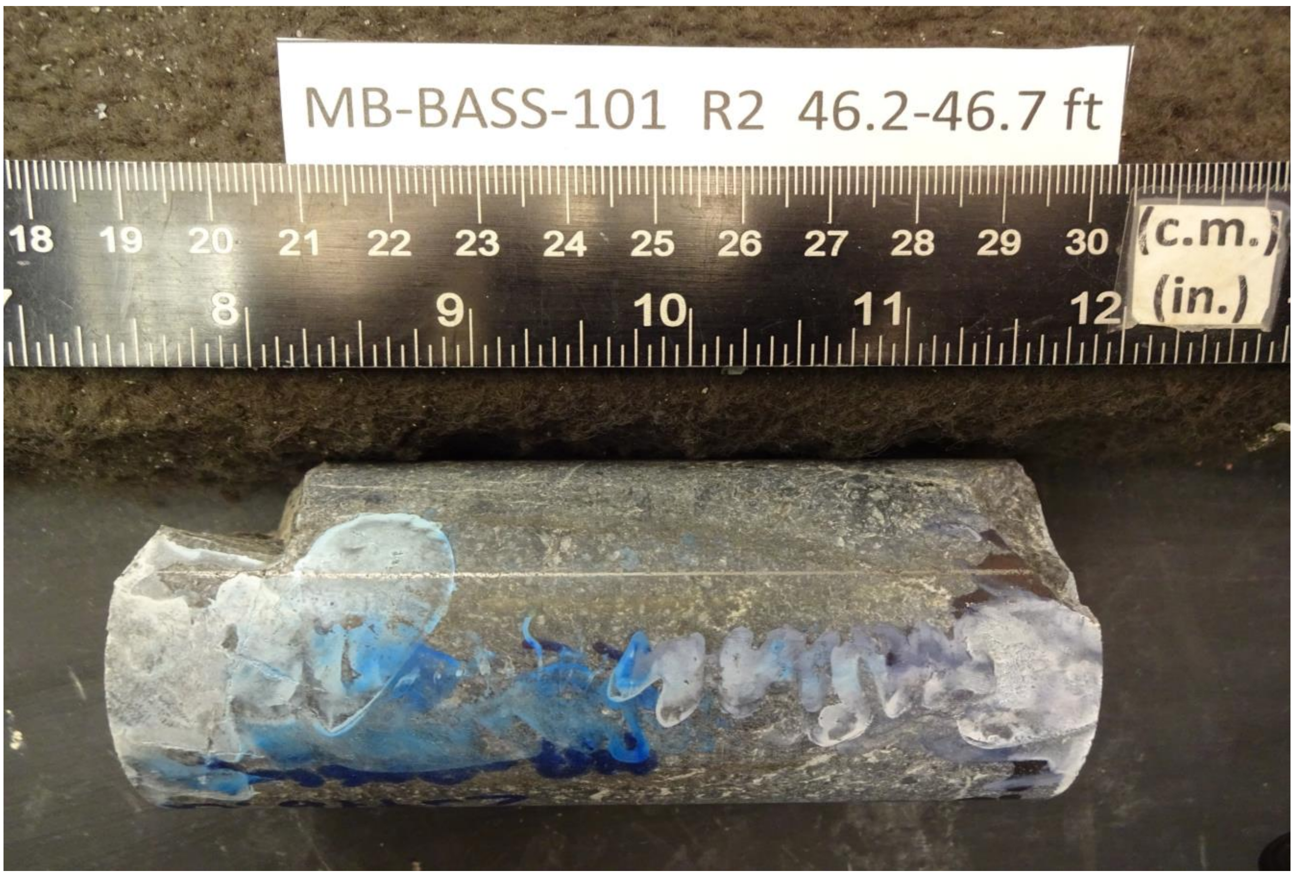


After cutting and grinding



After break

Client:	Schonewald Engineering Associates, Inc.
Project Name:	MeDOT WIN 23476.00 MSFT Bass Harbor, ME
Project Location:	Bass Harbor, ME
GTX #:	308803
Test Date:	09/20/18
Tested By:	crs
Checked By:	jsc
Boring ID:	MB-BASS-101
Sample ID:	R2
Depth, ft:	46.2-46.7 ft



Specimen after cutting showing chipped end

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

<u>TEST BORINGS</u>: 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.

