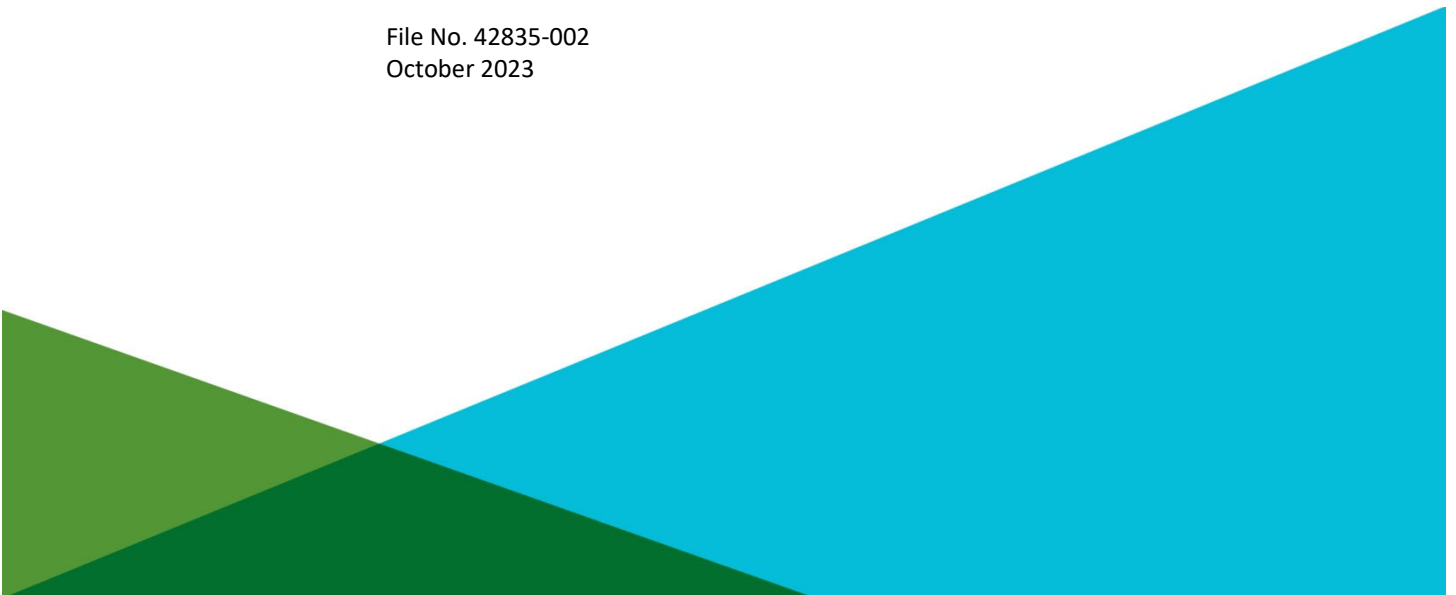


GEOTECHNICAL DESIGN REPORT
OCEANVILLE BRIDGE OVER OCEANVILLE THOROUGHFARE,
BRIDGE NO. 3696
MAINEDOT WIN 22630.00
STONINGTON, MAINE

by Haley & Aldrich, Inc.
Portland, Maine

for Fuss & O'Neill, Inc.
Manchester, New Hampshire

File No. 42835-002
October 2023





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File No. 42835-002

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Attention: Jaime French, P.E.
Associate | Transportation Department Manager

Subject: Geotechnical Design Report
Oceanville Bridge over Oceanville Thoroughfare, Bridge No. 3696
MaineDOT WIN 22630.00
Stonington, Maine

Ladies and Gentlemen:

We are pleased to submit our report entitled, "Geotechnical Design Report, Oceanville Bridge over Oceanville Thoroughfare, Bridge No. 3696, MaineDOT WIN 22630.00, Stonington, Maine." This Geotechnical Design Report (GDR) has been prepared in accordance with our proposal, dated 17 September 2021, revised 1 November 2021, and executed by your JoAnn Fryer on 14 April 2022.

Introduction

This GDR presents the results of preliminary design (Phase I) and final design (Phase II) phase subsurface investigation and laboratory testing programs, technical evaluations completed by Haley & Aldrich, Inc. (Haley & Aldrich), and provides geotechnical design recommendations for the proposed bridge that will carry vehicular traffic on the proposed Oceanville Road over Oceanville Thoroughfare in Stonington, Maine (see Figure 1, Project Locus).

HORIZONTAL COORDINATE SYSTEM, ELEVATION DATUM, AND BASELINE STATIONING

Plan locations of test borings are reported as northing and easting coordinates relative to the Maine State Plane Coordinate System, North American Datum of 1983 (NAD 83), Maine 2000 Central Zone. The project elevation datum and elevations referenced herein are in feet and reference the North American Vertical Datum of 1988 (NAVD 88).

PROJECT LOCATION AND EXISTING SITE CONDITIONS

The existing bridge carries Oceanville Road (two lanes) over the Oceanville Thoroughfare connecting the Whitmore Neck and Buckmaster Neck in the town of Stonington, Maine, as shown in Figure 1. Based on our review of the historic bridge plans dated 1939 (refer to Appendix C) and reports provided by Fuss &

O'Neill, the existing single-span bridge structure was constructed in 1940 with a length of 39 ft and width of 23 ft. The bridge is supported on approximately 15-ft tall abutments consisting of stacked granite blocks which appear to be resting on or near bedrock. At one time there were two additional spans to the south of the existing bridge, but they were filled in as part of the 1940 construction. There are two armored, causeway approaches to the bridge structure which extend into the channel approximately 125 ft to the south of the bridge and 75 ft to the north of the bridge.

PROPOSED BRIDGE STRUCTURE

During preliminary design, Fuss & O'Neill developed and evaluated multiple bridge alternatives considering several factors including but not limited to overall project cost, maintenance of traffic, and future bridge maintenance. The bridge replacement alternative recommended by Fuss & O'Neill in the Preliminary Design Report (PDR) consisted of a 75-ft long, single-span bridge that is supported on two micropile-supported, cast-in-place (CIP) concrete integral abutments at the stations and elevations summarized below.

Substructure	Station at Centerline of Oceanville Thoroughfare (ft)	Approximate Bottom of Abutment Elevation (ft, NAVD 88)
Abutment 1	Sta. 106+38	El. 3.0
Abutment 2	Sta. 107+13	El. 3.0

The bridge superstructure will be constructed using precast concrete 32 F Next Beam running parallel to the long dimension of the bridge, with a 9-in. thick CIP concrete deck including a 1-in. thick integral wearing surface. The bridge structure will be approximately 25.3-ft wide (edge-to-edge) and will consist of two, 10-ft wide travel lanes and two, 1-ft wide shoulders.

Existing and proposed site conditions are shown on Figure 2, Site and Subsurface Exploration Location Plan.

Geologic Setting

According to Maine Geological Survey's Surficial Geology of the Deer Isle Quadrangle, Maine (1974), surficial geologic units mapped within the site vicinity and the region consist largely of thin glacial till deposits overlying shallow bedrock, as well as areas with extensive bedrock outcrops. Glacial-marine deposits of the Presumpscot Formation are mapped within low lying areas and consist predominantly of marine clay and silt.

According to United States Geological Survey's Geology of Northern Penobscot Bay, Maine (1998), bedrock at the site is mapped as Devonian age igneous rock of the Stonington Granite Formation which consists of biotite granite. There are no indications of mapped faults within the site and vicinity.

Subsurface Exploration Programs

HISTORIC EXPLORATIONS BY OTHERS

No subsurface explorations are shown on the historic plans for the original construction of the bridge (refer to Appendix C). However, the historic plans do indicate that the existing abutments are probably supported on bedrock and the previously existing pier was near bedrock.

PRELIMINARY DESIGN PHASE (PHASE I) SUBSURFACE INVESTIGATION

Haley & Aldrich conducted a preliminary phase geotechnical exploration program at the site in March 2016. All test borings were drilled by New England Boring Contractors (NEBC) of Hermon, Maine. Boring locations were laid out in the field by Haley & Aldrich by taping/pacing distances from existing site features. "As-drilled" locations of the borings were determined in the field by MaineDOT using Global Positioning System (GPS) survey equipment. Coordinate location data and ground surface elevation at exploration locations are provided on individual test boring logs presented in Appendix A and are summarized in Table I. Boring locations are shown on Figure 2.

A total of two test borings (BB-SOT-101 and BB-SOT-102) were drilled close to the existing bridge abutments without drilling through the existing granite blocks. The two test borings were drilled using a truck-mounted Mobile B53 drill rig. Test borings were drilled to depths ranging from 33.8 to 40.0 ft below ground surface (BGS) using 4.0-in. (HW-size) inside diameter (ID) steel casing or 3.0-in. (NW-size) ID steel casing. Soil samples were collected continuously through the fill to 16 ft BGS in BB-SOT-101 and thereafter at standard 5-ft intervals. Soil samples were collected in BB-SOT-102 at standard 5-ft intervals. Soil samples were collected by driving a 1-3/8-in. ID split-spoon sampler with a 140-lb hammer dropped from a height of 30 in., as indicated on the test boring logs.

The number of hammer blows required to advance the sampler through each 6-in. interval was recorded and is provided on the test boring logs. The uncorrected SPT N-value is defined as the total number of blows required to advance the sampler through the middle 12 in. of the 24-in. sampling interval. The energy-corrected Standard Penetration Test (SPT) N-value (N_{60}) is equal to the uncorrected N-value multiplied by the hammer efficiency factor divided by 0.6 (i.e., 60 percent theoretical hammer efficiency). Both the raw blow count data and the corrected N-values are shown on the boring logs.

Bedrock core samples were obtained from both borings using a 2.0-in. (NQ-size) ID diamond-tipped core barrel. Approximately 10 ft (BB-SOT-101) to 15 ft (BB-SOT-102) of rock core was recovered at each boring. Refer to Appendix A for details of the bedrock core.

Drilling and sampling were performed in accordance with MaineDOT specifications. The drill rig was equipped with an automatic hammer per MaineDOT requirements. A theoretical hammer efficiency factor of 0.57 was used for the automatic hammer system based on hammer calibration.

All soil and bedrock samples were classified in accordance with MaineDOT classification system and were preserved in glass jars and wooden boxes. The samples that were not submitted for laboratory

testing are available for review upon request. Soil and bedrock samples are being stored at the Haley & Aldrich storage facility in Portland, Maine.

FINAL DESIGN PHASE (PHASE II) SUBSURFACE INVESTIGATION

Haley & Aldrich completed a design phase (Phase II) subsurface exploration program at the site in June 2022. The Phase II subsurface investigation consisted of two test borings, designated BB-SOT-201 and BB-SOT-202, which were drilled at/near the proposed abutment locations.

The Phase II test borings were laid out in the field by Haley & Aldrich by taping from existing features prior to the start of drilling. "As-drilled" test boring locations and ground surface elevations were determined in the field by MaineDOT using GPS survey equipment upon the completion of drilling and were provided to Haley & Aldrich. The Phase II test boring locations and ground surface elevations are summarized in Table I and are shown on Figure 2.

The Phase II test borings were drilled NEBC of Hermon, Maine using a Mobile Drill B-53 track-mounted drill rig. Test borings were advanced to depths ranging from approximately 50 to 51 ft BGS using similar means and methods that were used to drill, sample and conduct in-situ testing in the Phase I test borings. The hammer efficiency factor for the automatic hammer used to drive the split-spoon sampler was 0.922 (92.2 percent theoretical hammer efficiency) as shown on the test boring logs.

Test borings BB-SOT-201 and BB-SOT-202 were advanced approximately 25 ft into bedrock using a 2.0-in. (NQ-size) ID diamond-tipped core barrel.

Soil and bedrock samples were collected and preserved in glass jars and wooden boxes, respectively. The soil and bedrock samples that were not submitted for laboratory testing are currently being stored at the Haley & Aldrich storage facility in Portland, Maine and are available for review upon request.

The Phase II drilling and sampling activities were performed in accordance with MaineDOT requirements.

Generalized Subsurface Conditions

The subsurface conditions present at the site generally consist of a layer of man-placed fill overlying harbor bottom deposits, marine deposits, glacial till, and bedrock. Refer to Table II for a detailed summary of the soil units and encountered thicknesses at each test boring location. A general description of each soil/bedrock unit is provided separately, below. Detailed soil and bedrock descriptions are provided on the test boring logs included Appendix A. Refer to Figures 3 and 4, Interpretive Subsurface Profile, for a graphical representation of the subsurface conditions present along the proposed bridge alignment.

