



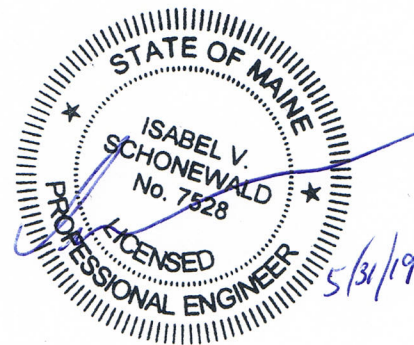
**GEOTECHNICAL DESIGN REPORT
FENDER SYSTEM MODIFICATIONS
MAINE STATE FERRY TERMINAL - VINALHAVEN, MAINE
MAINEDOT WIN 23823.00**

PREPARED FOR:

HNTB Corporation
Westbrook, Maine

PREPARED BY:

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SchonewaldEA Project No. 18-010

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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 Vinalhaven, Maine on West Penobscot Bay (MaineDOT WIN 23823.00).

SchonewaldEA's work on this project has been completed under a Subconsultant Task Order Agreement with HNTB that is dated March 29, 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 Vinalhaven, Maine on West Penobscot Bay is depicted on attached Figure 1 – Locus Plan.

The fender system modifications at the Maine State Ferry Terminal in Vinalhaven include the installation of two turning dolphins outboard of the footprint of the existing ferry pen, complete with fender panels and lighting. The proposed turning dolphins will be located approximately 40 feet further offshore from the two existing outboard turning dolphins to increase the length of the ferry slip in order to accommodate a larger vessel. Both turning dolphins 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 three timber wave screens between the three existing and one proposed dolphins that form the westerly side of the pen. Each of the three wave screens 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 shallow bedrock (thin drift) (Surficial Geologic Map of Maine). Bedrock geology is mapped as Plutonic Rocks of Devonian age (Bedrock Geology of North Haven and Vinalhaven Islands, Maine). Specifically, a biotite-hornblende granite with gabbro intrusions is mapped in the vicinity of the site.

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 program. These included Vinalhaven, Bass Harbor, Swans Island, Frenchboro, and Islesboro. As part of the overall program, one test boring was drilled at the Vinalhaven terminal in the vicinity of the proposed westerly turning dolphin to evaluate subsurface conditions. At the time of the subsurface investigation, only one new turning dolphin was proposed. The test boring was designated MB-VNHN-101. The test boring was drilled on August 7, 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 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-VNHN-101	R2	22.8 to 23.4 ft. BGS	bedrock; uniaxial compressive strength test
MB-VNHN-101	R2	23.6 to 24.0 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 log included in Appendix A and the laboratory test report is included as Appendix C.

SUBSURFACE CONDITIONS

Subsurface conditions encountered in the Vinalhaven test boring consisted of approximately 6 feet of very loose Organic Silt "Muck" (recent sediment) over an approximately 9-foot thickness of very soft Marine Silt-Clay. Bedrock underlying the Marine Silt-Clay consisted of a fine- to medium-grained Granite. Detailed descriptions of the overburden encountered in the test boring are provided on the log included in Appendix A.

Multiple test borings were drilled in early 1995 in the general area of the westerly arm of the current ferry pen. The test borings, designated 94-4, 94-7, 94-10, 94-13, encountered a relatively thin layer of organic silt "muck" over a varying thickness of marine silt-clay. In the borings located closer to shore, the marine silt-clay was underlain by a silty sand and gravel. The 1995 test boring that was drilled in the vicinity of

the existing outboard dolphin on the easterly arm of the ferry pen, designated 94-12, encountered approximately 5.5 feet of organic silt “muck” over 12 feet of very soft marine silt-clay that was underlain by approximately 8 feet of silty sand and gravel. Rock was cored in each boring, but the boring logs do not provide a description of the rock or of its quality.

Bedrock was encountered at approximate elevation -36.4 feet, which corresponds to a depth of 15.1 feet below the ground surface (BGS). The bedrock core obtained from the test boring consisted of very hard, typically fresh, fine- to medium-grained, medium grey to pink grey GRANITE, that changed to a very hard, typically fresh, aphanitic to fine-grained, dark grey GABBRO at approximately elevation -64.9 feet. Both GRANITE and GABBRO are mapped in the area and are very hard crystalline plutonic igneous rocks.

The Rock Quality Designations (RQDs) of the rock cores obtained in the test boring ranged from 60 to 93 percent. Two specimens of the rock core were submitted for uniaxial compressive strength tests. The test results reported the uniaxial compressive strength of the GRANITE to be 17.3 and 29.4 ksi.

Detailed descriptions of the rock encountered in the Vinalhaven 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 plutonic 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 Vinalhaven, Maine on West Penobscot Bay. 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 Vinalhaven, Frenchboro, Bass Harbor, and Swans Island were all similar very hard crystalline plutonic 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 and GABBRO underlying the Vinalhaven site.

TURNING DOLPHINS - GENERAL

The structural analysis of the turning dolphins was completed by HNTB using FB Multiplier software. HNTB identified the critical design scenario to be the greatest elevation difference from the top of the dolphin to bedrock. 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. For the four Maine State Ferry Terminal sites underlain by very hard crystalline plutonic rock, boring MB-BASS-101 at Bass Harbor was the critical case. 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

Each 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 Vinalhaven, 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 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 ($\phi_{stat} = 0.5$), 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 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 plutonic igneous rock (granite) that underlies the Vinalhaven terminal, the 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 Vinalhaven test boring, together with the rock core from the borings drilled at Frenchboro, Bass Harbor, and Swans Island, 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 Vinalhaven 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 one lift off test conducted on one rock anchor as part of the final post-tensioning process.

WAVE SCREENS

The fender system modifications at the Vinalhaven terminal call for the installation of wave screens between the three existing and one proposed dolphins that form the westerly side of the pen. Each of the three proposed wave screens 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 Vinalhaven wave screens is approximately 1.4 kips. The calculated factored uplift resistance is relatively low due to the limited thickness and the very soft consistency of the the overburden soils encountered in the test boring.

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 Vinalhaven, Maine in accordance with generally accepted geotechnical and foundation engineering practices. No other intended use or warranty is expressed or implied.

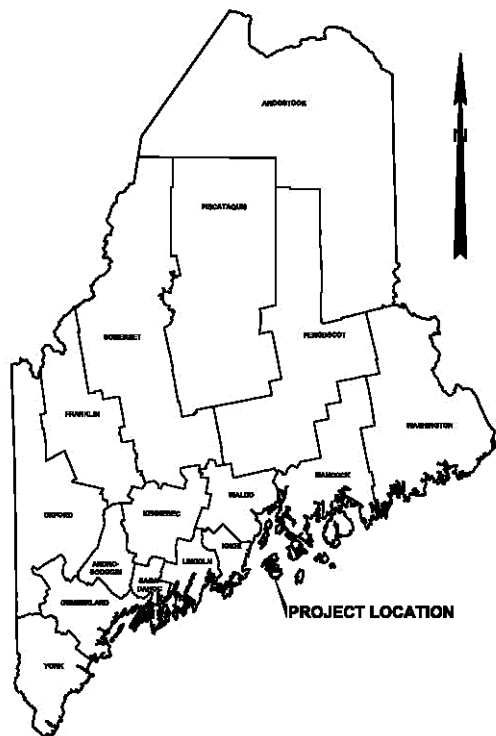
In the event that any changes in the nature, design, or location of the proposed project are planned, this report should be reviewed by a geotechnical engineer to assess the appropriateness of the conclusions and recommendations and to modify the recommendations as appropriate to reflect the changes 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.

FIGURES



Scale in Miles



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



PROJECT NO.: 18-010
DATE: APRIL 2019
DRAWN BY: IVS
APPROX. SCALE: AS NOTED

LOCUS PLAN
FENDER SYSTEM MODIFICATIONS
MAINE STATE FERRY TERMINAL
VINALHAVEN, MAINE
MAINEDOT WIN 23823.00

Figure No.:

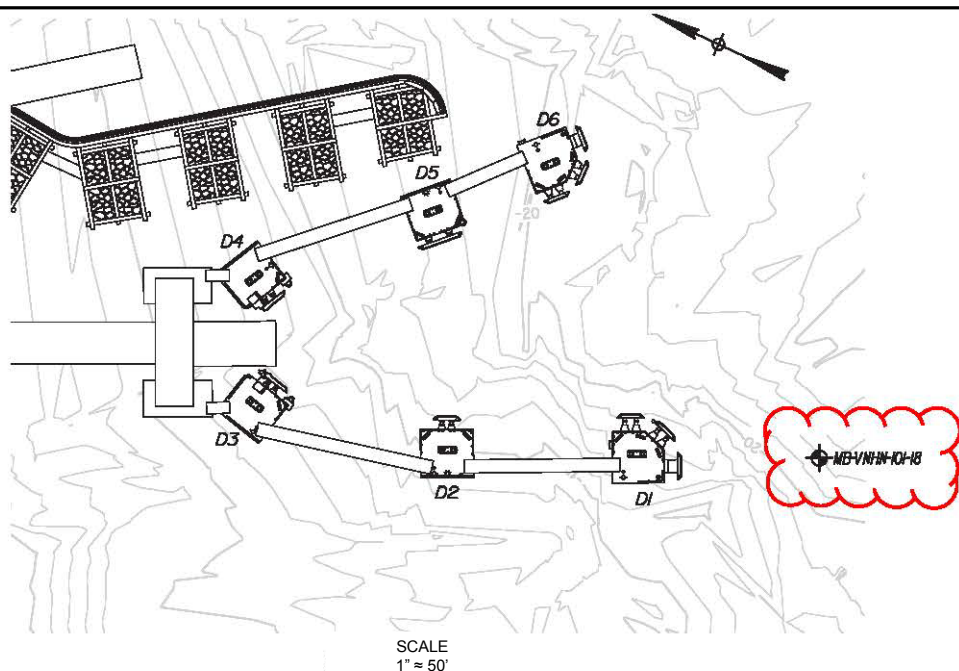
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DATE: APRIL 2019
DRAWN BY: IVS
APPROX. SCALE: AS NOTED