<u>LABORATORY TESTS</u>: 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
16.4
15.9
12.6
~~7.9~~
~~6.7~~
16.8
~~7.1~~
~~26.2~~
~~27.5~~

average peak uniaxial compressive strength = 16.4 ksi = 16,446 psi

throw out upper and lower results (strikethroughs),
average peak uniaxial compressive strength = 15.6 ksi = 15,571 psi

minimum peak uniaxial compressive strength = 6.7 ksi = 6,700 psi

maximum peak uniaxial compressive strength = 29.4 ksi = 29,400 psi

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

TEST BORINGS: 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.

LABORATORY TESTS: 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

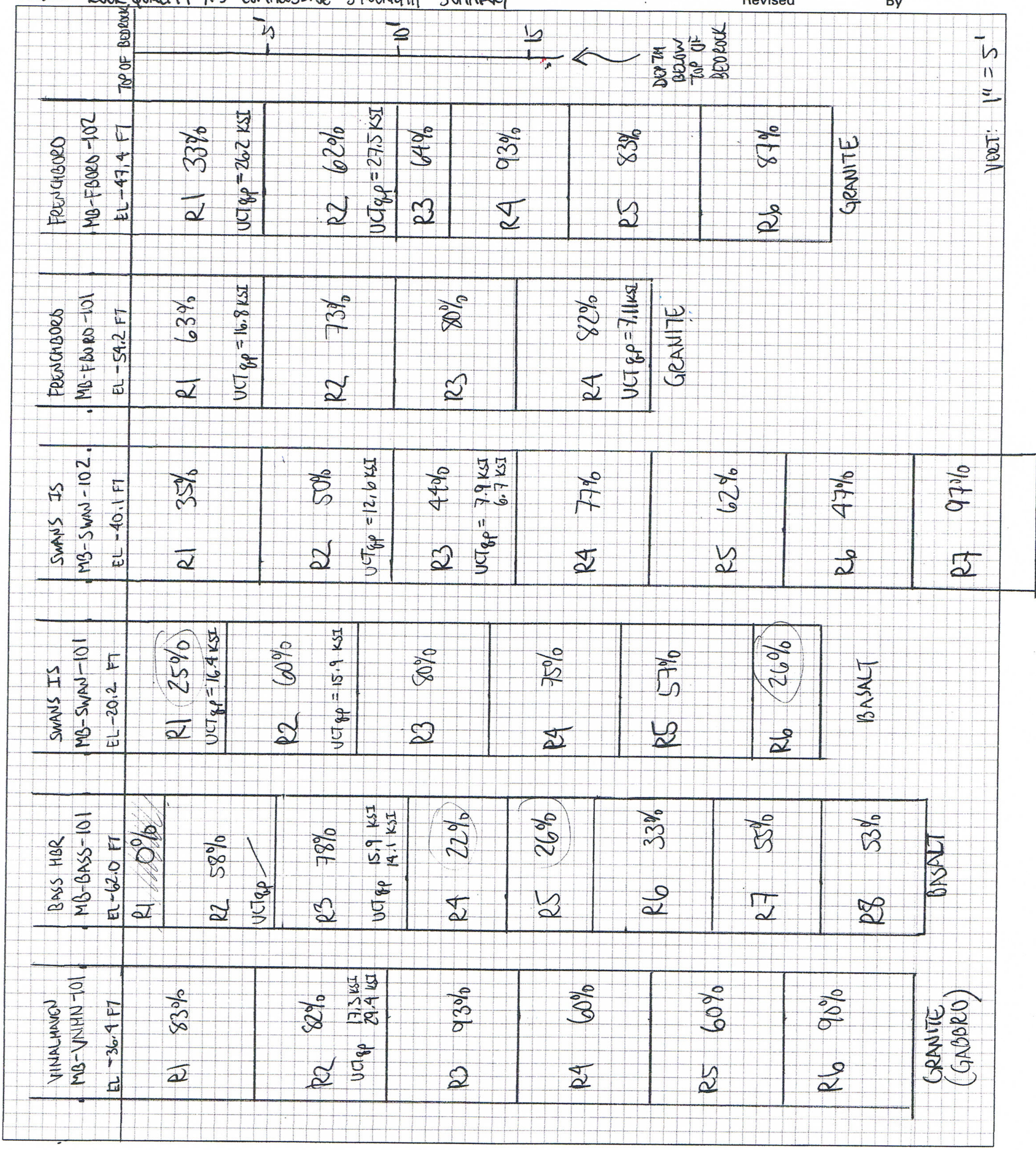
ISLESBORO

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)