Geologic Unit	Approximate Encountered Thickness (ft)	Generalized Description
Fill	9.7 to 19.4	Loose to dense fine to coarse SAND with variable amounts of silt and gravel; medium dense fine to coarse GRAVEL with variable amounts of sand and silt. Cobbles and boulders encountered throughout the fill. Bituminous concrete was encountered in the upper 0.3 to 0.7 ft of each test boring. <i>(encountered in each test boring)</i>
Harbor Bottom Deposit	3.6 to 10.3	Very loose to medium dense fine to medium SAND with variable amounts of silt and gravel; very soft organic SILT; loose silty GRAVEL with variable amounts of sand. Boulders encountered within this deposit. <i>(encountered in test borings BB-SOT-102 and BB-SOT-202)</i>
Marine Deposit	1.0 to 4.9	Very soft to stiff silty CLAY with variable amounts of sand and gravel <i>(encountered in test borings BB-SOT-102, BB-SOT-201, and BB-SOT-202)</i>
Glacial Till	3.0 to 6.9	Very loose to dense fine to coarse SAND with variable amounts of silt and gravel <i>(encountered in each test boring)</i>
Bedrock		Top of bedrock surface encountered at depths ranging from approximately 23 to 25.3 ft BGS (El. -10.6 to El. -13.5).

Please note that soil descriptions provided on the test boring logs do not represent actual field conditions other than at the specific test boring locations. The actual conditions encountered between test boring locations may vary from those described herein.

BEDROCK CONDITIONS

As stated previously, approximately 10 to 25 ft of bedrock was cored in the test borings. The sampled and recovered bedrock generally consisted of the following:

- Fresh to moderately weathered, fine to coarse grained GRANITE. Primary joints dipping at horizontal to high angles and are very close to wide, tight to open.

Rock quality designation (RQD) is a common parameter that is used to help assess the competency of sampled bedrock. RQD is defined as the sum of pieces of recovered bedrock greater than 4 in. in length divided by the total length of the bedrock core run. RQD values for the GRANITE encountered at the site ranged from 22 to 97 percent (average of 73 percent) indicating variable rock quality; from very poor to excellent (the average of 73 percent corresponds to fair rock quality).

Detailed bedrock core data and descriptions are provided on Table III and on the boring logs in Appendix A. Photographs of the recovered bedrock core samples are also provided for reference in Appendix A.

WATER ELEVATIONS

The following tidal data was provided by Fuss & O'Neill in a plan set received on 30 December 2022.

Mean Lower Low Water (MLLW): El. -5.65
Mean Low Water (MLW): El. -5.3
Mean Tide Level (MTL): El. -0.37
Mean High Water (MHW): El. 4.6
Mean Higher High Water (MHHW): El. 5.0
Storm Surge Elevation (10-year): El. 8.4
Storm Surge Elevation (100-year): El. 10.9

Geotechnical Laboratory Testing Program

Phase I and Phase II laboratory testing programs were conducted by Haley & Aldrich on representative soil and rock samples to aid in soil classification and determination of engineering soil properties. Laboratory testing was performed in accordance with applicable ASTM testing procedures by GeoTesting Express, Inc. of Acton, Massachusetts. A summary of laboratory test results is provided below.

Grain Size Analyses (ASTM D422, replaced by D6913)

Test Boring No.	Soil Sample No.	Sample Depth (ft)	Percent Finer than #200 Sieve (%)	USCS Classification	Strata
BB-SOT-101	2D	3.0 to 5.0	13.0	SM	Fill
BB-SOT-101	5D	10.0 to 12.0	13.1	SM	Fill
BB-SOT-102	1D	1.0 to 3.0	23.3	SM	Fill
BB-SOT-102	3D	15.0 to 17.0	9.4	SW-SM	Harbor Bottom Deposit

Compressive Strength with Elastic Moduli (ASTM D7012 – Method D)

Test Boring No.	Rock Core No.	Sample Depth (ft)	Peak Compressive Stress (psi)	Young's Modulus (psi)
BB-SOT-101	R1	24.2 to 24.6	20,296	6,330,000 to 7,310,000
BB-SOT-102	R2	31.3 to 31.7	17,668	7,360,000 to 7,760,000
BB-SOT-201	R2	32.4 to 32.8	22,053	6,480,000 to 6,820,000
BB-SOT-201	R3	36.8 to 37.1	23,037	5,190,000 to 5,540,000
BB-SOT-202	R2	29.1 to 29.5	23,764	5,500,000 to 6,000,000
BB-SOT-202	R4	39.4 to 39.8	24,125	5,510,000 to 5,580,000

Laboratory test results are shown on the test boring logs included in Appendix A and complete results are provided in Appendix B.

Geotechnical Evaluations and Design Recommendations

Geotechnical design recommendations, as discussed and provided herein, were developed in accordance with the following documents:

- AASHTO Load and Resistance Factor Design (LRFD) Bridge Design Specifications, Ninth Edition, 2020, referred to herein as AASHTO LRFD,
- AASHTO Guide Specifications for LRFD Seismic Bridge Design, Second Edition, 2011, referred to herein as Guide Specification; and
- MaineDOT Bridge Design Guide (BDG), August 2003, with Interim Revisions through June 2018, referred to herein as Bridge Design Guide.

Engineering calculations that support the design recommendations outlined in this report are provided for reference in Appendix D.

SEISMIC SITE CLASS AND DESIGN PARAMETERS

Site class was determined in accordance with AASHTO LRFD Section 3.10.3.1 using Method C.

Based on the nature and thickness of the overburden soils and depth to bedrock at the site, we recommend the site be considered "Site Class D." Spectral accelerations were determined based on the geographic site location and the recommended "Site Class D" designation using the United States Geological Survey (USGS) seismic design web service, which is based on a seismic event having a 7 percent probability of exceedance in 75 years (approximate 1,000-year return period). The recommended values are summarized below.

Design Parameter	Design Value
Site factor for short-period range of acceleration response spectrum, $F_a =$	1.6
Site factor for long-period range of acceleration response spectrum, $F_v =$	2.4
Site factor at zero-period on acceleration response spectrum, $F_{pga} =$	1.6
Horizontal response spectral acceleration coeff. at 0.2-s period on rock, $S_s (g) =$	0.117
Horizontal response spectral acceleration coeff. at 1.0-s period on rock, $S_1 (g) =$	0.038
Peak seismic ground acceleration coeff. on rock, $PGA (g) =$	0.052
Horizontal response spectral acceleration coeff. at 0.2-s period modified by F_a , $S_{DS} (g) =$	0.188
Horizontal response spectral acceleration coeff. at 1.0-s period modified by F_v , $S_{D1} (g) =$	0.091
Peak seismic ground acceleration coefficient modified by F_{pga} , $A_s (g) =$	0.083

In accordance with AASHTO LRFD Section 3.10.6, the site falls within Seismic Zone 1 based on the calculated value of S_{D1} (i.e., $0.09 < 0.15 =$ Seismic Zone 1 from AASHTO LRFD Table 3.10.6.1).

The minimum factor of safety against liquefaction was determined to be 2.1, following the procedure outlined in Youd and Idriss (2001). Because of this, it is our opinion that the soils at the site are not susceptible to liquefaction during the design earthquake event.

APPROACH EMBANKMENTS

Global Embankment Stability

A series of computer-assisted, two-dimensional global stability evaluations were performed using the computer program Slide 9.0 by Rocscience to evaluate the likelihood of global stability failures along the approach embankments. The global stability under earthquake loading was analyzed using the pseudo-static approach.

Static and pseudo-static stability evaluations were conducted at/near Abutment 1 where soft harbor bottom deposits and marine deposits were encountered. These conditions were not encountered near Abutment 2 and the change in grade is minimal. For these reasons, evaluations at Abutment 2 were not conducted. The proposed abutment micropiles were included in the longitudinal evaluations. Please note that the micropiles are needed to develop acceptable factors of safety.

Typical soil profiles were developed based on the subsurface conditions encountered in the Phase I and Phase II test borings completed at the site. Based on the nature and consistency of the collected soil samples and the SPT N-values measured during soil sampling, physical and strength properties were selected. Refer to Appendix D for the soil properties used at each section. In addition, a 250 psf live load surcharge was assumed to act over the embankment width in each model (permanent condition only).

The factor of safety for pseudo-static load cases was calculated using a horizontal acceleration coefficient, k_h , equal to one half of the acceleration coefficient, A_s . A value of $A_s/2$ was selected in accordance with AASHTO LRFD guidance in Section 11.6.5.2.2; the reduction from A_s is due to soil slope flexibility and the fact that the peak ground acceleration during an earthquake lasts only for a short period of time.

The minimum required factor of safety as specified by both AASHTO LRFD, and the MaineDOT BDG is 1.3 for earth slopes where the geotechnical parameters and subsurface stratigraphy are well defined. The minimum required factor of safety for embankments subjected to pseudo-static loading is 1.1 as specified in AASHTO LRFD or 1.0 as specified in the MaineDOT BDG.

The results of global stability evaluations are summarized below.

Section	Factor of Safety	
	Static	Pseudo-static
Longitudinal, East Profile (100-Series Borings)	1.3	1.2
Longitudinal, West Profile (200-Series Borings)	1.3	1.2
Transverse, Sta. 106+25	1.3	1.1

Elastic and Consolidation Settlement

Minimal raise-in-grade of approach embankments is proposed (less than 0.5 ft). Therefore, we do not anticipate immediate settlement or long-term consolidation settlement will affect the final design of the bridge approaches.

BRIDGE ABUTMENT FOUNDATION SUPPORT

Based on our discussions with Fuss & O'Neill, it is our understanding that the preferred bridge abutment foundation support alternative consists of drilled micropiles with a maximum axial compressive pile load (demand) equal to 160 kips. As discussed below and based on the subsurface conditions present at the site, the micropiles will need to be installed through the existing fill, harbor bottom deposits, marine deposits, and glacial till to gain the majority of their resistance through side resistance in bedrock. Additional micropile evaluations and recommendations are provided below.

Corrosion and Deterioration

The geotechnical engineering design of the proposed micropiles included consideration of corrosion in accordance with AASHTO LRFD Section 10.7.5. The micropiles will be drilled in a marine environment, therefore we have assumed 1/16-in reduction of cross-sectional area of the permanent casing over the planned design life of the bridge (75 years) to account for corrosion.

Axial Compression

For design, we recommend that a strength limit factored unit frictional resistance of 18.6 ksf in bedrock be used to resist axial loads. This design resistance value is based on a resistance factor of 0.55, assuming tension load tests will be performed. This assumes that the micropiles will resist axial loads using the rock socket frictional resistance only.

A summary of the recommended strength limit factored axial geotechnical resistances for several micropile lengths are provided in the table below. It is our understanding that Fuss & O'Neill intends to use 9.625-in. O.D. casing to top of rock and nominal 9-in. diameter uncased pile in bedrock.

Embedment in Bedrock (ft)	Strength Limit Factored Axial Geotechnical Resistance (kips)
5	220
10	435
15	655
20	875

Micropile Settlement and Elastic Compression

Micropile settlement due to elastic shortening of the micropiles as well as pile tip settlement was evaluated based on the maximum axial compressive loads provided by Fuss & O'Neill. Elastic pile compression is estimated to be less than ¼-in. for micropiles with rock sockets less than 20 ft in length.

The values summarized above do not include micropile tip settlement, which is considered to be negligible for two primary reasons: 1) the relatively small load transmitted to the micropile tip during service limit state loading and 2) the micropiles will be installed to rock. The elastic shortening of the micropiles is anticipated to occur primarily during construction, soon after the superstructure loads are applied.

Lateral Pile Resistance

Soil profiles, soil properties, and rock properties at the abutments were developed for Fuss & O'Neill's use in conducting soil structure interaction/lateral pile evaluations. Recommended lateral soil properties and profiles are summarized in Table IV and V.

Pile Embedment, Spacing, and Clearance

We recommend that minimum pile spacing, clearance and embedment (into the abutment pile caps) meet the requirements of AASHTO LRFD Section 10.9.1.2.