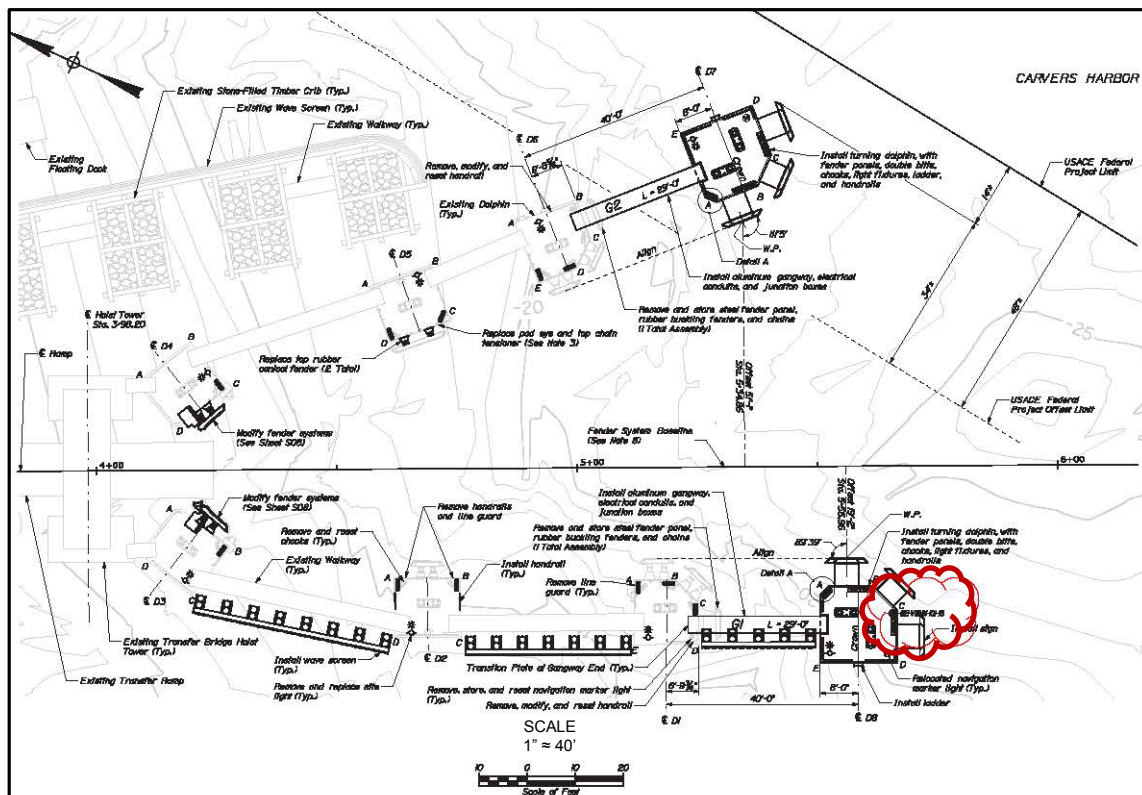
BORING LOCATION PLAN
FENDER SYSTEM MODIFICATIONS
MAINE STATE FERRY TERMINAL
VINALHAVEN, MAINE
MAINEDOT WIN 23823.00

Figure No.:

2



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



EXPLORATION LOCATION PLAN
RELATIVE TO PROPOSED CONDITIONS
(BASE PLAN TAKEN FROM SHEET 5 OF PROJECT PLAN SET PREPARED BY HNTB)



APPENDIX A

SUBSURFACE EXPLORATION LOG

SCHONEWALD ENGINEERING ASSOCIATES, INC.		PROJECT: Fender System Modifications Maine State Ferry Terminal LOCATION: Vinalhaven, Maine		Boring No.: MB-VNHN-101-18 WIN: 23823.00																																																																																	
Driller: New England Boring Contractors		Elevation (ft.): -21.3 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: 8/7/18; 0830-1430		Drilling Method: cased wash boring		Hammer Type: auto hammer																																																																																	
Boring Location: per plan (B1A); see remarks		Casing ID/OD: HW to 14.7 ft/ NW to 15.2 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																																																																																	
<table><tr><th rowspan="2">Depth (ft.)</th><th colspan="7">Sample Information</th><th rowspan="2">Graphic Log</th><th rowspan="2">Visual Description and Remarks</th><th rowspan="2">Lab. Testing Results</th></tr><tr><th>Sample No.</th><th>Pen./Rec. (in.)</th><th>Sample Depth (ft.)</th><th>Blows (6 in.) Shear Strength (psf) or RQD (%)</th><th>N-uncorrected</th><th>N-60</th><th>Casing Blows</th><th>Elevation (ft.)</th></tr><tr><td>0</td><td>1D</td><td>24/20</td><td>0.0 - 2.0</td><td>WOR/24"</td><td>3"dia</td><td></td><td>PUSH</td><td></td><td rowspan="10"></td><td rowspan="10">1D: Dark grey, ORGANIC SILT, trace to little fine sand with shells; strong organic odor. Dark grey, fine to medium Sandy ORGANIC SILT; changing at 5.8 ft to: 2D: Dark grey, Silty CLAY. 13.6 ft: Unable to push HW past 13.6 ft. 14.7 ft: HW refusal. Roller cone to 15.1 ft. Spin NW to 15.2 ft. Top of bedrock at Elev. -36.4 ft. R1: Bedrock: Very hard, typically fresh, fine to medium grained, medium grey GRANITE with occasional feldspar phenocrysts. One high-angle break (along crack); undulating, rough, slightly discolored, and open. One open fracture from 17.7 to 17.8 ft. (Devonian Granite Pluton) Core times: 5:20/ 4:25/ 4:15/ 2:35/ 3:05 min:sec/ft ROCK QUALITY = GOOD R2: Similar rock as R1 with coarse grained layer from 22.5 to 23.4 ft. One moderately dipping break; same as R1. 2:10/ 3:15/ 5:30/ 6:30/ 5:40 min:sec/ft ROCK QUALITY = GOOD</td><td rowspan="10">UCT qp = 17.28 ksi UCT qp = 29.41 ksi</td></tr><tr><td>5</td><td>2D</td><td>24/24</td><td>5.0 - 7.0</td><td>WOR/24"</td><td>--</td><td></td><td></td><td></td></tr><tr><td>10</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td></tr><tr><td>15</td><td>R1</td><td>60/60</td><td>15.2 - 20.2</td><td>RQD: 50%=83%</td><td></td><td></td><td>NQ2</td><td>-36.4</td></tr><tr><td>20</td><td>R2</td><td>60/60</td><td>20.3 - 25.3</td><td>RQD: 49%=82%</td><td></td><td></td><td></td><td></td></tr><tr><td>25</td><td></td><td></td><td></td><td>sample 22.8-23.4 sample 23.6-24.0</td><td></td><td></td><td></td><td></td></tr></table>										Depth (ft.)	Sample Information							Graphic Log	Visual Description and Remarks	Lab. Testing Results	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N-60	Casing Blows	Elevation (ft.)	0	1D	24/20	0.0 - 2.0	WOR/24"	3"dia		PUSH			1D: Dark grey, ORGANIC SILT, trace to little fine sand with shells; strong organic odor. Dark grey, fine to medium Sandy ORGANIC SILT; changing at 5.8 ft to: 2D: Dark grey, Silty CLAY. 13.6 ft: Unable to push HW past 13.6 ft. 14.7 ft: HW refusal. Roller cone to 15.1 ft. Spin NW to 15.2 ft. Top of bedrock at Elev. -36.4 ft. R1: Bedrock: Very hard, typically fresh, fine to medium grained, medium grey GRANITE with occasional feldspar phenocrysts. One high-angle break (along crack); undulating, rough, slightly discolored, and open. One open fracture from 17.7 to 17.8 ft. (Devonian Granite Pluton) Core times: 5:20/ 4:25/ 4:15/ 2:35/ 3:05 min:sec/ft ROCK QUALITY = GOOD R2: Similar rock as R1 with coarse grained layer from 22.5 to 23.4 ft. One moderately dipping break; same as R1. 2:10/ 3:15/ 5:30/ 6:30/ 5:40 min:sec/ft ROCK QUALITY = GOOD	UCT qp = 17.28 ksi UCT qp = 29.41 ksi	5	2D	24/24	5.0 - 7.0	WOR/24"	--				10									15	R1	60/60	15.2 - 20.2	RQD: 50%=83%			NQ2	-36.4	20	R2	60/60	20.3 - 25.3	RQD: 49%=82%					25				sample 22.8-23.4 sample 23.6-24.0				
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5	2D	24/24	5.0 - 7.0	WOR/24"	--																																																																																
10																																																																																					
15	R1	60/60	15.2 - 20.2	RQD: 50%=83%			NQ2	-36.4																																																																													
20	R2	60/60	20.3 - 25.3	RQD: 49%=82%																																																																																	
25				sample 22.8-23.4 sample 23.6-24.0																																																																																	
Remarks: 8/7/18; 0905 hrs: 8.3' top dolphin #1 to water; 7.7' barge deck to water; 37.5' barge deck to mudline top of dolphin #1: elev. 16.84 ft MLLW 47.5' off cleat of dolphin #1 to borehole; approx. on line through cleats on dolphins #1 and #2.																																																																																					
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.												Page 1 of 2																																																																									
* 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-VNHN-101-18																																																																									