SCHONEWALD ENGINEERING ASSOCIATES, INC.
129 Middle Road, Cumberland, ME 04021
Be@SchonewaldEngineering.com
www.SchonewaldEngineering.com

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	



SCHONEWALD ENGINEERING ASSOCIATES, INC.
129 Middle Road, Cumberland, ME 04021
Be@SchonewaldEngineering.com
www.SchonewaldEngineering.com

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

TOP OF BEDROCK

DEPTH BELOW TOP OF BEDROCK (FT): 5' — 10' — 15' —

ISLESBORO MB-ISLES-101, EL -31.1 FT
- R1 78%, UCT qp = 11.1 KSI
- R2 78%
- R3 75%, UCT qp = 8.2 KSI
- R4 60%, UCT qp = 8.5 KSI
- R5 32%
- R6 83%

METAPELITE

ISLESBORO MB-ISLES-102, EL -37.4 FT
- R1 19%
- R2 20%, UCT qp = 1.0 KSI
- R3 0%
- 13.0'
- R4 72%, UCT qp = 6.9 KSI
- R5 73%
- R6 80%, UCT qp = 8.5 KSI
- R7 85%

METAPELITE

ISLESBORO MB-ISLES-103, EL -22.6 FT
- R1 8%
- R2 0%
- R3 17%
- 10.0'
- R4 57%, UCT qp = 7.0 KSI, 7.6 KSI
- R5 80%, UCT qp = 6.11 KSI
- R6 85%
- R7 85%

METAPELITE

ISLESBORO MB-ISLES-104, EL -24.7 FT
- R1 55%, UCT qp = 6.1 KSI
- R2 42%
- R3 (60% REC.) 13%, UCT qp = 1.4 KSI
- R4 (NO REC.) 0%
- 16.8'
- R4A 63%
- R5 78%
- R6 62%, UCT qp 5.2 KSI
- R7 60%
- R8 98%

METAPELITE

VERT: 1" = 5'

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*φ_{stat} where: FS = factor of safety
φ_{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*φ_{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 φ_{up} = 0.35 and after the alpha method is φ_{up} = 0.25.

$R_R = R_s\varphi_{up}$ where: R_R = factored uplift resistance
φ_{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