ABUTMENT DESIGN

For structural design of integral abutments, we recommend use of passive earth pressures, using a Coulomb passive earth pressure coefficient, K_p , of 6.6. Developing full passive pressure assumes that the ratio of lateral abutment movement to abutment height (y/H) exceeds 0.005 (where H is measured from bottom of pile cap to top of abutment). If the calculated displacements are significantly less than that required to develop full passive pressure (i.e., y/H less than 0.005), the Rankine passive earth pressure coefficient, 3.25, should be used.

Soil properties of Soil Type 4, granular borrow, should be used for abutment backfill material. The backfill properties per BDG Section 3.6.1 are as follows:

- Friction Angle, ϕ , 32 degrees
- Total Unit Weight, γ , 125 pcf
- Soil-Concrete Interface Friction Angle, δ , 24 degrees.

In addition, walls should be designed for a live load surcharge equivalent to the earthfill height summarized in LRFD Tables 3.11.6.4-1 and 3.11.6.4-2. A uniform lateral load equal to the surcharge times the lateral earth pressure coefficient should be applied to abutments and walls to account for the live load surcharge.

We also recommend that the abutment design include a drainage system in accordance with the requirements of BDG Sections 5.4.1.9 and 5.4.2.13 to intercept any groundwater and direct it to a suitable discharge point that does not adversely affect the performance of the abutments. Using a free-draining backfill material and providing adequate drainage will substantially reduce the potential for unbalanced hydrostatic pressures from developing.

If determined applicable by Fuss & O'Neill, the abutments should be designed for a uniform lateral load to account for seismic soil loading in accordance with LRFD Article A.11.3.1 (Mononobe-Okabe Method). Based on the seismic site class (Site Class "D"), we recommend a seismic active earth pressure coefficient, K_{AE} , of 0.33 be used for design. Note that this soil pressure includes both the static and seismic lateral earth loads.

FROST PROTECTION

The minimum depth of embedment/cover for footings or other below grade structures was evaluated in accordance with Section 5.2.1 of the MaineDOT BDG. Based on a design freezing index equal to 1,050 freezing degree-days, we recommend that walls or other foundations bear a minimum of 5 ft below the lowest adjacent ground surface exposed to freezing. This frost depth used an assumed water content of 20% and only considered granular soils (embankment fill). However, in accordance with MaineDOT BDG Section 5.4.2.8, the bottom of the integral abutment pile cap should be a minimum of 4 ft below the adjacent ground surface.

Construction Considerations

Based on the subsurface conditions encountered and our understanding of the proposed bridge replacement, we offer the following general geotechnical observations regarding the planned bridge construction.

EXCAVATION

We anticipate that the construction of the new bridge abutments will require excavation through the in-situ fill and harbor bottom deposits. We anticipate that excavation can be accomplished using normal earth-excavating equipment (i.e., hydraulic backhoes and excavators). Soft cohesive material beneath the proposed abutments should be removed, if encountered.

As noted on the test boring logs, the existing fill and harbor bottom deposit may contain cobbles and possibly some large boulders. We recommend that the Contract Documents require the Contractor to include provisions for cobble/boulder removal in their bid.

MICROPILES

Micropiles will need to be advanced through cobbles, boulders, wood and other structures in the existing fill and glacial till. Micropile installation is well suited to penetrate through oversized rock fragments. Obstruction allowances should be considered in the contract. We note that care should be exercised by the Contractor when drilling with compressed air since this method of drilling can cause subsurface erosion near the installed piles.

Duplex pile drilling is recommended in order to minimize erosion of soils near the pile installation.

MICROPILE LOAD TESTING PROGRAM

The Contractor will be required to confirm the minimum required nominal micropile resistances in the field. We recommend that Contractor perform one verification load test at each abutment (two total). The micropiles should be tested to a nominal resistance equal to the maximum factored axial compressive micropile load divided by a resistance factor equal to 0.7 in accordance with AASHTO LRFD Table 10.5.5.2.5-1. We recommend that the testing be completed prior to production micropile installation.

CONSTRUCTION MONITORING

The geotechnical design and earthwork recommendations contained herein are based on the known and predictable behavior of a properly engineered and constructed foundation. Monitoring of the foundation construction activities is required to enable the geotechnical engineer to confirm that procedures and techniques used by the contractor during construction are appropriate and will not impact the design of the bridge. Therefore, we recommend that an individual representing MaineDOT, qualified by geotechnical training and experience be present at the site to provide monitoring during the foundation construction activities listed below.

- Installation and testing of micropiles.
- Placement and compaction of compacted fills.

Limitations

This report is prepared for the exclusive use of Fuss & O'Neill and MaineDOT relative to the subject project. There are no intended beneficiaries other than Fuss & O'Neill and MaineDOT. Haley & Aldrich shall owe no duty whatsoever to any other person or entity on account of the Agreement or the report. Use of this report by any person or entity other than Fuss & O'Neill and MaineDOT for any purpose whatsoever is expressly forbidden unless such other person or entity obtains written authorization from Fuss & O'Neill and Haley & Aldrich. Use of this report by such other person or entity without the written

authorization of Fuss & O'Neill and Haley & Aldrich shall be at such other person's or entities sole risk and shall be without legal exposure or liability to Haley & Aldrich.

Use of this report by any person or entity, including by Fuss & O'Neill and MaineDOT, for a purpose other than relative to the subject project is expressly prohibited unless such person or entity obtains written authorization from Haley & Aldrich indicating that the report is adequate for such other use. Use of this report by any other person or entity for such other purpose without written authorization by Haley & Aldrich shall be at such person's or entities sole risk and shall be without legal exposure or liability to Haley & Aldrich.

The information provided herein is based, in part, upon the data obtained from the referenced subsurface explorations. The nature and extent of variations between explorations may not become evident until construction. If variations then appear, it may be necessary to reevaluate the recommendations of this report.

It is our understanding that this report may be included as a reference document in the documents that will be provided to the prospective Contractors for bidding. Please note that the recommendations included herein are superseded by the information contained in the documents and that the information contained in the documents takes precedence over the information provided in this report.

Closure

We appreciate the opportunity to continue to provide Fuss & O'Neill and MaineDOT services on this project. Please do not hesitate to contact us if you have any questions or comments.

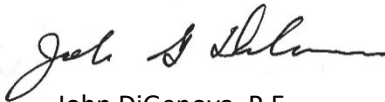
Sincerely yours,
HALEY & ALDRICH, INC.



Eric Hunstein, E.I.
Staff Geotechnical Engineer



Erin Force, P.E.
Program Manager



John DiGenova, P.E.
Program Manager

Enclosures:

Table I –	Subsurface Exploration Location Data
Table II –	Subsurface Exploration Subsurface Data
Table III –	Subsurface Exploration Bedrock Core Data
Table IV –	LPile Parameters – Mean High Water
Table V –	LPile Parameters – Mean Low Water
Figure 1 –	Project Locus
Figure 2 –	Site and Subsurface Exploration Location Plan
Figure 3 –	Interpretive Subsurface Profile (1 of 2)
Figure 4 –	Interpretive Subsurface Profile (2 of 2)
Appendix A –	Test Boring Logs and Rock Core Photographs
Appendix B –	Laboratory Test Results
Appendix C –	Historic Bridge Drawings
Appendix D –	Geotechnical Design Calculations



TABLE I

Subsurface Exploration Location Data

Oceanville Bridge over Oceanville Thoroughfare, Bridge No. 3696

MaineDOT WIN 22630.00

Stonington, Maine

Haley & Aldrich, Inc. File No.: 42835-002

Test Boring No. ¹	Ground Surface Elevation (ft) ³	Station ⁴	Offset Distance (ft) & Direction ^{4,5}	Horizontal Coordinates ²	
				Northing (Y)	Easting (X)
BB-SOT-101	12.4	107+15	5.2 RT	248,364	1,770,689
BB-SOT-102	12.2	106+31	6.9 RT	248,448	1,770,686
BB-SOT-201	11.8	107+22	6.9 LT	248,455	1,770,673
BB-SOT-202	11.7	106+35	5.5 LT	248,368	1,770,676

Notes:¹ Test boring locations are shown on Figure 2, Site and Subsurface Exploration Location Plan.² As-drilled coordinates of test borings were determined by MaineDOT using GPS survey equipment, are measured in feet (ft), and reference NAD83, Maine 2000 Central Zone coordinate system.³ Ground surface elevations at test boring locations were determined in the field by MaineDOT using GPS survey equipment, are measured in ft, and reference the North American Vertical Datum of 1988 (NAVD 88).⁴ Station and offset information shown are approximate, are relative to the Oceanville Road baseline, and were determined by Haley & Aldrich based on information provided by MaineDOT and rounded to the nearest ft.⁵ LT = offset distance toward left direction; RT = offset distance toward right direction.

	Individual	Date
Prepared By:	EMH	2/10/2023
Checked By:	NAS	3/7/2023
Reviewed By:	EAF	9/29/2023

TABLE II
Design Phase and Final Design Subsurface Exploration Subsurface Data
Oceanville Bridge over Oceanville Thoroughfare, Bridge No. 3696
MaineDOT WIN 22630.00
Stonington, Maine

Haley & Aldrich, Inc. File No.: 42835-002

Test Boring No. ¹	Ground Surface Elevation ² (ft)	Stratigraphy Data ^{2,3}														Bottom of Exploration Depth (ft)	Elevation of Bottom of Exploration ² (ft)
		Fill ⁴			Harbor Bottom Deposit ⁵			Marine Deposit			Glacial Till			Bedrock			
		Depth to Top (ft)	Elev. of Top (ft)	Thickness (ft)	Depth to Top (ft)	Elev. of Top (ft)	Thickness (ft)	Depth to Top (ft)	Elev. of Top (ft)	Thickness (ft)	Depth to Top (ft)	Elev. of Top (ft)	Thickness (ft)	Depth to Top (ft)	Elev. of Top (ft)		
BB-SOT-101	12.4	0.0	12.4	19.4	NE	NE	NE	NE	NE	NE	19.4	-7.0	3.6	23.0	-10.6	33.8	-21.4
BB-SOT-102	12.2	0.0	12.2	15.0	15.0	-2.8	3.6	18.6	-6.4	2.9	21.5	-9.3	3.0	24.5	-12.3	40.0	-27.8
BB-SOT-201	11.8	0.0	11.8	13.5	NE	NE	NE	13.5	-1.7	4.9	18.4	-6.6	6.9	25.3	-13.5	50.5	-38.7
BB-SOT-202	11.7	0.0	11.7	9.7	9.7	2.0	10.3	20.0	-8.3	1.0	21.0	-9.3	3.5	24.5	-12.8	50.0	-38.3

Notes:

- ¹ Test boring locations are shown on Figure 2, Site and Subsurface Exploration Location Plan.
- ² Ground surface elevations at test boring locations were determined in the field by MaineDOT using GPS survey equipment, are measured in feet (ft) and reference the North American Vertical Datum of 1988 (NAVD 88).
- ³ "NE" indicates stratum was not encountered in test boring.
- ⁴ Bituminous concrete was encountered in the upper 0.3 to 0.7 ft of each test boring.
- ⁵ Boulders were encountered in the Harbor Bottom Deposit in test borings BB-SOT-102 and BB-SOT-202.