 SCHONEWALD ENGINEERING ASSOCIATES, INC.		PROJECT: Fender System Modifications Maine State Ferry Terminal LOCATION: Vinalhaven, Maine		Boring No.: MB-VNHN-101-18 WIN: 23823.00																																																																																																																																																																																																																																
Driller: New England Boring Contractors		Elevation (ft.): -21.3 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: 8/7/18; 0830-1430		Drilling Method: cased wash boring		Hammer Type: auto hammer																																																																																																																																																																																																																																
Boring Location: per plan (B1A); see remarks		Casing ID/OD: HW to 14.7 ft/ NW to 15.2 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																																																																																																																																																																																																																																				
<table border="1"> <thead> <tr> <th rowspan="2">Depth (ft.)</th> <th colspan="8">Sample Information</th> <th rowspan="2">Graphic Log</th> <th rowspan="2">Visual Description and Remarks</th> <th rowspan="2">Lab. Testing Results</th> </tr> <tr> <th>Sample No.</th> <th>Pen./Rec. (in.)</th> <th>Sample Depth (ft.)</th> <th>Blows (6 in.) Shear Strength (psf) or RQD (%)</th> <th>N-uncorrected</th> <th>N-60</th> <th>Casing</th> <th>Blows</th> <th>Elevation (ft.)</th> </tr> </thead> <tbody> <tr> <td>25</td> <td>R3</td> <td>60/58</td> <td>25.3 - 30.3</td> <td>RQD: 56"=93%</td> <td></td> <td></td> <td></td> <td></td> <td></td> <td rowspan="12"> </td> <td rowspan="12"></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> </tr> <tr> <td>30</td> <td>R4</td> <td>60/60</td> <td>30.3 - 35.3</td> <td>RQD: 36"=60%</td> <td></td> <td></td> <td></td> <td></td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> </tr> <tr> <td></td> <td>R4A</td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> </tr> <tr> <td>35</td> <td>R5</td> <td>60/60</td> <td>35.3 - 40.3</td> <td>RQD: 36"=60%</td> <td></td> <td></td> <td></td> <td></td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> </tr> <tr> <td>40</td> <td>R6</td> <td>60/60</td> <td>40.3 - 45.3</td> <td>RQD: 54"=90%</td> <td></td> <td></td> <td></td> <td></td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> </tr> <tr> <td></td> <td>R6A</td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> </tr> <tr> <td>45</td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> </tr> <tr> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> </tr> <tr> <td>50</td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> <td></td> </tr> </tbody> </table>						Depth (ft.)	Sample Information								Graphic Log	Visual Description and Remarks	Lab. Testing Results	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N-60	Casing	Blows	Elevation (ft.)	25	R3	60/58	25.3 - 30.3	RQD: 56"=93%																																						30	R4	60/60	30.3 - 35.3	RQD: 36"=60%																	R4A																													35	R5	60/60	35.3 - 40.3	RQD: 36"=60%																										40	R6	60/60	40.3 - 45.3	RQD: 54"=90%																	R6A									45																																								50									
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APPENDIX B

ROCK CORE PHOTOGRAPHS



Photo 1: Core box containing dried rock core from test borings MB-VNHN-101; left side of core box (top portion of cores).

Slots from top to bottom:

- 1) MB-VNHN-101, R1;
- 2) MB-VNHN-101, R2;
- 3) MB-VNHN-101, R3;
- 4) Empty.



Photo 2: Core box containing dried rock core from test boring MB-VNHN-101; right side of core box (bottom portion of cores).

Slots from top to bottom:

- 1) MB-VNHN-101, R1;
- 2) MB-VNHN-101, R2;
- 3) MB-VNHN-101, R3;
- 4) Empty.



Photo 3: Core box containing dried rock core from test borings MB-VNHN-101; left side of core box (top portion of cores).

Slots from top to bottom:

- 1) MB-VNHN-101, R4;
- 2) MB-VNHN-101, R5;
- 3) MB-VNHN-101, R6;
- 4) Empty.



Photo 4: Core box containing dried rock core from test boring MB-VNHN-101; right side of core box (bottom portion of cores).

Slots from top to bottom:

- 1) MB-VNHN-101, R4;
- 2) MB-VNHN-101, R5;
- 3) MB-VNHN-101, R6;
- 4) Empty.



SCHONEWALD
ENGINEERING
ASSOCIATES, INC.

ROCK CORE PHOTOGRAPHS
FENDER SYSTEM MODIFICATIONS
MAINE STATE FERRY TERMINAL
VINALHAVEN, MAINE
MAINEDOT WIN 23823.00

Sheet No.:



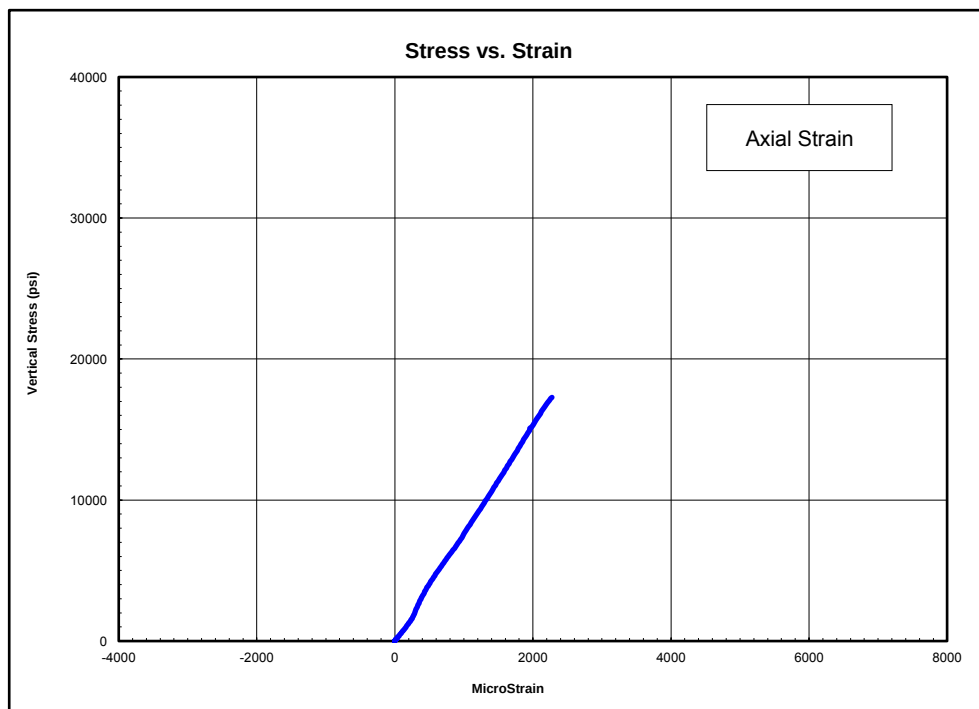
APPENDIX C

LABORATORY TEST REPORTS



Client:	Schonewald Engineering Associates, Inc.
Project Name:	MeDOT WIN 23823.00 MSFT Vinalhaven, ME
Project Location:	Vinalhaven, ME
GTX #:	308808
Test Date:	9/20/2018
Tested By:	tlm
Checked By:	jsc
Boring ID:	MB-VNHN-101
Sample ID:	R-2
Depth, ft:	22.8-23.4
Sample Type:	rock core
Sample Description:	See photographs Intact material failure

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



Peak Compressive Stress: 17,280 psi

Both of the lateral strain gauge failed to record meaningful data. Poisson's Ratio could not be determined.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1700-6300	8,220,000	---
6300-10900	7,490,000	---
10900-15500	7,910,000	---