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

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

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

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

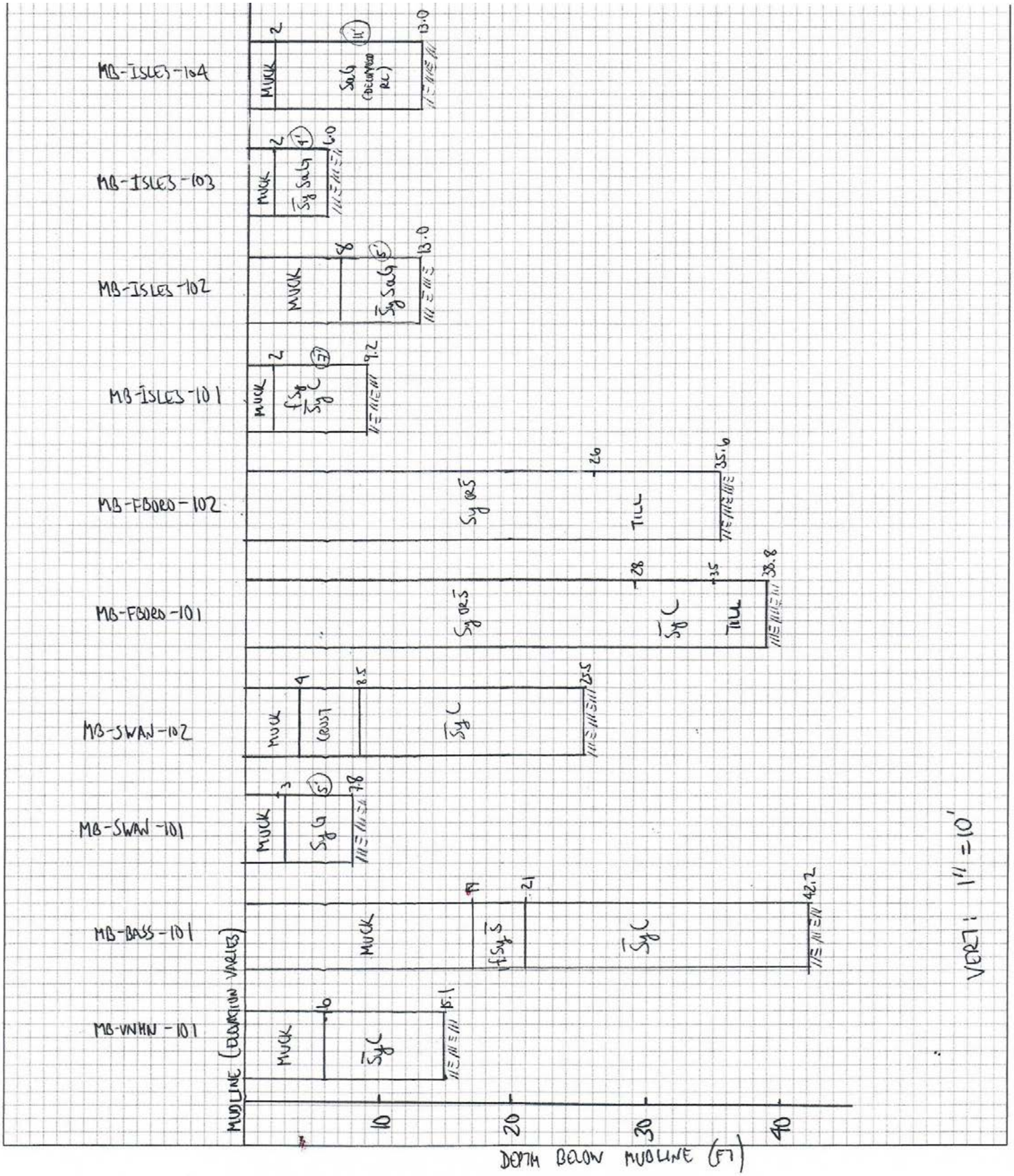
NOTE applies to the prop. Bass Harbor wave screen; could be conservatively low, but have limited subsurface data

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



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

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

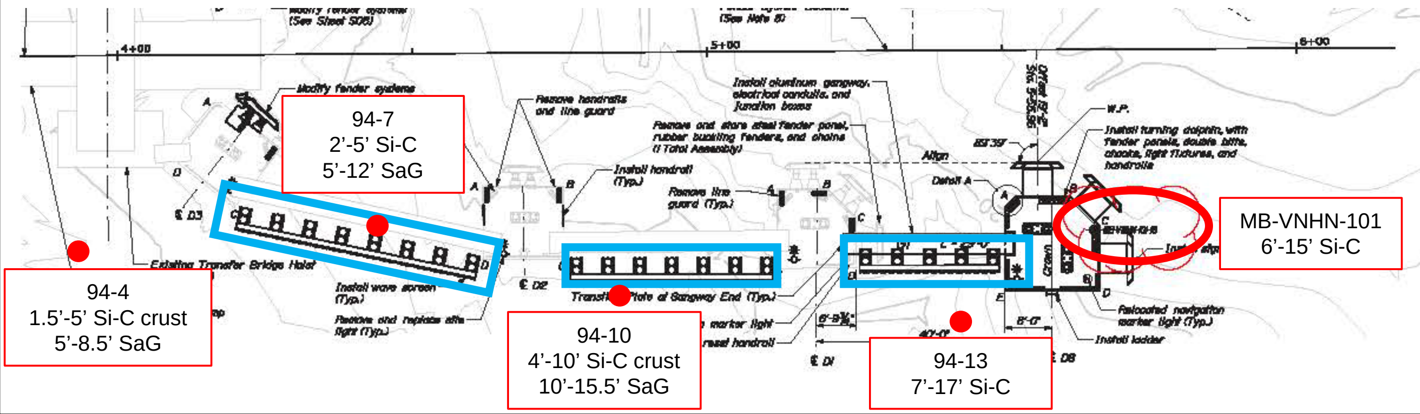
Subject: Geotechnical uplift capacity of wave screen timber piles **Last updated:** Apr 2019 **By** IVS