	Individual	Date
Prepared By:	EMH	2/10/2023
Checked By:	NAS	3/7/2023
Reviewed By:	EAF	9/29/2023

TABLE III
Design Phase and Final Design Subsurface Exploration Bedrock Core Data
Oceanville Bridge over Oceanville Thoroughfare, Bridge No. 3696
MaineDOT WIN 22630.00
Stonington, Maine

Haley & Aldrich, Inc. File No.: 42835-002

Test Boring No. ¹	Estimated Ground Surface Elevation ² (ft)	Bedrock Core Diameter (in.)	Run					Total Core Recovery ³		Rock Quality Designation ^{4,5}		Physical Rock Parameters		Peak Compressive Strength ⁶ (ksf)	Lithologic, Rock Mass, and Discontinuity Description
			No.	Depth Below Ground Surface (ft)			Total Length (ft)	Recovered Length (ft)	%	%	Designation	Weathering	Estimated Field Strength		
				Top	Bottom	Midpoint									
BB-SOT-101	12.4	NQ (2")	R1	23.8	28.8	26.3	5.0	5.0	100%	90%	Good	Fresh	Very hard	2,923	Pink to light gray, fine to coarse grained GRANITE, primary joints dipping at moderate angles, spacing very close, aperatures tight, occasionally healed with hematite coatings on joint surfaces. Secondary joint sets dipping at low angles, moderately close spacing, aperatures tight to open, smooth to rough and undulating, occasional silt infilling
			R2	28.8	33.8	31.3	5.0	5.0	100%	97%	Excellent	Fresh	Very hard	-	
BB-SOT-102	12.2	NQ (2")	R1	25.0	30.0	27.5	5.0	5.0	100%	42%	Poor	Slightly	Hard to Very Hard	-	Pinkish-red to pinkish-gray, fine to coarse grained GRANITE, primary joints dipping at moderate to high angles, wide spacing, apeatures open, smooth, silt infilling. Secondary joints low angle to horizontal, spacing close to moderately close, aperatures open, joint surfaces rough and undulating, occasional silt infilling.
			R2	30.0	35.0	32.5	5.0	5.0	100%	75%	Fair	Fresh to Slightly	Hard to Very Hard	2,544	Pink to light gray, fine to coarse grained GRANITE, primary joints moderately dipping, moderate spacing, aperatures open, smooth to rough, weathered joint surfaces. Secondary joints horizontal to very low angle, very close to close spacing, aperatures open, rough, weathered joint surfaces.
			R3	35.0	40.0	37.5	5.0	5.0	100%	22%	Very Poor	Slightly to Moderately	Hard	-	Dark red to reddish-gray, fine to coarse grained GRANITE, primary joints dipping at moderate to high angles, spacing close, aperatures open, rough and undulating, frequent silt infilling, occasional chlorite coatings on joint surfaces. Secondary joints low angle, close spacing, aperatures open, rough, silt infilling.

TABLE III
Design Phase and Final Design Subsurface Exploration Bedrock Core Data
Oceanville Bridge over Oceanville Thoroughfare, Bridge No. 3696
MaineDOT WIN 22630.00
Stonington, Maine

Haley & Aldrich, Inc. File No.: 42835-002

Test Boring No. ¹	Estimated Ground Surface Elevation ² (ft)	Bedrock Core Diameter (in.)	Run					Total Core Recovery ³		Rock Quality Designation ^{4,5}		Physical Rock Parameters		Peak Compressive Strength ⁶ (ksf)	Lithologic, Rock Mass, and Discontinuity Description
			No.	Depth Below Ground Surface (ft)			Total Length (ft)	Recovered Length (ft)	%	%	Designation	Weathering	Estimated Field Strength		
				Top	Bottom	Midpoint									
BB-SOT-201	11.8	NQ (2")	R1	25.5	30.5	28.0	5.0	5.0	100%	65%	Fair	Fresh to Slightly	Hard to Very Hard	-	Pink to grey, medium to coarse-grained GRANITE, joints dipping at horizontal to high angles, spacing very close to close, aperatures tight to open, slightly weathered joint surfaces. Highly weathered zone at depth 49.7 to 49.8 ft.
			R2	30.5	35.5	33.0	5.0	5.0	100%	82%	Good	Fresh to Slightly	Hard to Very Hard	3,176	
			R3	35.5	40.5	38.0	5.0	4.9	98%	73%	Fair	Fresh to Slightly	Hard to Very Hard	3,317	
			R4	40.5	45.5	43.0	5.0	5.0	100%	83%	Good	Fresh to Slightly	Hard to Very Hard	-	
			R5	45.5	50.5	48.0	5.0	5.0	100%	87%	Good	Fresh to Slightly	Hard to Very Hard	-	

TABLE III
Design Phase and Final Design Subsurface Exploration Bedrock Core Data
Oceanville Bridge over Oceanville Thoroughfare, Bridge No. 3696
MaineDOT WIN 22630.00
Stonington, Maine

Haley & Aldrich, Inc. File No.: 42835-002

Test Boring No. ¹	Estimated Ground Surface Elevation ² (ft)	Bedrock Core Diameter (in.)	Run					Total Core Recovery ³		Rock Quality Designation ^{4,5}		Physical Rock Parameters		Peak Compressive Strength ⁶ (ksf)	Lithologic, Rock Mass, and Discontinuity Description
			No.	Depth Below Ground Surface (ft)			Total Length (ft)	Recovered Length (ft)	%	%	Designation	Weathering	Estimated Field Strength		
				Top	Bottom	Midpoint									
BB-SOT-202	11.7	NQ (2")	R1	25.0	29.0	27.0	4.0	4.0	100%	67%	Fair	Fresh to Slightly	Very Hard	-	Pink to grey, medium to coarse-grained GRANITE, joints dipping from moderate to high angles, spacing close to very close, aperatures tight to open.
			R2	29.0	33.0	31.0	4.0	4.0	100%	67%	Fair	Fresh to Slightly	Very Hard	3,422	Pink to grey, medium to coarse-grained GRANITE, primary joints dipping from moderate to high angles, spacing close to very close, aperatures tight to open. Secondary joints horizontal and vertical, close to moderately close spacing, aperatures tight to open. Highly fractured zone at approximately depth 21.1 to 33.0 ft.
			R3	33.0	38.0	35.5	5.0	4.8	97%	88%	Good	Fresh to Slightly	Very Hard	-	Pink to grey, medium to coarse-grained GRANITE, primary joints dipping from moderate to high angles, spacing close to very close, aperatures tight to open. Secondary joints horizontal and low angle, moderately close spacing, aperatures tight to open.
			R4	38.0	43.0	40.5	5.0	5.0	100%	77%	Good	Fresh to Slightly	Very Hard	3,473	Pink to grey, medium to coarse-grained GRANITE, primary joints dipping from moderate to horizontal angles, spacing close to very close, aperatures tight to open. Second high angle joint at approximately 42.5 ft, very close to moderately close spacing, aperatures tight to open.
			R5	43.0	48.0	45.5	5.0	5.1	101%	92%	Excellent	Fresh to Slightly	Very Hard	-	Pink to grey, medium to coarse-grained GRANITE, primary joints dipping from moderate to high angles, spacing close to moderately close, aperatures tight to open, two high angle healed joints. Secondary vertical joint at approximately 47.5 ft.
			R6	48.0	50.0	49.0	2.0	1.2	58%	58%	Fair	Fresh to Slightly	Very Hard	-	Pink to grey, medium to coarse-grained GRANITE, joints dipping from low to high angles, spacing close, aperatures open.

Notes:
¹ Test boring locations are shown on Figure 2, Site and Subsurface Exploration Location Plan.
² Ground surface elevations at test boring locations were determined in the field by MaineDOT using GPS survey equipment, are measured in feet (ft) and reference the North American Vertical Datum of 1988 (NAVD 88).
³ TCR = total core recovery. Total core recovery is the length of core recovered divided by the length of the run.
⁴ RQD = rock quality designation. RQD is the total length of intact, full-diameter core pieces recovered with a length greater than or equal to twice the core diameter (i.e., length of at least 4 in.) measured along the core axis.
The percent RQD is the total length of RQD measured versus the run length. Note that vertical discontinuities are not included in determination of RQD.
⁵ Designation based on RQD in accordance with MaineDOT Geotechnical Section "Key to Soil and Rock Descriptions and Terms" Field Identification Information.
⁶ Peak compressive strength was determined in accordance with ASTM D7012 - Method D.

	Individual	Date
Prepared By:	EMH	2/10/2023
Checked By:	NAS	3/7/2023
Reviewed By:	EAF	9/29/2023

TABLE IV

L-Pile Parameters - Mean High Water
 Oceanville Bridge over Oceanville Thoroughfare, Bridge No. 3696
 MaineDOT WIN 22630.00
 Stonington, Maine

Haley & Aldrich, Inc. File No.: 42835-002

Abutment No. 1 (based on boring BB-SOT-202)

Soil Layer	Soil Model	Top Elevation (ft)	Effective Unit Weight, γ' (pcf)	Undrained Shear Strength, S_u (psf)	Internal Angle of Friction, ϕ' (degrees)	Soil Modulus, k (pci)	Strain Factor, ϵ_{50}
Fill	Sand (Reese)	3.0	56.0	N/A	32	60	N/A
Harbor Bottom Cohesive	Soft Clay (Matlock)	2.0	46.0	400	N/A	N/A	0.020
Harbor Bottom Cohesionless	Sand (Reese)	-4.1	46.0	N/A	32	60	N/A
Glacial Till	Sand (Reese)	-9.3	66.0	N/A	36	125	N/A
Bedrock	See Below	-12.8	N/A	N/A	N/A	N/A	N/A

Rock Layer	Rock Model	Top Elevation (ft)	Bottom Elevation (ft)	Effective Unit Weight (pcf)	γ (in)	P (lb/in)
Bedrock	User Input P-y Curves	-12.8	bottom of micropile	100	0	0
					0.01	18,000
					10	18,000

Abutment No. 2 (based on boring BB-SOT-201)

Soil Layer	Soil Model	Top Elevation (ft)	Effective Unit Weight, γ' (pcf)	Undrained Shear Strength, S_u (psf)	Internal Angle of Friction, ϕ' (degrees)	Soil Modulus, k (pci)	Strain Factor, ϵ_{50}
Fill	Sand (Reese)	3.0	56.0	N/A	32	60	N/A
Marine Deposit	Soft Clay (Matlock)	-1.7	51.0	1000	N/A	N/A	0.010
Glacial Till	Sand (Reese)	-6.6	66.0	N/A	36	125	N/A
Bedrock	See Below	-13.5	N/A	N/A	N/A	N/A	N/A

Rock Layer	Rock Model	Top Elevation (ft)	Bottom Elevation (ft)	Effective Unit Weight (pcf)	γ (in)	P (lb/in)
Bedrock	User Input P-y Curves	-13.5	bottom of micropile	100	0	0
					0.01	36,000
					10	36,000

	Individual	Date
Prepared By:	EMH	8/2/2023
Checked By:	MMB	8/2/2023
Reviewed By:	EAJ	8/2/2023

TABLE V

LPile Parameters - Mean Low Water

Oceanville Bridge over Oceanville Thoroughfare, Bridge No. 3696

MaineDOT WIN 22630.00

Stonington, Maine

Haley & Aldrich, Inc. File No.: 42835-002

Abutment No. 1 (based on boring BB-SOT-202)

Soil Layer	Soil Model	Top Elevation (ft)	Effective Unit Weight, γ' (pcf)	Undrained Shear Strength, S_u (psf)	Internal Angle of Friction, ϕ' (degrees)	Soil Modulus, k (pci)	Strain Factor, ϵ_{50}
Fill	Sand (Reese)	3.0	120.0	N/A	32	90	N/A
Harbor Bottom Cohesive	Soft Clay (Matlock)	2.0	110.0	400	N/A	N/A	0.020
Harbor Bottom Cohesionless	Sand (Reese)	-4.1	46.0	N/A	32	60	N/A
Glacial Till	Sand (Reese)	-9.3	66.0	N/A	36	125	N/A
Bedrock	See Below	-12.8	N/A	N/A	N/A	N/A	N/A

Rock Layer	Rock Model	Top Elevation (ft)	Bottom Elevation (ft)	Effective Unit Weight (pcf)	γ (in)	P (lb/in)
Bedrock	User Input P-y Curves	-12.8	bottom of micropile	100	0	0
					0.01	18,000
					10	18,000

Abutment No. 2 (based on boring BB-SOT-201)

Soil Layer	Soil Model	Top Elevation (ft)	Effective Unit Weight, γ' (pcf)	Undrained Shear Strength, S_u (psf)	Internal Angle of Friction, ϕ' (degrees)	Soil Modulus, k (pci)	Strain Factor, ϵ_{50}
Fill	Sand (Reese)	3.0	120.0	N/A	32	90	N/A
Marine Deposit	Soft Clay (Matlock)	-1.7	115.0	1000	N/A	N/A	0.010
Glacial Till	Sand (Reese)	-6.6	66.0	N/A	36	125	N/A
Bedrock	See Below	-13.5	N/A	N/A	N/A	N/A	N/A

Rock Layer	Rock Model	Top Elevation (ft)	Bottom Elevation (ft)	Effective Unit Weight (pcf)	γ (in)	P (lb/in)
Bedrock	User Input P-y Curves	-13.5	bottom of micropile	100	0	0
					0.01	36,000
					10	36,000