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

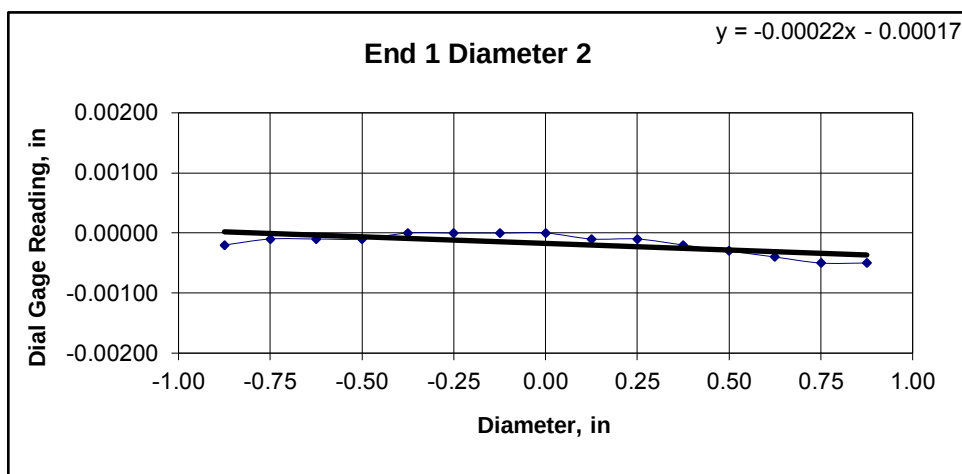
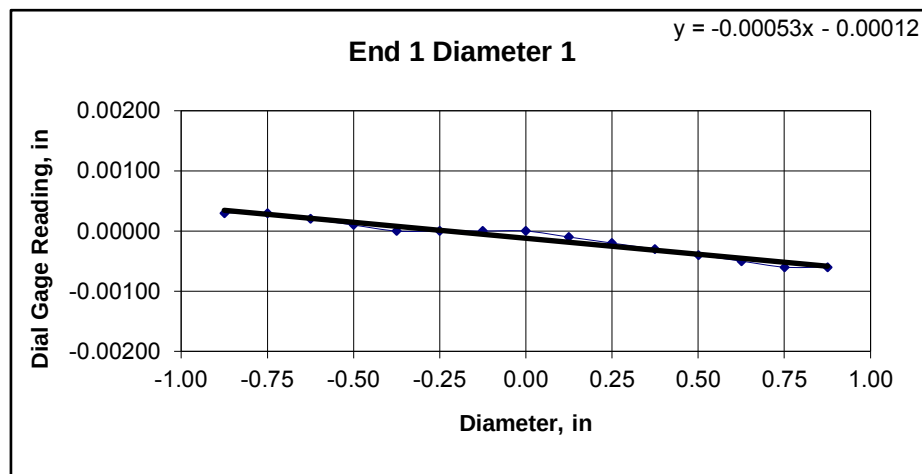


Client:	Schonewald Engineering Associates, Inc.	Test Date:	9/20/2018
Project Name:	MeDOT WIN 23823.00 MSFT Vinalhaven, ME	Tested By:	tlm
Project Location:	Vinalhaven, ME	Checked By:	jsc
GTX #:	308808		
Boring ID:	MB-VNHN-101		
Sample ID:	R-2		
Depth:	22.8-23.4 ft		
Visual Description:	See photographs		

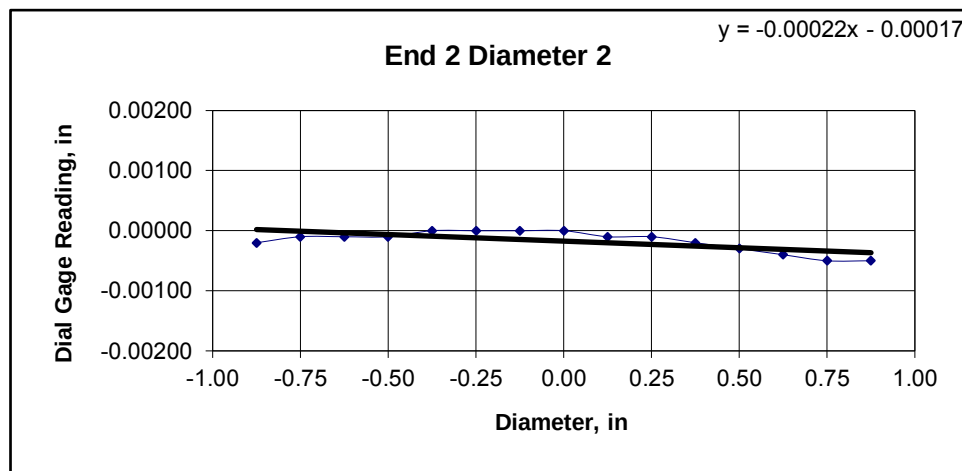
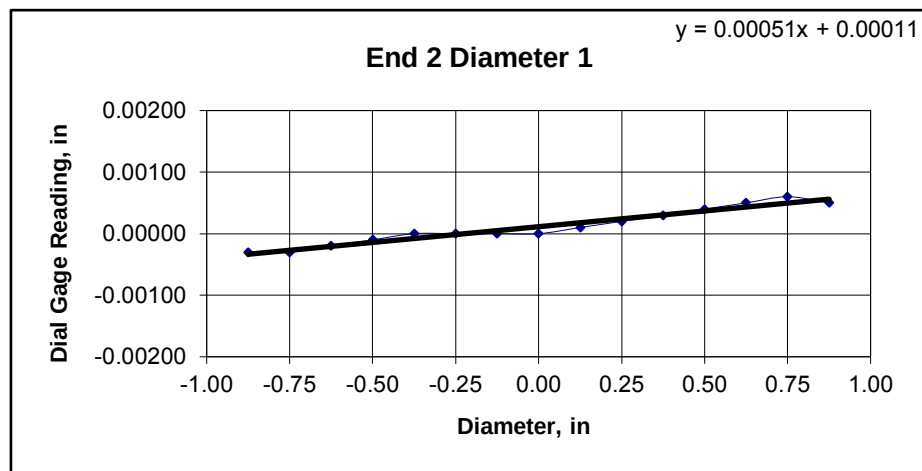
UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

BULK DENSITY				DEVIATION FROM STRAIGHTNESS (Procedure S1)	
	1	2	Average	Maximum gap between side of core and reference surface plate:	
Specimen Length, in:	4.29	4.29	4.29	Is the maximum gap $\leq$ 0.02 in.?	
Specimen Diameter, in:	1.99	1.99	1.99	YES	
Specimen Mass, g:	577.94			Maximum difference must be < 0.020 in.	
Bulk Density, lb/ft ³	165			Straightness Tolerance Met?	
Length to Diameter Ratio:	2.2	Minimum Diameter Tolerance Met?	YES	YES	
		Length to Diameter Ratio 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	-0.00060
Diameter 2, in (rotated 90°)	-0.00020	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00020	-0.00030	-0.00040	-0.00050	-0.00050
Difference between max and min readings, in:															
0° = 0.00090 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	-0.00030	-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	0.00050
Diameter 2, in (rotated 90°)	-0.00020	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00020	-0.00030	-0.00040	-0.00050	-0.00050
Difference between max and min readings, in:															
0° = 0.0009 90° = 0.0005															
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00045															
Flatness Tolerance Met? YES															



DIAMETER 1	
End 1:	
Slope of Best Fit Line	0.00053
Angle of Best Fit Line:	0.03045
End 2:	
Slope of Best Fit Line	0.00051
Angle of Best Fit Line:	0.02930
Maximum Angular Difference:	0.00115
Parallelism Tolerance Met?	YES
Spherically Seated	



DIAMETER 2	
End 1:	
Slope of Best Fit Line	0.00022
Angle of Best Fit Line:	0.01261
End 2:	
Slope of Best Fit Line	0.00022
Angle of Best Fit Line:	0.01261
Maximum Angular Difference:	0.00000
Parallelism Tolerance Met?	YES
Spherically Seated	

PERPENDICULARITY (Procedure P1)						Maximum angle of departure must be $\leq$ 0.25°	
(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	0.00090	1.990	0.00045	0.026	YES		
Diameter 2, in (rotated 90°)	0.00050	1.990	0.00025	0.014	YES	Perpendicularity Tolerance Met?	YES
END 2							
Diameter 1, in	0.00090	1.990	0.00045	0.026	YES		
Diameter 2, in (rotated 90°)	0.00050	1.990	0.00025	0.014	YES		

Client:	Schonewald Engineering Associates, Inc.
Project Name:	MeDOT WIN 23823.00 MSFT Vinalhaven, ME
Project Location:	Vinalhaven, ME
GTX #:	308808
Test Date:	9/21/2018
Tested By:	tlm
Checked By:	jsc
Boring ID:	MB-VNHN-101
Sample ID:	R2
Depth, ft:	22.8-23.4



After cutting and grinding

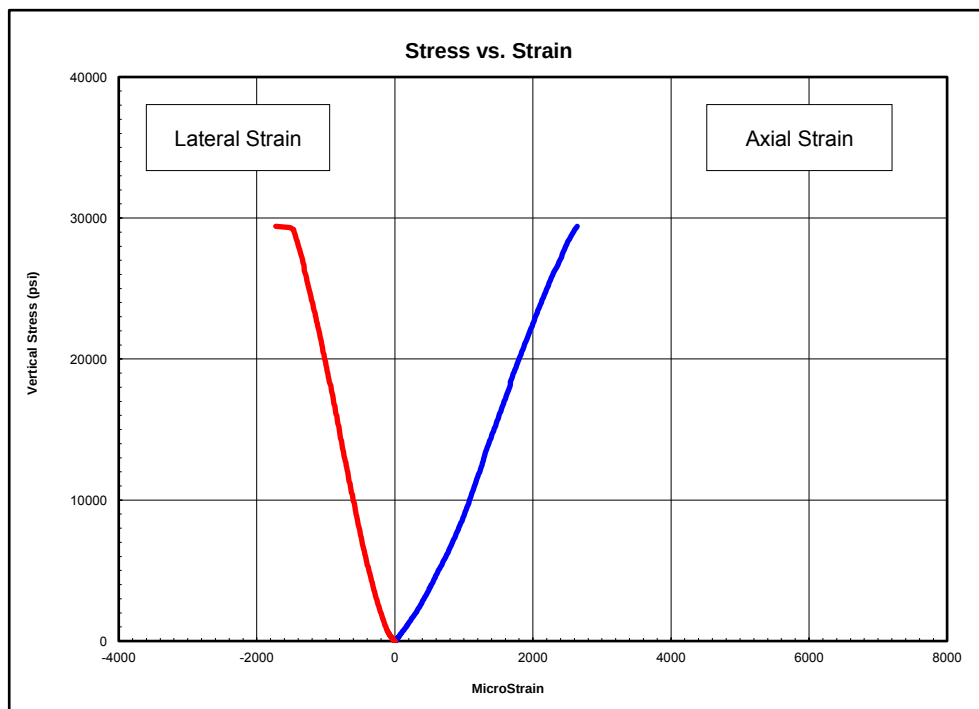


After break



Client:	Schonewald Engineering Associates, Inc.
Project Name:	MeDOT WIN 23823.00 MSFT Vinalhaven, ME
Project Location:	Vinalhaven, ME
GTX #:	308808
Test Date:	9/20/2018
Tested By:	tlm
Checked By:	jsc
Boring ID:	MB-VNHN-101
Sample ID:	R-2
Depth, ft:	23.6-24.0
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: 29,413 psi

The strain values recorded for this test produce values of Poisson's Ratio that exceed maximum values found in rocks.