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

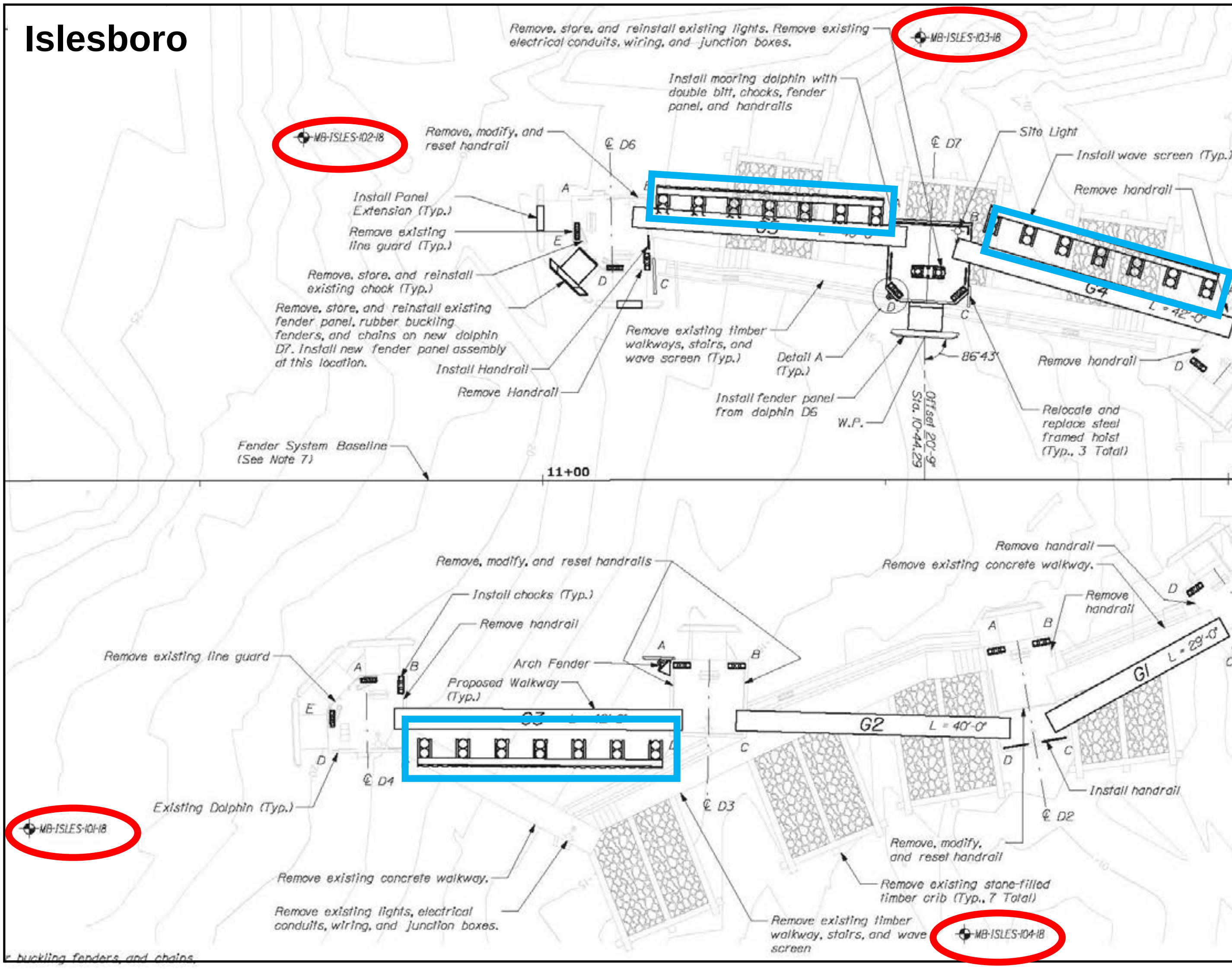
Legend (Pg 5 & 6):