	Individual	Date
Prepared By:	EMH	8/2/2023
Checked By:	MMB	8/2/2023
Reviewed By:	EAF	8/2/2023



MAP SOURCE: ESRI

SITE COORDINATES: 44°10'49"N, 68°37'42"W



**HALEY
ALDRICH**

OCEANVILLE BRIDGE OVER OCEANVILLE THOROUGHFARE
MAINEDOT WIN 22630.00, BRIDGE NO. 3696
STONINGTON, MAINE

PROJECT LOCUS

APPROXIMATE SCALE: 1 IN = 2000 FT
OCTOBER 2023

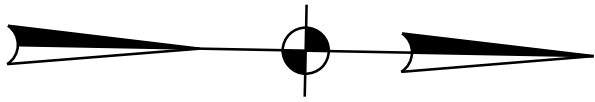
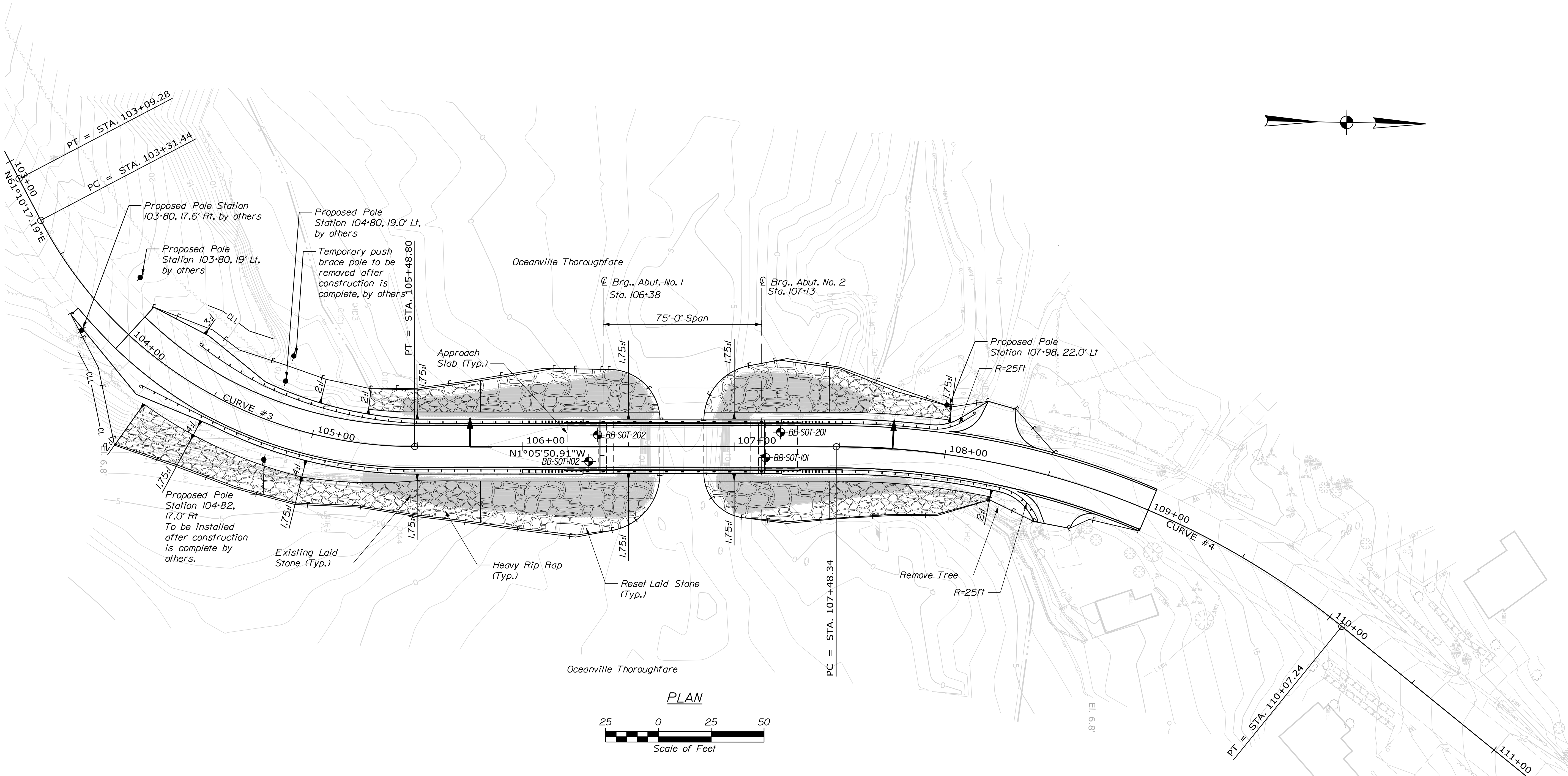
FIGURE 1

Date: 10/4/2023

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Division:

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DESIGNATION AND LOCATION OF TEST BORING DRILLED BY NEW ENGLAND BORING CONTRACTORS BETWEEN 22 AND 23 MARCH 2016 (BB-SOT-101 & BB-SOT-102) AND BETWEEN 7 AND 9 JUNE 2022 (BB-SOT-201 & BB-SOT-202).

DESIGNATION, APPROXIMATE LOCATION, AND ORIENTATION OF INTERPRETIVE SUBSURFACE PROFILES (SEE SHEETS 3 & 4).

- NOTES
- EXISTING AND PROPOSED GROUND SURFACE/MUDLINE ELEVATIONS AND BRIDGE STRUCTURE GEOMETRY WERE TAKEN FROM THE ELECTRONIC FILES PROVIDED BY FUSS & O'NEILL, INC. ON 10 NOVEMBER 2016 AND 27 JUNE 2023.
 - AS-DRILLED LOCATIONS OF TEST BORINGS WERE DETERMINED IN THE FIELD BY MAINEDOT USING GPS SURVEY EQUIPMENT.
 - SUBSURFACE EXPLORATIONS WERE MONITORED IN THE FIELD BY HALEY & ALDRICH PERSONNEL.
 - REFER TO APPENDIX A FOR TEST BORING LOGS.
 - ELEVATIONS ARE IN FEET AND REFERENCE THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD 1988).

OCEANVILLE BRIDGE OCEANVILLE THOROUGHFARE STONINGTON	STATE OF MAINE DEPARTMENT OF TRANSPORTATION		
	STP-2263(000)		
	BRIDGE NO. 3696 WIN 22630.00 BRIDGE PLANS		
OCEANVILLE THOROUGHFARE HANCOCK COUNTY STONINGTON SITE AND SUBSURFACE EXPLORATION LOCATION PLAN	PROJ. MANAGER A. Lohr		BY K. Post
	CHECKED-DETAILED E. Force		DATE 6/23
	CHECKED-REVIEWED E. Force		SIGNATURE 10/23
	DESIGN-DETAILED -		P.E. NUMBER -
	REVISIONS 1 -		DATE -

SHEET NUMBER

2

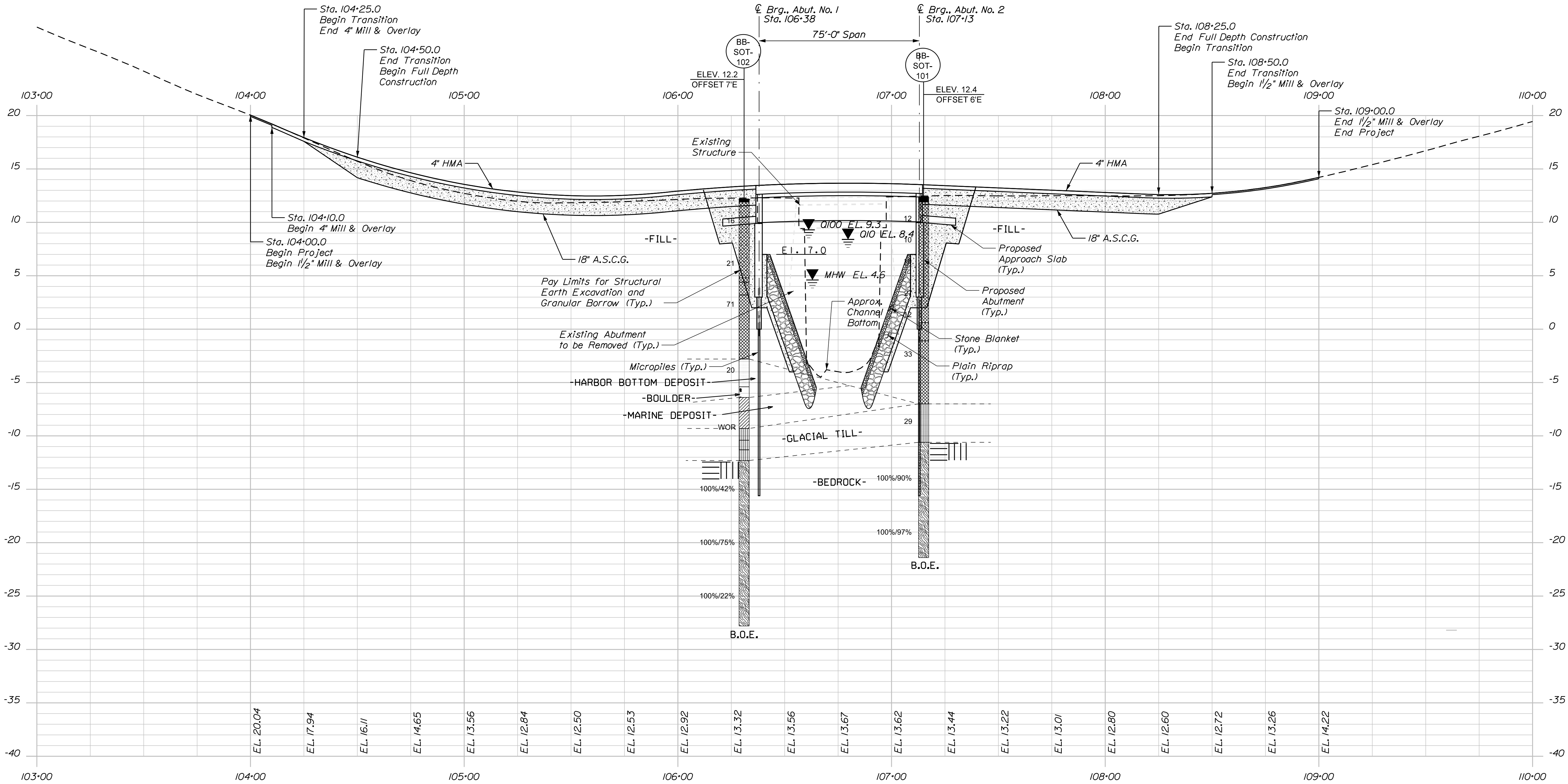
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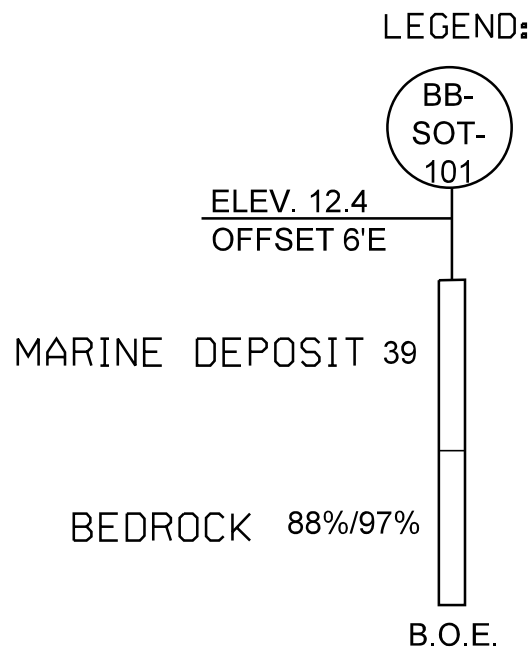
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EAST PROFILE

NOTES:

- BORING OFFSETS ARE REPORTED RELATIVE TO THE OCEANVILLE ROAD CENTERLINE.
- EXISTING AND PROPOSED GROUND SURFACE/MUDLINE ELEVATIONS AND BRIDGE STRUCTURE GEOMETRY ALONG OCEANVILLE ROAD CENTERLINE WERE TAKEN FROM THE ELECTRONIC FILES PROVIDED BY FUSS & O'NEILL INC. ON 27 JUNE 2023.
- THE "AS-DRILLED" LOCATIONS AND ELEVATIONS OF TEST BORINGS WERE DETERMINED IN THE FIELD BY MAINEDOT USING GPS SURVEY EQUIPMENT.
- LINES REPRESENTING CHANGES IN STRATA ARE BASED ON LINEAR INTERPOLATION BETWEEN SUBSURFACE EXPLORATIONS. THESE INTERPRETED STRATA LINES DO NOT REPRESENT ACTUAL FIELD CONDITIONS OTHER THAN AT SPECIFIC EXPLORATION LOCATIONS. ACTUAL FIELD CONDITIONS ALONG THE SUBSURFACE PROFILE WILL VARY FROM THOSE SHOWN.
- ELEVATIONS ARE IN FEET AND REFERENCE THE NORTH AMERICAN VERTICAL DATUM OF 1988 (NAVD88).