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
2900-10800	10,600,000	---
10800-18600	13,900,000	---
18600-26500	12,300,000	---

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

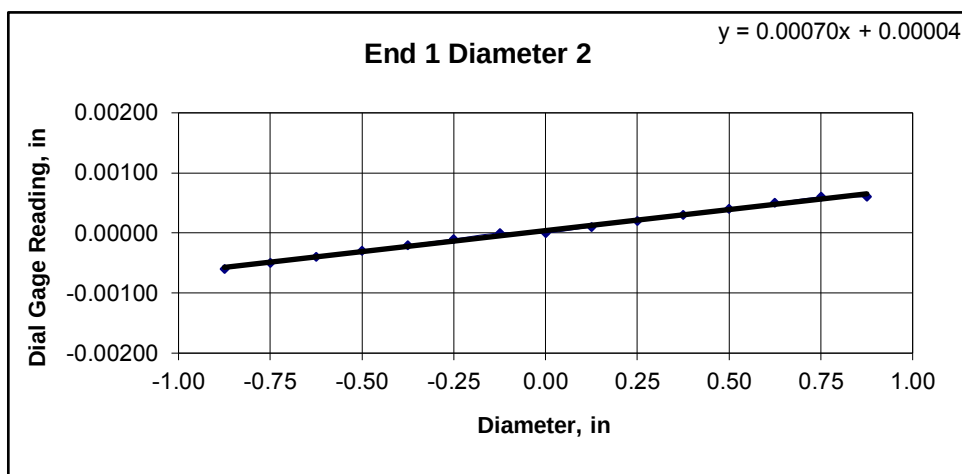
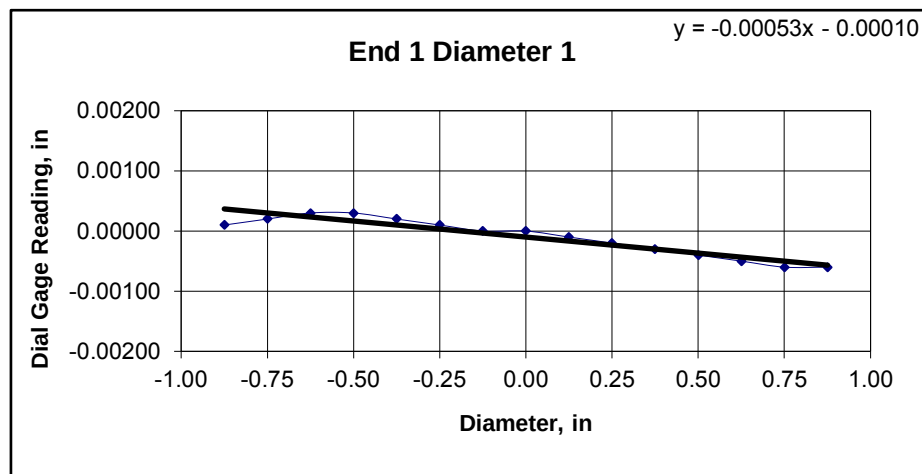


Client:	Schonewald Engineering Associates, Inc.	Test Date:	9/20/2018
Project Name:	MeDOT WIN 23823.00 MSFT Vinalhaven, ME	Tested By:	tlm
Project Location:	Vinalhaven, ME	Checked By:	jsc
GTX #:	308808		
Boring ID:	MB-VNHN-101		
Sample ID:	R-2		
Depth:	23.6-24.0 ft		
Visual Description:	See photographs		

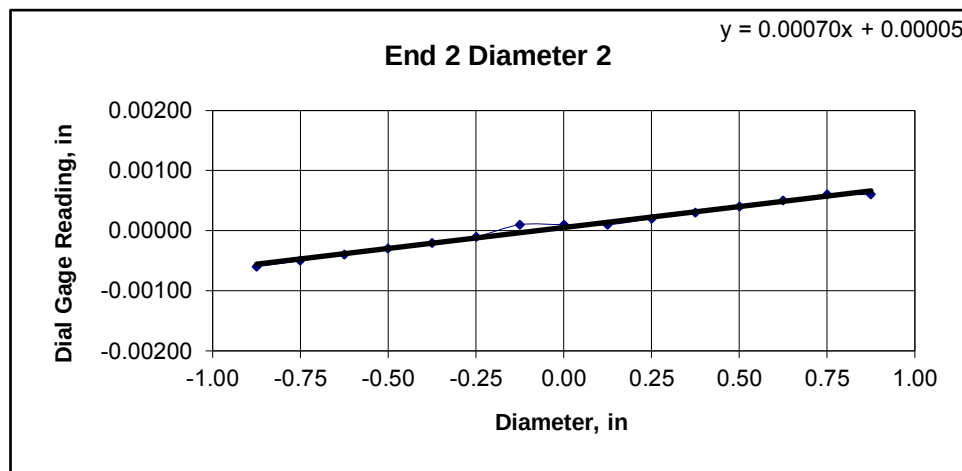
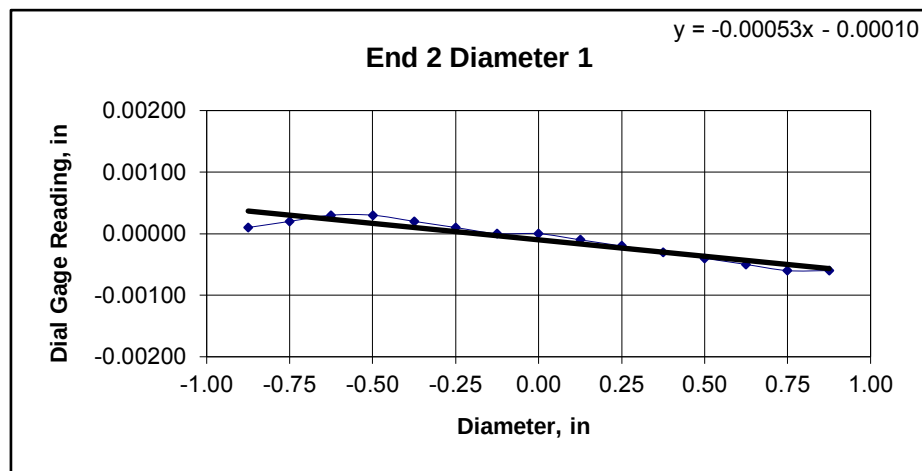
UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

BULK DENSITY				DEVIATION FROM STRAIGHTNESS (Procedure S1)	
	1	2	Average	Maximum gap between side of core and reference surface plate:	
Specimen Length, in:	4.37	4.36	4.37	Is the maximum gap $\leq$ 0.02 in.?	
Specimen Diameter, in:	1.99	1.99	1.99	YES	
Specimen Mass, g:	593.14			Maximum difference must be < 0.020 in.	
Bulk Density, lb/ft ³	166			Straightness Tolerance Met?	
Length to Diameter Ratio:	2.2			YES	
		Minimum Diameter Tolerance Met?	YES		
		Length to Diameter Ratio Tolerance Met?	YES		

END FLATNESS AND PARALLELISM (Procedure FP1)															
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00010	0.00020	0.00030	0.00030	0.00020	0.00010	0.00000	0.00000	-0.00010	-0.00020	-0.00030	-0.00040	-0.00050	-0.00060	-0.00060
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	0.00060
Difference between max and min readings, in:															
0° = 0.00090 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	0.00010	0.00020	0.00030	0.00030	0.00020	0.00010	0.00000	0.00000	-0.00010	-0.00020	-0.00030	-0.00040	-0.00050	-0.00060	-0.00060
Diameter 2, in (rotated 90°)	-0.00060	-0.00050	-0.00040	-0.00030	-0.00020	-0.00010	0.00010	0.00010	0.00010	0.00020	0.00030	0.00040	0.00050	0.00060	0.00060
Difference between max and min readings, in:															
0° = 0.0009 90° = 0.0012															
Maximum difference must be < 0.0020 in. Difference = $\pm$ 0.00060															
Flatness Tolerance Met?															
YES															