- Prop. Wave Screen
- Recent Boring
- 94-13 7'-17' Si-C — Approx. Historic Boring w/ Summary of Conditions

Vinalhaven



Islesboro



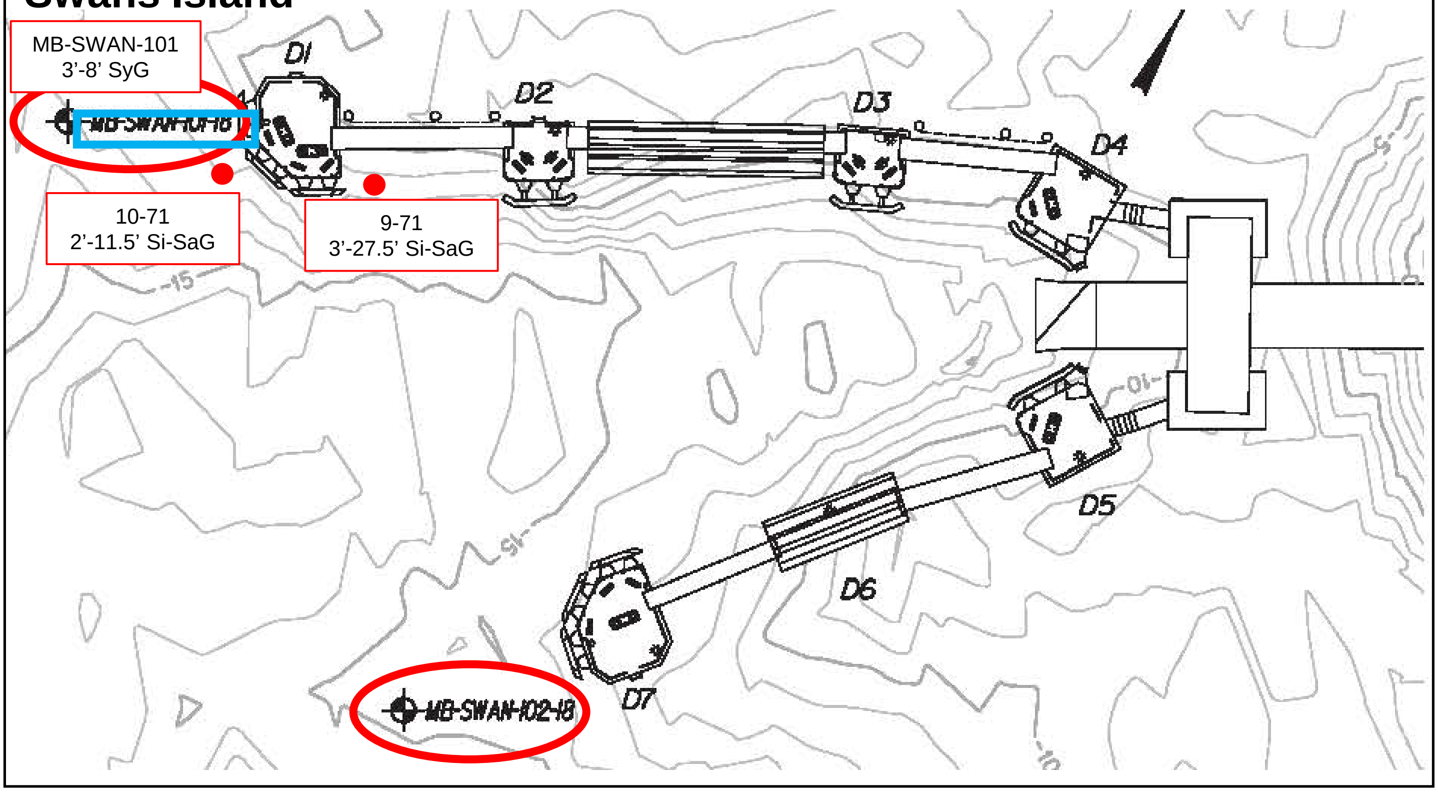
Project: MaineDOT MSFT Fender System Modifications **Proj. No.** 18-010 - 18-014

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