TEST BORING DESIGNATION

GROUND SURFACE ELEVATION AT TEST BORING
APPROXIMATE PERPENDICULAR OFFSET DISTANCE AND
DIRECTION MEASURED FROM PROPOSED CENTERLINE

ENERGY CORRECTED STANDARD PENETRATION TEST (SPT) N60-VALUE
(BLOWS PER FOOT); WOR = WEIGHT OF RODS

STRATA INTERFACE

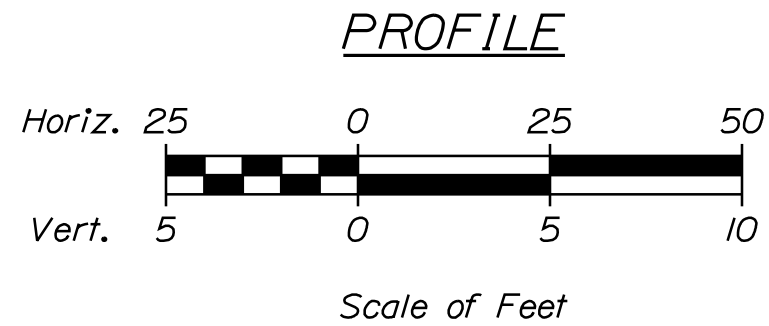
PERCENT RECOVERY
PERCENT ROCK QUALITY DESIGNATION (ROD)

DENOTES BOTTOM OF EXPLORATION AT APPROXIMATE DEPTH SHOWN

STRATA SYMBOLS

- FILL
- PAVING
- SILTY SAND
- COBBLES
- BEDROCK

- WELL GRADED SAND
- LOW PLASTICITY CLAY



HALEY
ALDRICH

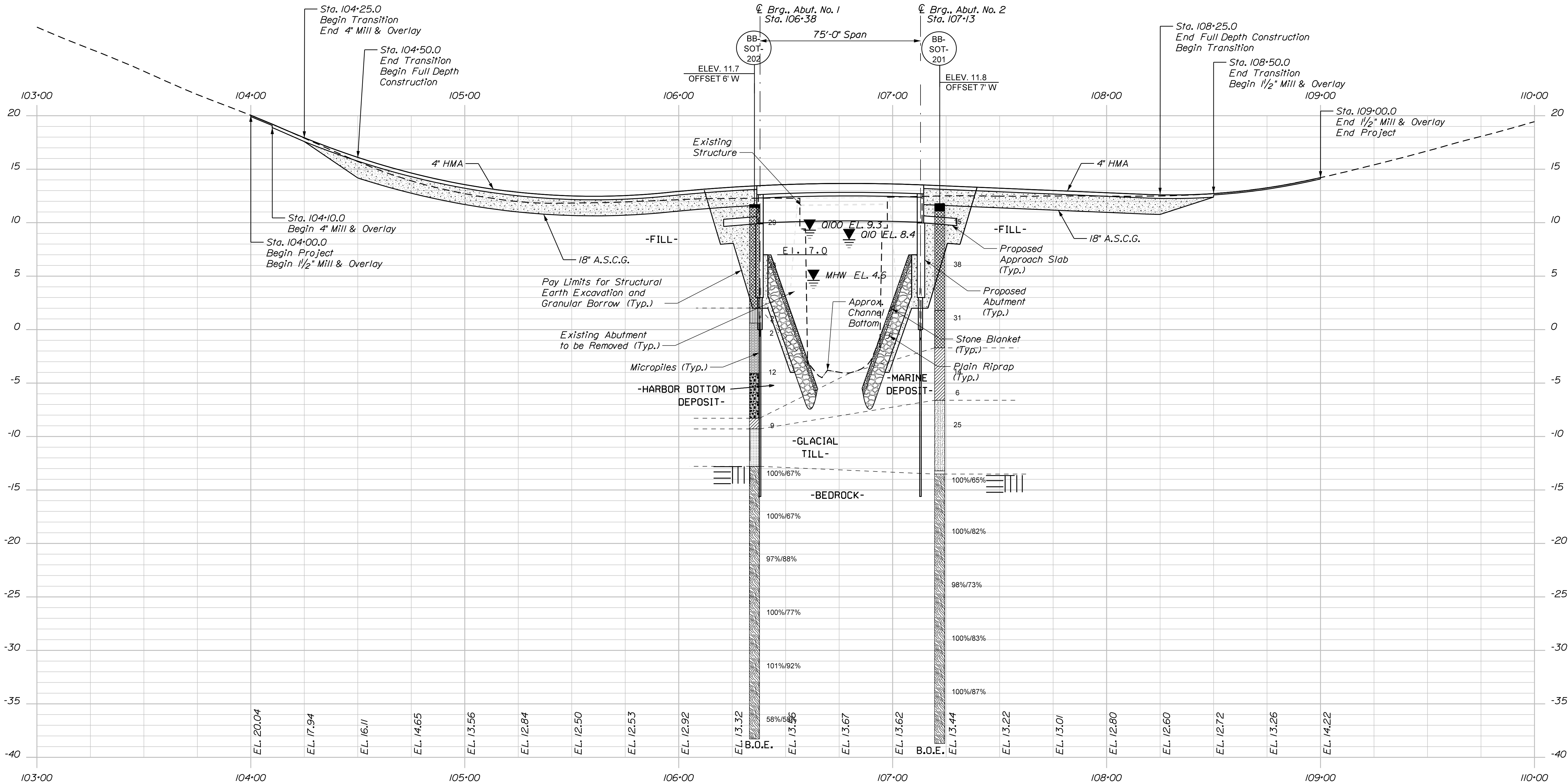
OCEANVILLE BRIDGE		PROJ. MANAGER		A. Lotte	BY	DATE
OCEANVILLE THOROUGHFARE		DESIGN-DETAILED		E. Force	K. Post	6/23
STONINGTON HANCOCK COUNTY		CHECKED-REVIEWED		E. Force	K. Post	10/23
INTERPRETIVE SUBSURFACE		DESIGN2-DETAILED2		-	-	SIGNATURE
PROFILE (1 OF 2)		DESIGN3-DETAILED3		-	-	P.E. NUMBER
		REVISIONS 1		-	-	
		REVISIONS 2		-	-	
		REVISIONS 3		-	-	DATE
		REVISIONS 4		-	-	
		FIELD CHANGES		-	-	
SHEET NUMBER		BRIDGE NO. 3696				
3		WIN				
OF 4		22630.00				
		BRIDGE PLANS				
		STATE OF MAINE				
		DEPARTMENT OF TRANSPORTATION				
		STP-2263(000)				

Date:10/4/2023

Username:

Division:

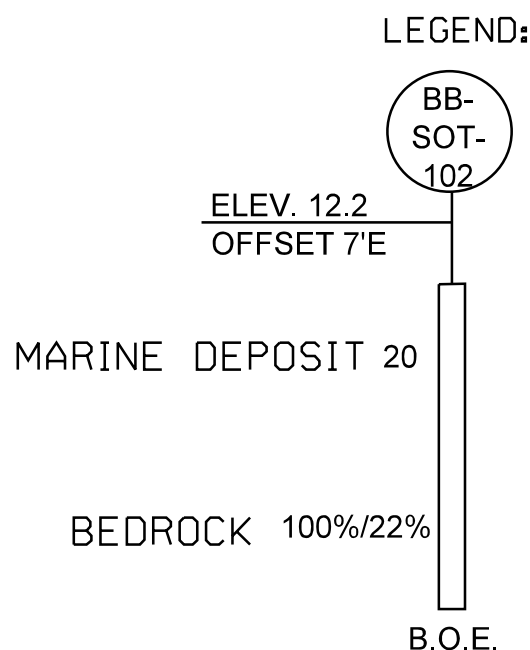
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WEST PROFILE

NOTES:

- BORING OFFSETS ARE REPORTED RELATIVE TO THE OCEANVILLE ROAD CENTERLINE.
- EXISTING AND PROPOSED GROUND SURFACE/MUDLINE ELEVATIONS AND BRIDGE STRUCTURE GEOMETRY ALONG OCEANVILLE ROAD CENTERLINE WERE TAKEN FROM THE ELECTRONIC FILES PROVIDED BY FUSS & O'NEILL INC. ON 27 JUNE 2023.
- THE "AS-DRILLED" LOCATIONS AND ELEVATIONS OF TEST BORINGS WERE DETERMINED IN THE FIELD BY MAINEDOT USING GPS SURVEY EQUIPMENT.
- LINES REPRESENTING CHANGES IN STRATA ARE BASED ON LINEAR INTERPOLATION BETWEEN SUBSURFACE EXPLORATIONS. THESE INTERPRETED STRATA LINES DO NOT REPRESENT ACTUAL FIELD CONDITIONS OTHER THAN AT SPECIFIC EXPLORATION LOCATIONS. ACTUAL FIELD CONDITIONS ALONG THE SUBSURFACE PROFILE WILL VARY FROM THOSE SHOWN.
- ELEVATIONS ARE IN FEET AND REFERENCE THE NORTH AMERICAN VERITCAL DATUM OF 1988 (NAVD88).



TEST BORING DESIGNATION

GROUND SURFACE ELEVATION AT TEST BORING
APPROXIMATE PERPENDICULAR OFFSET DISTANCE AND
DIRECTION MEASURED FROM PROPOSED CENTERLINE

ENERGY CORRECTED STANDARD PENETRATION TEST (SPT) N60-VALUE
(BLOWS PER FOOT); WOR = WEIGHT OF RODS

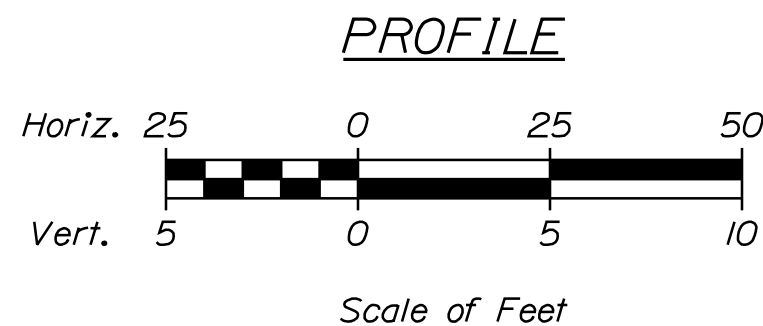
STRATA INTERFACE

PERCENT RECOVERY
PERCENT ROCK QUALITY DESIGNATION (ROD)

DENOTES BOTTOM OF EXPLORATION AT APPROXIMATE DEPTH SHOWN

STRATA SYMBOLS

- FILL
- PAVING
- SILTY SAND
- COBBLES
- BEDROCK
- WELL GRADED SAND
- LOW PLASTICITY CLAY



HALEY
ALDRICH

SHEET NUMBER		OCEANVILLE BRIDGE				PROJ. MANAGER		DATE		STATE OF MAINE	
4	OF 4	OCEANVILLE THOROUGHFARE				DESIGN-DETAILED	E. Force	K. Post	6/23	DEPARTMENT OF TRANSPORTATION	
		STONINGTON HANCOCK COUNTY				CHECKED-REVIEWED	E. Force	K. Post	10/23		
						DESIGN2-DETAILED2	-	-	-	SIGNATURE	
						DESIGN3-DETAILED3	-	-	-		
						REVISIONS 1	-	-	-	P.E. NUMBER	
		INTERPRETIVE SUBSURFACE				REVISIONS 2	-	-	-		
		PROFILE (2 OF 2)				REVISIONS 3	-	-	-		
						REVISIONS 4	-	-	-	DATE	
						FIELD CHANGES	-	-	-		
										BRIDGE NO. 3696	
										WIN	
										22630.00	
										BRIDGE PLANS	