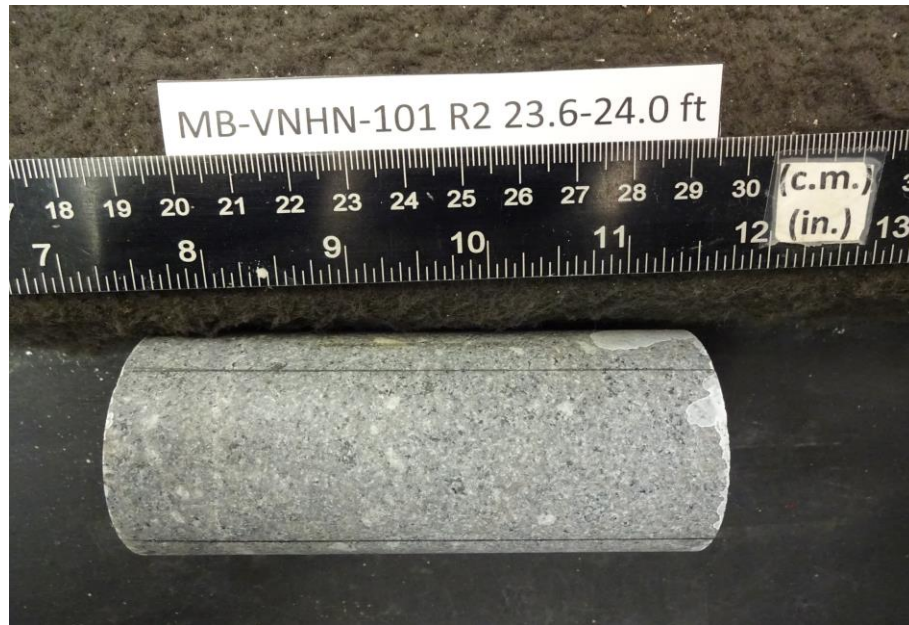
DIAMETER 1	
End 1:	
Slope of Best Fit Line	0.00053
Angle of Best Fit Line:	0.03061
End 2:	
Slope of Best Fit Line	0.00053
Angle of Best Fit Line:	0.03061
Maximum Angular Difference:	0.00000
Parallelism Tolerance Met?	YES
Spherically Seated	



DIAMETER 2	
End 1:	
Slope of Best Fit Line	0.00070
Angle of Best Fit Line:	0.04011
End 2:	
Slope of Best Fit Line	0.00070
Angle of Best Fit Line:	0.03994
Maximum Angular Difference:	0.00016
Parallelism Tolerance Met?	YES
Spherically Seated	

PERPENDICULARITY (Procedure P1)						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	0.00090	1.990	0.00045	0.026	YES		
Diameter 2, in (rotated 90°)	0.00120	1.990	0.00060	0.035	YES	Perpendicularity Tolerance Met?	YES
END 2							
Diameter 1, in	0.00090	1.990	0.00045	0.026	YES		
Diameter 2, in (rotated 90°)	0.00120	1.990	0.00060	0.035	YES		

Client:	Schonewald Engineering Associates, Inc.
Project Name:	MeDOT WIN 23823.00 MSFT Vinalhaven, ME
Project Location:	Vinalhaven, ME
GTX #:	308808
Test Date:	9/21/2018
Tested By:	tlm
Checked By:	jsc
Boring ID:	MB-VNHN-101
Sample ID:	R2
Depth, ft:	23.6-24.0



After cutting and grinding



After break



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), average peak uniaxial compressive strength = 15.6 ksi = 15,571 psi		
12.6			
7.9			
6.7	minimum peak uniaxial compressive strength =	6.7 ksi =	6,700 psi
16.8			
7.4	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

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.

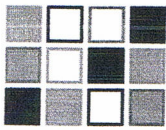
LABORATORY TESTS: results of laboratory uniaxial compressive strength tests on the METAPELITE encountered in Islesboro are as follows:

UCT q_p (ksi)			
11.4			
8.2			
8.5			
4.0	average peak uniaxial compressive strength =	6.5 ksi =	6,467 psi
6.9			
8.5	throw out upper and lower results (strikethroughs), average peak uniaxial compressive strength = 7.1 ksi = 7,122 psi		
7.0			
7.6			
6.1	minimum peak uniaxial compressive strength =	1.0 ksi =	1,000 psi
6.1			
4.4	maximum peak uniaxial compressive strength =	11.1 ksi =	11,100 psi
5.2			

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)



Client HNTB
Project MEDOT MSFT FENDER SYSTEM MODIFICATIONS
Location VINHANG, FRENCHBORO, BASS HBR, SWANS IS,
Subject ROCK QUALITY AND COMPRESSIVE STRENGTH SUMMARY

File No. 18-010 - 18-014
Prepared FEB 2019 By IUS
Checked By
Revised By

VINALMAVEN MB-VINAL-101 EL -36.4 FT	R1 83%	R2 82% UCT _{gp} = 17.3 KSI 20.4 KSI	R3 93%	R4 60%	R5 60%	R6 90%	GRANITE (GABRO)		
BASS HBR MB-BASS-101 EL -62.0 FT	R1 100%	R2 58% UCT _{gp} —	R3 78% UCT _{gp} = 15.9 KSI 14.1 KSI	R4 22%	R5 26%	R6 33%	R7 55%	R8 53%	BASALT
SWANS IS MB-SWAN-101 EL -20.2 FT	R1 25% UCT _{gp} = 16.4 KSI	R2 60% UCT _{gp} = 15.9 KSI	R3 80%	R4 75%	R5 57%	R6 26%	BASALT		
SWANS IS MB-SWAN-102 EL -40.1 FT	R1 35%	R2 50% UCT _{gp} = 12.6 KSI	R3 44% UCT _{gp} = 7.9 KSI 6.7 KSI	R4 77%	R5 62%	R6 47%	R7 97%	BASALT	
FRENCHBORO MB-FBORO-101 EL -54.2 FT	R1 63% UCT _{gp} = 16.8 KSI	R2 73%	R3 80%	R4 82% UCT _{gp} = 7.11 KSI	GRANITE				
FRENCHBORO MB-FBORO-102 EL -47.4 FT	R1 33% UCT _{gp} = 26.2 KSI	R2 62% UCT _{gp} = 27.5 KSI	R3 64%	R4 93%	R5 83%	R6 87%	GRANITE		

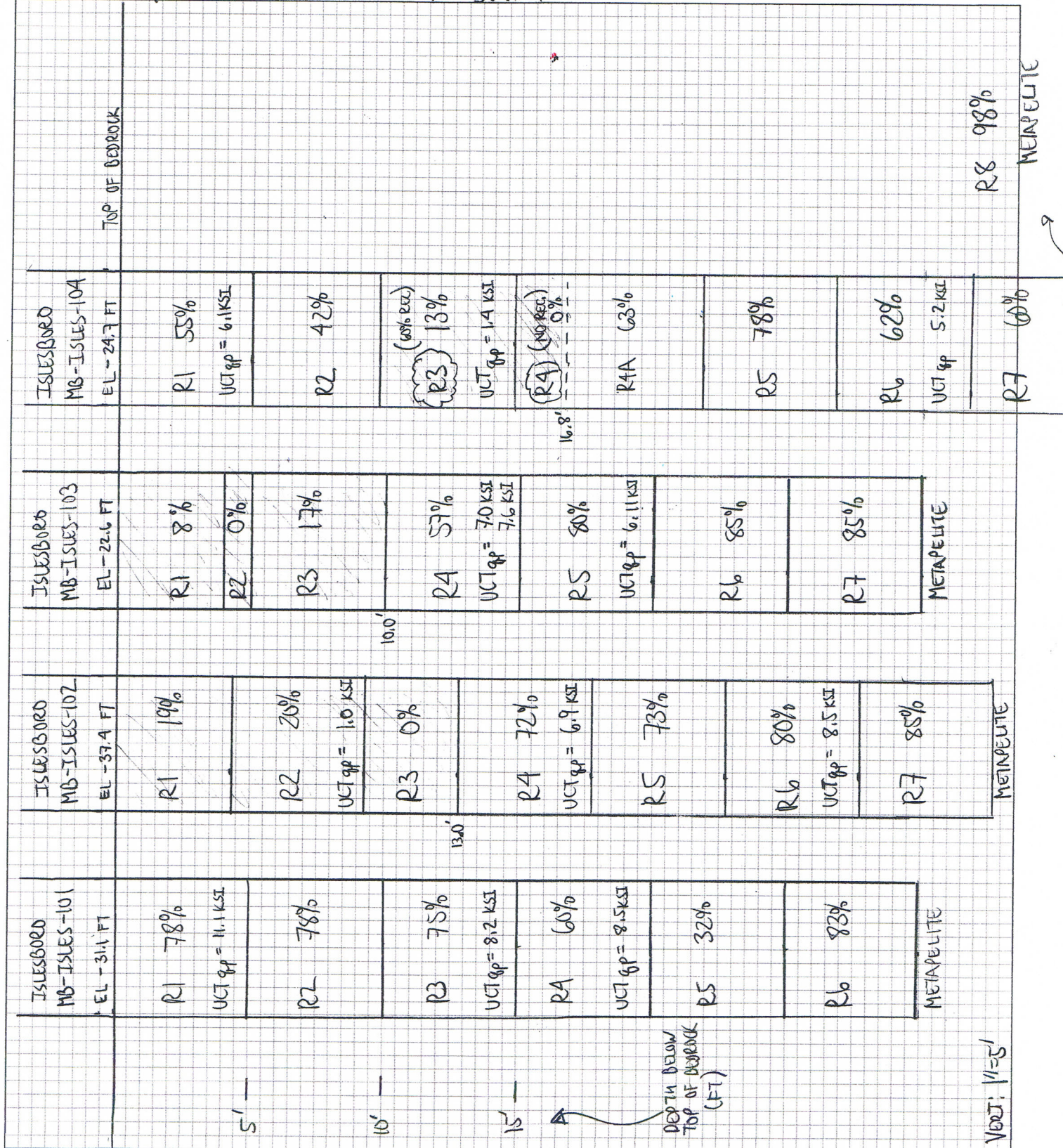
TOP OF BEDROCK

DEPTH BELOW TOP OF BEDROCK

VERT: 1" = 5'