APPENDIX A

Test Boring Logs and Rock Core Photographs

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS						Project: Oceanville Road Over Oceanville Thoroughfare Location: Stonington, Maine		Boring No.: BB-SOT-101																																																																																																																																																																																											
						WIN: 22630.00																																																																																																																																																																																													
Driller: New England Boring Contractors				Elevation (ft.): 12.4		Auger ID/OD: --																																																																																																																																																																																													
Operator: C. Dupuis				Datum: NAVD 88		Sampler: Standard Split Spoon																																																																																																																																																																																													
Logged By: K. Russ				Rig Type: Mobile B53 Truck		Hammer Wt./Fall: 140 lb./30 in.																																																																																																																																																																																													
Date Start/Finish: 3-22-16/3-22-16				Drilling Method: HW drive to 10'; NW spun to 23.8'		Core Barrel: NQ-2.0 in. ID																																																																																																																																																																																													
Boring Location: 107+15, 5.2 ft Rt.				Casing ID/OD: HW-4.0 in. ID; NW-3.0 in. ID		Water Level*: Tidal																																																																																																																																																																																													
Hammer Efficiency Factor: 0.57				Hammer Type: Automatic☒ Hydraulic☐ Rope & Cathead☐																																																																																																																																																																																															
Definitions: D = Split Spoon SampleMD = Unsuccessful Split Spoon Sample AttemptU = Thin Wall Tube SampleMU = Unsuccessful Thin Wall Tube Sample AttemptV = Field Vane Shear Test, PP = Pocket PenetrometerMV = Unsuccessful Field Vane Shear Test Attempt R = Rock Core SampleSSA = Solid Stem AugerHSA = Hollow Stem AugerRC = Roller ConeWOH = Weight of 140lb. HammerWOR/C = Weight of Rods or CasingWO1P = Weight of One Person S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)S_{u(lab)} = Lab Vane Undrained Shear Strength (psf)q_p = Unconfined Compressive Strength (ksf)N-uncorrected = Raw Field SPT N-valueHammer Efficiency Factor = Rig Specific Annual Calibration ValueN₆₀ = SPT N-uncorrected Corrected for Hammer EfficiencyN₆₀ = (Hammer Efficiency Factor/60%) * N-uncorrected T_v = Pocket Torvane Shear Strength (psf)WC = Water Content, percentLL = Liquid LimitPL = Plastic LimitPI = Plasticity IndexG = Grain Size AnalysisC = Consolidation Test																																																																																																																																																																																																			
Sample Information <table><thead><tr><th>Depth (ft.)</th><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>N₆₀</th><th>Casing Blows</th><th>Elevation (ft.)</th><th>Graphic Log</th><th>Visual Description and Remarks</th><th>Laboratory Testing Results/AASHTO and Unified Class.</th></tr></thead><tbody><tr><td>0</td><td></td><td></td><td></td><td></td><td></td><td></td><td>HW Drive</td><td>11.9</td><td rowspan="10"></td><td>Bituminous Concrete</td><td rowspan="10">G#376434 (A-1-b(0)) SM</td></tr><tr><td></td><td>1D</td><td>24/8</td><td>1.0 - 3.0</td><td>11/9/4/3</td><td>13</td><td>12</td><td></td><td></td><td>Brown, dry, medium dense, fine to coarse SAND, some gravel, trace silt, well graded, contains asphalt pieces, (Fill).</td></tr><tr><td></td><td>2D</td><td>24/5</td><td>3.0 - 5.0</td><td>2/2/9/13</td><td>11</td><td>10</td><td></td><td></td><td>Brown, dry, loose, fine to coarse SAND, some gravel, little silt, well graded, contains asphalt, (Fill).</td></tr><tr><td></td><td>3D</td><td>24/2</td><td>5.0 - 7.0</td><td>5/4/6/12</td><td>10</td><td>10</td><td>7</td><td></td><td>Brown, dry, loose, coarse GRAVEL, poorly-graded, (Fill). Note: Two 1.5 in. diameter coarse gravel pieces recovered.</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>23</td><td></td><td></td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>26</td><td></td><td></td></tr><tr><td></td><td>4D</td><td>24/6</td><td>8.0 - 10.0</td><td>96/18/10/10</td><td>28</td><td>27</td><td>18</td><td></td><td>Brown, wet, medium dense, fine to coarse SAND, little silt and gravel, well graded, (Fill).</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>32</td><td></td><td></td></tr><tr><td>10</td><td>5D</td><td>24/6</td><td>10.0 - 12.0</td><td>7/5/8/50(4.0")</td><td>13</td><td>12</td><td>NW</td><td></td><td>Brown, wet, medium dense, fine to coarse SAND, some gravel, little silt, well graded, (Fill). Note: Split spoon refusal at 11.8 ft. Advanced roller bit to 13.5 ft through probable boulder. Note: Probable boulder from 11.8 to 13.5 ft.</td></tr><tr><td></td><td>6D</td><td>24/3</td><td>13.7 - 15.7</td><td>15/18/17/25</td><td>35</td><td>33</td><td></td><td></td><td>Brown, wet, dense, fine to coarse SAND, trace silt and gravel, well graded, (Fill)</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>Note: Driller action indicates frequent cobbles and boulders from 15.7 to 19.4 ft.</td></tr><tr><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>-7.0</td><td>Note: Stratum change at 19.4 ft based on drill wash.</td></tr><tr><td>20</td><td>7D</td><td>24/7</td><td>20.0 - 22.0</td><td>18/15/16/9</td><td>31</td><td>29</td><td></td><td></td><td>Olive-grey, moist, medium dense, fine to coarse SAND, some silt, trace gravel, moderately bonded, well graded, (Glacial Till).</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>R1</td><td>60/60</td><td>23.8 - 28.8</td><td>RQD = 90%</td><td></td><td></td><td></td><td>-10.6</td><td>Top of Bedrock at El. -10.6 Note: Spun N-casing to 23.8 ft, began NQ rock core. R1: Pink to light grey, fine to coarse grained GRANITE. Very hard, fresh. Primary joints dipping at moderate angles, moderately</td></tr><tr><td>25</td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td></td><td>R1: qp=2,923 ksf</td></tr></tbody></table>										Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.	0							HW Drive	11.9		Bituminous Concrete	G#376434 (A-1-b(0)) SM		1D	24/8	1.0 - 3.0	11/9/4/3	13	12			Brown, dry, medium dense, fine to coarse SAND, some gravel, trace silt, well graded, contains asphalt pieces, (Fill).		2D	24/5	3.0 - 5.0	2/2/9/13	11	10			Brown, dry, loose, fine to coarse SAND, some gravel, little silt, well graded, contains asphalt, (Fill).		3D	24/2	5.0 - 7.0	5/4/6/12	10	10	7		Brown, dry, loose, coarse GRAVEL, poorly-graded, (Fill). Note: Two 1.5 in. diameter coarse gravel pieces recovered.								23										26				4D	24/6	8.0 - 10.0	96/18/10/10	28	27	18		Brown, wet, medium dense, fine to coarse SAND, little silt and gravel, well graded, (Fill).								32			10	5D	24/6	10.0 - 12.0	7/5/8/50(4.0")	13	12	NW		Brown, wet, medium dense, fine to coarse SAND, some gravel, little silt, well graded, (Fill). Note: Split spoon refusal at 11.8 ft. Advanced roller bit to 13.5 ft through probable boulder. Note: Probable boulder from 11.8 to 13.5 ft.		6D	24/3	13.7 - 15.7	15/18/17/25	35	33			Brown, wet, dense, fine to coarse SAND, trace silt and gravel, well graded, (Fill)										Note: Driller action indicates frequent cobbles and boulders from 15.7 to 19.4 ft.									-7.0	Note: Stratum change at 19.4 ft based on drill wash.	20	7D	24/7	20.0 - 22.0	18/15/16/9	31	29			Olive-grey, moist, medium dense, fine to coarse SAND, some silt, trace gravel, moderately bonded, well graded, (Glacial Till).																						R1	60/60	23.8 - 28.8	RQD = 90%				-10.6	Top of Bedrock at El. -10.6 Note: Spun N-casing to 23.8 ft, began NQ rock core. R1: Pink to light grey, fine to coarse grained GRANITE. Very hard, fresh. Primary joints dipping at moderate angles, moderately	25											R1: qp=2,923 ksf
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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS							Project: Oceanville Road Over Oceanville Thoroughfare Location: Stonington, Maine			Boring No.: BB-SOT-101 WIN: 22630.00					
Driller: New England Boring Contractors				Elevation (ft.): 12.4				Auger ID/OD: --							
Operator: C. Dupuis				Datum: NAVD 88				Sampler: Standard Split Spoon							
Logged By: K. Russ				Rig Type: Mobile B53 Truck				Hammer Wt./Fall: 140 lb./30 in.							
Date Start/Finish: 3-22-16/3-22-16				Drilling Method: HW drive to 10'; NW spun to 23.8'				Core Barrel: NQ-2.0 in. ID							
Boring Location: 107+15, 5.2 ft Rt.				Casing ID/OD: HW-4.0 in. ID; NW-3.0 in. ID				Water Level*: Tidal							
Hammer Efficiency Factor: 0.57				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐											
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt				R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140 lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person				Su = Peak/Remolded Field Vane Undrained Shear Strength (psf) Su(lab) = Lab Vane Undrained Shear Strength (psf) qp = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N60 = SPT N-uncorrected Corrected for Hammer Efficiency N60 = (Hammer Efficiency Factor/60%)*N-uncorrected				Tv = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test			
Depth (ft.)	Sample Information									Graphic Log	Laboratory Testing Results/AASHTO and Unified Class.				
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N60	Casing Blows	Elevation (ft.)	Visual Description and Remarks						
25										close, tight, occasionally healed, with hematite coatings on joint surfaces. Secondary joint sets dipping at low angles, moderately close, tight to open, smooth to rough and undulating, occasional silt infilling. Recovery=100% Rock Mass Quality=Good -STONINGTON GRANITE- R1 Core Times (min:sec): 23.8-24.8' (2:32); 24.8-25.8' (2:32); 25.8-26.8' (3:55); 26.8-27.8' (5:30); 27.8-28.8' (5:29) R2: Pink to light grey, fine to coarse grained GRANITE. Very hard, fresh. Primary joints dipping at moderate angles, moderately close, tight, occasionally healed, with hematite coatings on joint surfaces. Secondary joint sets dipping at low angles, moderately close, tight to open, smooth to rough and undulating, occasional silt infilling. Recovery=100% Rock Mass Quality=Excellent -STONINGTON GRANITE- R2 Core Times (min:sec): 28.8-29.8' (4:02); 29.8-30.8' (4:51); 30.8-31.8' (3:10); 31.8-32.8' (2:40); 32.8-33.8' (3:32) Bottom of Exploration at 33.8 feet below ground surface.					
	R2	60/60	28.8 - 33.8	RQD = 97%											
Remarks:															
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.										Page 2 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.: BB-SOT-101					

[illegible]