Client HMB
Project MBOT MSFT FENDER SYSTEM MODIFICATIONS
Location ISLESBORO
Subject ROCK QUALITY AND COMPRESSIVE STRENGTH SUMMARY

File No. 18-010 - 18-014
Prepared Feb 2019 By IUS
Checked By
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:

$$Q_{ult} = Q_{avail} * FS$$

$$Q_{ult} = (UCT_{qp}) * (\pi(B/2)^2) \quad \text{from Wyllie Eq 8.9 and conservatively assuming a FS=2}$$

where: Q_{ult} = 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)

$$Q_{ult} = (UCT_{qp}) * (\pi(B/2)^2)$$

$$Q_{ult} = (15.6 \text{ ksi}) * [(\pi)(8 \text{ in})^2]$$

$$Q_{ult} = 3,136 \text{ kips}$$

$$Q_{avail} = Q_{ult}/FS = Q_{ult} * \phi_{stat}$$

where: FS = factor of safety

ϕ_{stat} = resistance factor (tip resistance on rock)

$$3,136 \text{ kips} * 0.5$$

$$1,568 \text{ kips}$$

per HNTB TD design: $Q_{req'd} = 345$ kips

$Q_{avail} = 1,568 \text{ kips} \gg Q_{req'd} = 345 \text{ kips}$

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)

$$Q_{ult} = (UCT_{qp}) * (\pi(B/2)^2)$$

$$Q_{ult} = (7.1 \text{ ksi}) * [(\pi)(8 \text{ in})^2]$$

$$Q_{ult} = 1,427 \text{ kips}$$

$$Q_{avail} = Q_{ult}/FS = Q_{ult} * \phi_{stat}$$

$$1,427 \text{ kips} * 0.5$$

$$714 \text{ kips}$$

per HNTB MD design: $Q_{req'd} = 163$ kips

$Q_{avail} = 714 \text{ kips} \gg Q_{req'd} = 163 \text{ kips}$

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:

$$Q_{avail} = Q_{ult} / FS$$

$$Q_{avail} = (Lb)(\pi)(d)(Ta) \quad \text{from Wyllie Eq 9.8, which includes a } FS=3 \text{ through the } Ta \text{ term}$$

where: Q_{avail} = 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

Ta = working bond strength of rock-grout interface

$Ta \approx (UCT \text{ qp})/30$ 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 $UCT \text{ qp} = 15.6 \text{ ksi}$ (refer to rock quality and compressive strength calcs)

$$Ta \approx (UCT \text{ qp})/30 = (15,600 \text{ psi})/30 = 520 \text{ psi} \quad 520 \text{ psi} > 203 \text{ psi, use } Ta = 203 \text{ psi}$$

$$Q_{avail} = (Lb)(\pi)(d)(Ta)$$

$$Q_{avail} = (15 \text{ ft}) * (\pi)(0.375 \text{ ft}) [(203 \text{ lb/in}^2)(144 \text{ in}^2/\text{ft}^2)]$$

$$Q_{avail} = 516,557 \text{ lbs}$$

$$Q_{avail} = 517 \text{ kips}$$

per HNTB TD design: $Q_{req'd} = 112 \text{ kips}$ $Q_{avail} = 517 \text{ kips} >> Q_{req'd} = 112 \text{ kips}$ **OKAY**

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

$$Ta \approx (UCT \text{ qp})/30 = (7,100 \text{ psi})/30 = 237 \text{ psi} \quad 237 \text{ psi} > 203 \text{ psi, use } Ta = 203 \text{ psi}$$

$$Q_{avail} = (Lb)(\pi)(d)(Ta)$$

$$Q_{avail} = (15 \text{ ft}) * (\pi)(0.375 \text{ ft}) [(203 \text{ lb/in}^2)(144 \text{ in}^2/\text{ft}^2)]$$

$$Q_{avail} = 516,557 \text{ lbs}$$

$$Q_{avail} = 517 \text{ kips}$$

per HNTB MD design: $Q_{req'd} = 49 \text{ kips}$ $Q_{avail} = 517 \text{ kips} >> Q_{req'd} = 49 \text{ kips}$ **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, R_s , 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 $\phi_{up} = 0.35$ and after the alpha method is $\phi_{up} = 0.25$.

$$R_R = R_s \phi_{up}$$

where: R_R = factored uplift resistance

ϕ_{up} = uplift resistance factor (see above discussion)

R_s = nominal side resistance

$$R_s = q_s A_s$$

where: q_s = unit side resistance of pile

A_s = surface area of pile = circumference * L = $C * L$

C (timber pile) = $2\pi r = 2\pi(9 \text{ in}/2) = 2.36 \text{ ft}$

Analysis - Norlund method for cohesionless soils:

$$q_s = K \delta C_f \phi' v [(\sin(\delta + \omega)) / \cos \omega]$$

from: LRFD Manual Eq 10.7.3.8.6f-1 with ϕ_f taken as 30 deg.

where: $K \delta$ = coef. of lateral earth pressure at mid-point of layer under consideration; Fig 10.7.3.8.6f-2

C_f = correction factor for $K \delta$; Fig 10.7.3.8.6f-5

$\phi' v$ = 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: ϕ_f taken as 30 deg.

function of pile: $K \delta = 2.55$ from Fig 10.7.3.8.6f-2 with $\omega = 0.47$ deg

$\delta = 16.5$ from Fig 10.7.3.8.6f-6 for timber pile and $Vol = 0.55 \text{ cf/ft}$

$C_f = 0.8$ from Fig 10.7.3.8.6f-5 with $\phi_f = 30$ deg and $\delta = 16.5$ deg

Analysis - alpha method for cohesive soils:

$$q_s = (S_u)(\alpha)$$

where: S_u = average undrained shear strength of soil

taken as 400 psf based on field observation and experience

α = function of S_u ; taken from LRFD Fig 10.7.3.8.6b-1

with $D = 9 \text{ in} = 0.75 \text{ ft}$ and D_b from soil profile

where D_b 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 subsurface 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

$$\phi'v = (115 \text{ pcf} - 62.4 \text{ pcf})(4 \text{ ft}) = 210 \text{ psf (at midpoint of layer)}$$

$$qs = K\delta Cf \phi'v [(\sin(\delta+\omega)) / \cos \omega]$$

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

$$qs = 125 \text{ psf}$$

$$Rs = qs \cdot As$$

$$Rs = 125 \text{ psf} \cdot 4 \text{ ft} \cdot 2.36 \text{ ft}$$

$$Rs = 1,180 \text{ lbs}$$

$$Rs = 1.2 \text{ kips}$$

$$R_R = R_s \phi_{up}$$

$$R_R = 1.2 \text{ kips} \cdot 0.35$$

$$R_R = 0.4 \text{ kips}$$

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

$$\phi'v = (115 \text{ pcf} - 62.4 \text{ pcf})(7.5 \text{ ft}) = 395 \text{ psf (at midpoint of layer)}$$

$$qs = K\delta Cf \phi'v [(\sin(\delta+\omega)) / \cos \omega]$$

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

$$qs = 235 \text{ psf}$$

$$Rs = qs \cdot As$$

$$Rs = 235 \text{ psf} \cdot 11 \text{ ft} \cdot 2.36 \text{ ft}$$

$$Rs = 6,101 \text{ lbs}$$

$$Rs = 6.1 \text{ kips}$$

$$R_R = R_s \phi_{up}$$

$$R_R = 6.1 \text{ kips} \cdot 0.35$$

$$R_R = 2.1 \text{ kips}$$

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 $S_u = 400$ psf

$q_s = (S_u)(\alpha)$ with α taken from LRFD Fig 10.7.3.8.6b-1 with $D = 0.75$ ft and $D_b = 10$ ft
and $D_b/D = 13.3$
 $\alpha = 0.6$ (from center fig)

$$q_s = (400)(0.6)$$

$$q_s = 240 \text{ psf}$$

$$R_s = q_s \cdot A_s$$

$$R_s = 240 \text{ psf} \cdot 10 \text{ ft} \cdot 2.36 \text{ ft}$$

$$R_s = 5,664 \text{ lbs}$$

$$R_s = 5.7 \text{ kips}$$

$$R_R = R_s \phi_{up}$$

$$R_R = 5.7 \text{ kips} \cdot 0.25$$

$$R_R = 1.4 \text{ kips}$$

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 $S_u = 400$ psf

$q_s = (S_u)(\alpha)$ with α taken from LRFD Fig 10.7.3.8.6b-1 with $D = 0.75$ ft and $D_b = 21.5$ ft
and $D_b/D = 28.7$
 $\alpha = 0.95$ (from center fig)

$$q_s = (400)(0.95)$$

$$q_s = 380 \text{ psf}$$

$$R_s = q_s \cdot A_s$$

$$R_s = 380 \text{ psf} \cdot 21.5 \text{ ft} \cdot 2.36 \text{ ft}$$

$$R_s = 19,281 \text{ lbs}$$

$$R_s = 19.2 \text{ kips}$$

$$R_R = R_s \phi_{up}$$

$$R_R = 19.2 \text{ kips} \cdot 0.25$$

$$R_R = 4.8 \text{ kips}$$

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

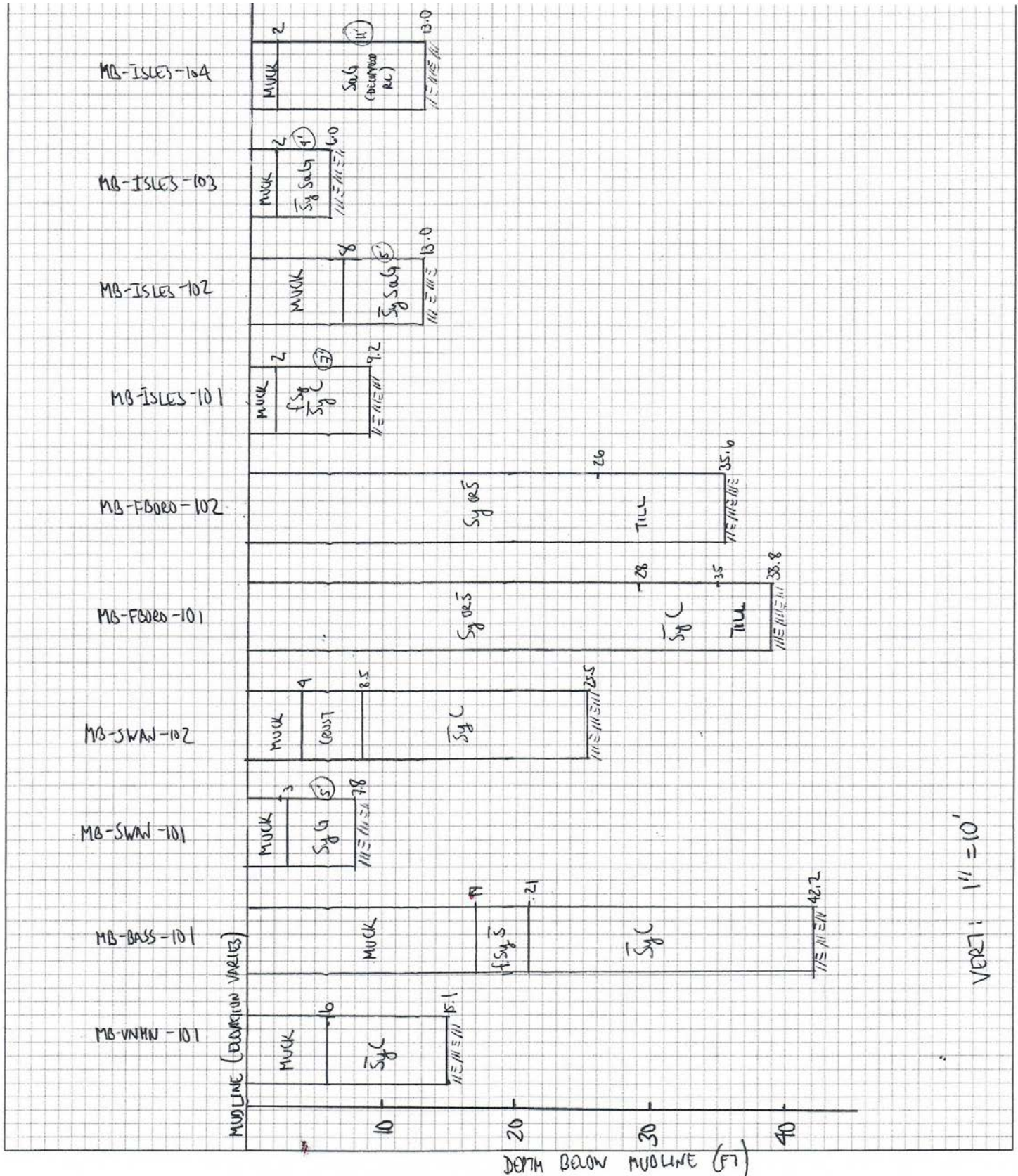
Location: Vinalhaven, Frenchboro, Bass Hbr, Swans Is, Islesboro

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



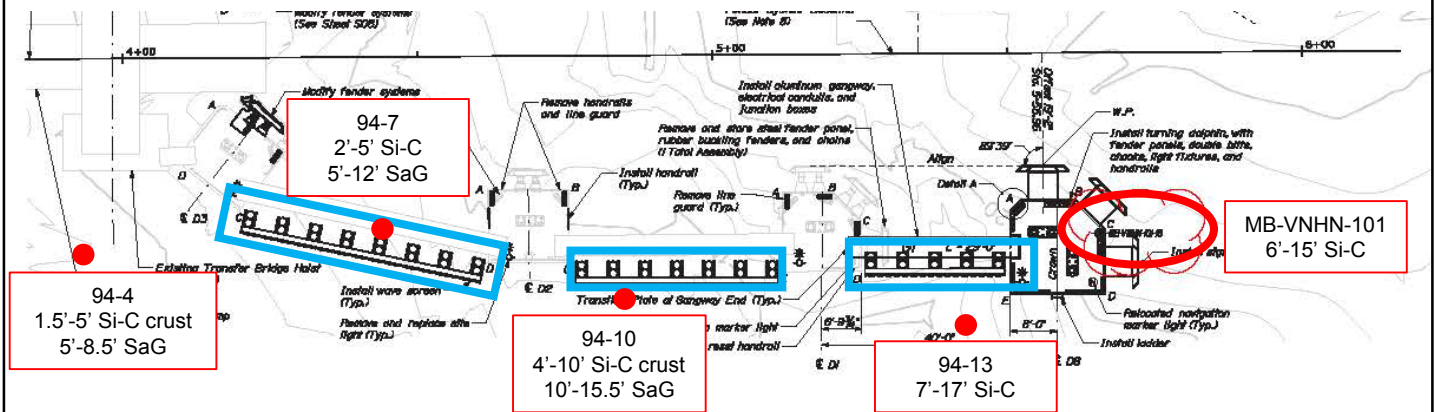
Recent Boring



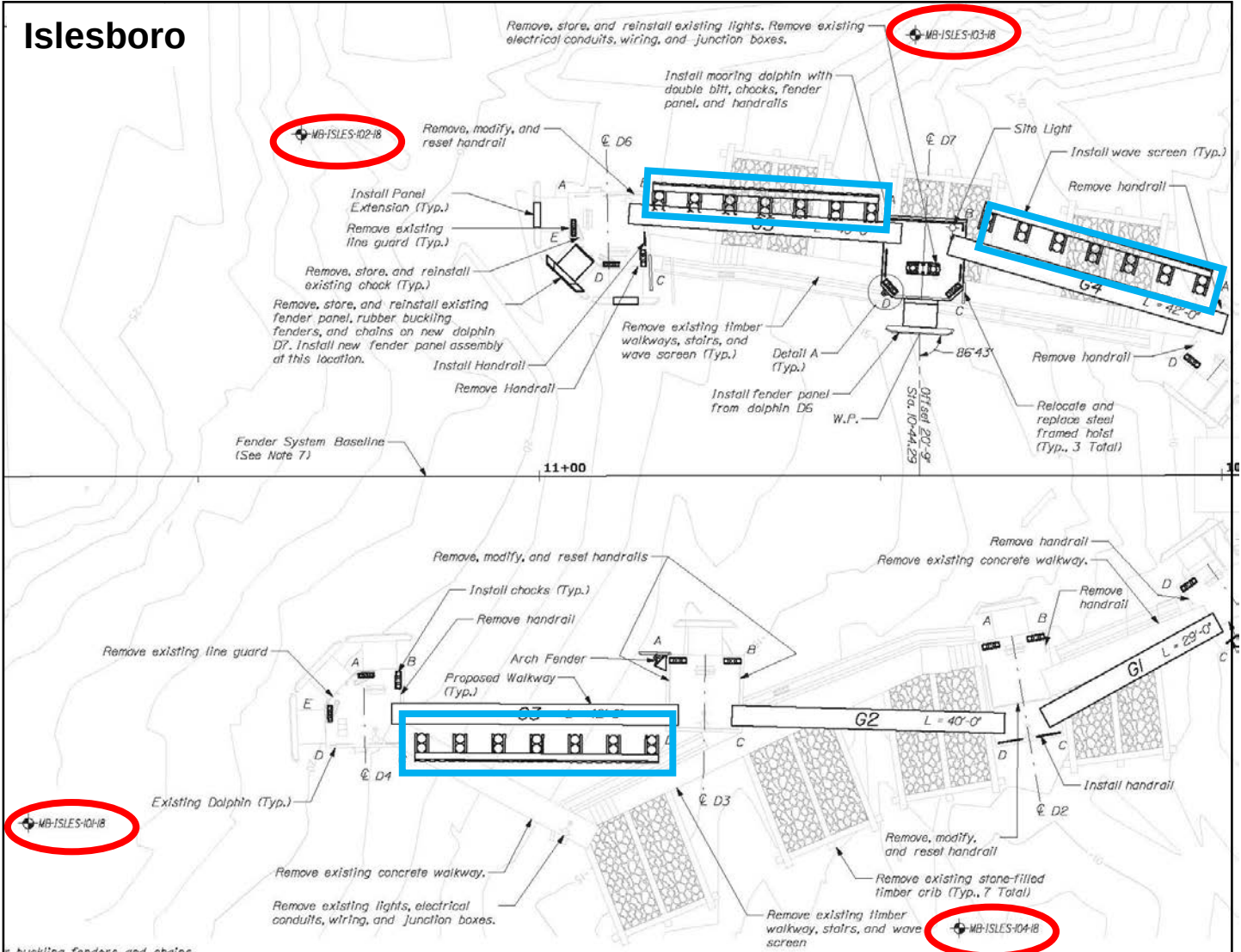
94-13
7'-17' Si-C

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