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Oceanville Road Over Oceanville Thoroughfare Location: Stonington, Maine				Boring No.: BB-SOT-102 WIN: 22630.00			
Driller: New England Boring Contractors				Elevation (ft.): 12.2				Auger ID/OD: --			
Operator: C. Dupuis				Datum: NAVD 88				Sampler: Standard Split Spoon			
Logged By: K. Russ				Rig Type: Mobile B53 Truck				Hammer Wt./Fall: 140 lb./30 in.			
Date Start/Finish: 3-23-16/3-23-16				Drilling Method: HW-4.0 in. ID; NW-3.0 in. ID				Core Barrel: NQ-2.0 in. ID			
Boring Location: 106+31, 6.9 ft Rt.				Casing ID/OD: HW-4.0 in. ID; NW-3.0 in. ID				Water Level*: Tidal			
Hammer Efficiency Factor: 0.57				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐							
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140 lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S_{u(lab)} = Lab Vane Undrained Shear Strength (psf) q_p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected T_y = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test											
Depth (ft.)	Sample Information								Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (16 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	Elevation (ft.)			
25	R1	60/60	25.0 - 30.0	RQD = 42%						Top of Bedrock at El. -12.3 Note: Split spoon refusal at 24.5 ft. Advanced NW casing to 25.0 ft. R1: Pinkish-red to pinkish-grey, fine to coarse grained GRANITE. Hard to very hard, slightly weathered. Primary joints dipping at moderately to high angles, wide spacing, open, silt infilling, smooth joint surfaces. Secondary joints low angle to horizontal, close to moderately close, open, joint surfaces rough and undulating, occasional silt infilling. Recovery=100% Rock Mass Quality=Poor -STONINGTON GRANITE- R1 Core Times (min:sec): 25.0-26.0' (2:48); 26.0-27.0' (4:15); 27.0-28.0' (6:03); 28.0-29.0' (6:03); 29.0-30.0' (7:22) R2: Pink to light grey, fine to coarse grained GRANITE. Very hard, fresh to slightly weathered. Primary joints moderately dipping, moderately spaced, open, smooth to rough, weathered joint surfaces. Secondary joints horizontal to low angle, very close to close, open, rough, weathered joint surfaces. Recovery=100% Rock Mass Quality=Fair -STONINGTON GRANITE- R2 Core Times (min:sec): 30.0-31.0' (5:15); 31.0-32.0' (5:35); 32.0-33.0' (4:18); 33.0-34.0' (6:20); 34.0-35.0' (4:45) R3: Dark red to reddish-grey, fine to coarse grained GRANITE. Hard, slightly to moderately weathered. Primary joints moderately dipping to high angle, close, open, rough and undulating, frequent silt infilling, occasional chlorite coatings on joint surfaces. Secondary joints low angle, close, open, rough, silt infilling. Recovery=100% Rock Mass Quality=Very Poor -STONINGTON GRANITE- R3 Core Times (min:sec): 35.0-36.0' (4:51) 36.0-37.0' (9:51); 37.0-38.0' (9:38); 38.0-39.0' (8:41); 39.0-40.0' (8:22) Bottom of Exploration at 40.0 feet below ground surface.	R2: qp=2,544 ksf
30	R2	60/60	30.0 - 35.0	RQD = 75%							
35	R3	60/60	35.0 - 40.0	RQD = 22%							
40											
45											
50											
Remarks:											
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.										Page 2 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.: BB-SOT-102	

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Oceanville Road Over Oceanville Thoroughfare, Bridge No. 3696 Location: Stonington, Maine		Boring No.: BB-SOT-201				
Driller: New England Boring Contractors				Elevation (ft.): 11.8		Auger ID/OD: --				
Operator: M. Porter				Datum: NAVD 88		Sampler: Standard Split Spoon				
Logged By: M. Snow/J. Ilunga				Rig Type: Mobile B-53 Track		Hammer Wt./Fall: 140#/30"; 300#/24"				
Date Start/Finish: 6-7-22/6-7-22				Drilling Method: NW Drive		Core Barrel: NQ-2.0 in. ID				
Boring Location: 107+22, 6.9 ft L.L.				Casing ID/OD: NW-3.0 in. ID		Water Level*: 10.0 ft				
Hammer Efficiency Factor: 0.922				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐						
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S_{u(lab)} = Lab Vane Undrained Shear Strength (psf) q_p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected T_v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test										
Depth (ft.)	Sample Information							Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows			
0							SSA	11.1		
	1D	24/10	1.0 - 3.0	8/6/4/4	10	15				
5										
	2D	24/6	5.0 - 7.0	8/22/3/7	25	38	23			
							50			
							94			
							130			
							190			
10										
	3D	24/10	10.0 - 12.0	5/10/10/11	20	31	79	1.8		
							37			
							54			
							60			
							36			
15										
	4D	24/23	15.0 - 17.0	3/4/5/7	9	14	29	-1.7		
							39			
	5D/A	24/24	17.0 - 19.0	1/2/2/3	4	6	42			
							34			
							47			
20										
	6D	24/4	20.0 - 22.0	5/6/10/22	16	25	34	-6.6		
							29			
							35			
							36			
25							55			
Remarks:										
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.

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Boring No.: BB-SOT-201

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Oceanville Road Over Oceanville Thoroughfare, Bridge No. 3696 Location: Stonington, Maine				Boring No.: BB-SOT-201																																																																																																
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Boring Location: 107+22, 6.9 ft Lt.				Casing ID/OD: NW-3.0 in. ID				Water Level*: 10.0 ft																																																																																																
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R1: Pink to grey, medium to coarse-grained GRANITE, hard to very hard, fresh to slightly weathered. Joints dipping at moderate to high angles, very close to close, tight to open, slightly weathered joint surfaces. Rock Quality=Fair -STONINGTON GRANITE- Recovery=100% R1 Core Times (min:sec): 25.5-26.5' (1:27); 26.5-27.5' (1:58); 27.5-28.5' (2:17); 28.5-29.5' (2:38); 29.5-30.5' (3:40) R2: Similar to R1, except joints horizontal to lower angles. Rock Quality=Good -STONINGTON GRANITE- Recovery=100% R2 Core Times (min:sec): 30.5-31.5' (2:58); 31.5-32.5' (1:43); 32.5-33.5' (2:46); 33.5-34.5' (3:17); 34.5-35.5' (3:26)</td></tr><tr><td>30</td><td>R2</td><td>60/60</td><td>30.5 - 35.5</td><td>RQD = 82%</td><td></td><td></td><td></td><td></td><td>R3: Similar to R1, except joints are tight from 39.6 to 40.5 ft, frequent joints, very close spacing. Rock Quality=Fair -STONINGTON GRANITE- Recovery=98% R3 Core Times (min:sec): 35.5-36.5' (3:08); 36.5-37.5' (2:46); 37.5-38.5' (2:57); 38.5-39.5' (2:40); 39.5-40.5' (3:04)</td></tr><tr><td>35</td><td>R3</td><td>60/59</td><td>35.5 - 40.5</td><td>RQD = 73%</td><td></td><td></td><td></td><td></td><td>R4: Pink to grey, medium to coarse-grained GRANITE. Hard to very hard, fresh to slightly weathered. Joints dipping at horizontal to low angles, very close to close, tight to open, slightly weathered joint surfaces. Rock Quality=Good -STONINGTON GRANITE- Recovery=100% R4 Core Times (min:sec): 40.5-41.5' (4:01); 41.5-42.5' (3:42); 42.5-43.5' (4:21); 43.5-44.5' (6:47); 44.5-45.5' (2:25)</td></tr><tr><td>40</td><td>R4</td><td>60/60</td><td>40.5 - 45.5</td><td>RQD = 83%</td><td></td><td></td><td></td><td></td><td>R5: Similar to R1, except highly weathered zone at 49.7 ft (49.7 to 49.8 ft decomposed to sand). Rock Quality=Good -STONINGTON GRANITE- Recovery=100% R5 Core Times (min:sec): 45.5-46.5' (3:23); 46.5-47.5' (3:09); 47.5-48.5' (2:57); 48.5-49.5' (2:38); 49.5-50.5' (2:16)</td></tr><tr><td>45</td><td>R5</td><td>60/60</td><td>45.5 - 50.5</td><td>RQD = 87%</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><td></td><td></td></tr></table>										Depth (ft.)	Sample Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (16 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows	25	7D R1	4/4 60/60	25.0 - 25.3 25.5 - 30.5	30/4" RQD = 65%			RC NQ CORE	-13.2 -13.5		Brown, wet, fine to coarse SAND, little gravel, little silt, well bonded, (Glacial Till).	qp=3,176 ksf (32.39'-32.77') qp=3,317 ksf (36.76'-37.13')										Top of Bedrock at El. -13.5 Note: Advanced roller bit to 25.5 ft. R1: Pink to grey, medium to coarse-grained GRANITE, hard to very hard, fresh to slightly weathered. Joints dipping at moderate to high angles, very close to close, tight to open, slightly weathered joint surfaces. Rock Quality=Fair -STONINGTON GRANITE- Recovery=100% R1 Core Times (min:sec): 25.5-26.5' (1:27); 26.5-27.5' (1:58); 27.5-28.5' (2:17); 28.5-29.5' (2:38); 29.5-30.5' (3:40) R2: Similar to R1, except joints horizontal to lower angles. 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Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Oceanville Road Over Oceanville Thoroughfare, Bridge No. 3696 Location: Stonington, Maine		Boring No.: BB-SOT-201 WIN: 22630.00					
Driller: New England Boring Contractors			Elevation (ft.): 11.8		Auger ID/OD: --						
Operator: M. Porter			Datum: NAVD 88		Sampler: Standard Split Spoon						
Logged By: M. Snow/J. Ilunga			Rig Type: Mobile B-53 Track		Hammer Wt./Fall: 140#/30"; 300#/24"						
Date Start/Finish: 6-7-22/6-7-22			Drilling Method: NW Drive		Core Barrel: NQ-2.0 in. ID						
Boring Location: 107+22, 6.9 ft Lt.			Casing ID/OD: NW-3.0 in. ID		Water Level*: 10.0 ft						
Hammer Efficiency Factor: 0.922			Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐								
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt			R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140 lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person S _u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _u (lab) = Lab Vane Undrained Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected T _y = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test								
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	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (1/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				
50								-38.7		Bottom of Exploration at 50.5 feet below ground surface.	
55											
60											
65											
70											
75											
Remarks:											
Stratification lines represent approximate boundaries between soil types; transitions may be gradual.										Page 3 of 3	
* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.										Boring No.: BB-SOT-201	

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Oceanville Road Over Oceanville Thoroughfare, Bridge No. 3696 Location: Stonington, Maine		Boring No.: BB-SOT-202 WIN: 22630.00	
Driller: New England Boring Contractors		Elevation (ft.): 11.7		Auger ID/OD: --			
Operator: M. Porter		Datum: NAVD 88		Sampler: Standard Split Spoon			
Logged By: M. Snow/J. Ilunga		Rig Type: Mobile B-53 Track		Hammer Wt./Fall: 140#/30"; 300#/24"			
Date Start/Finish: 6-8-22/6-9-22		Drilling Method: NW Drive		Core Barrel: NQ-2.0 in. ID			
Boring Location: 106+35, 5.5 ft L.L.		Casing ID/OD: NW-3.0 in. ID		Water Level*: 9.7 ft			
Hammer Efficiency Factor: 0.922		Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐					
Definitions: D = Split Spoon Sample MD = Unsuccessful Split Spoon Sample Attempt U = Thin Wall Tube Sample MU = Unsuccessful Thin Wall Tube Sample Attempt V = Field Vane Shear Test, PP = Pocket Penetrometer MV = Unsuccessful Field Vane Shear Test Attempt R = Rock Core Sample SSA = Solid Stem Auger HSA = Hollow Stem Auger RC = Roller Cone WOH = Weight of 140lb. Hammer WOR/C = Weight of Rods or Casing WO1P = Weight of One Person S _u = Peak/Remolded Field Vane Undrained Shear Strength (psf) S _{u(lab)} = Lab Vane Undrained Shear Strength (psf) q _p = Unconfined Compressive Strength (ksf) N-uncorrected = Raw Field SPT N-value Hammer Efficiency Factor = Rig Specific Annual Calibration Value N ₆₀ = SPT N-uncorrected Corrected for Hammer Efficiency N ₆₀ = (Hammer Efficiency Factor/60%)*N-uncorrected T _v = Pocket Torvane Shear Strength (psf) WC = Water Content, percent LL = Liquid Limit PL = Plastic Limit PI = Plasticity Index G = Grain Size Analysis C = Consolidation Test							

Depth (ft.)	Sample Information							Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/AASHTO and Unified Class.
	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N ₆₀	Casing Blows				
0							SSA	11.4		Bituminous Concrete	
	1D	24/12	1.0 - 3.0	8/9/10/13	19	29				Brown, dry, medium dense, fine to coarse SAND, some gravel, trace silt, (Fill).	
5	2D	24/7	5.0 - 7.0	11/9/6/4	15	23	28			Brown, dry, medium dense, fine to coarse SAND, little gravel, trace silt, (Fill).	
							44				
10	3D/A	24/20	10.0 - 12.0	WOH/WOH/1/2	1	2	-	2.0	3D: Grey-brown, wet, very loose, Silty SAND, little coarse sand, trace fine gravel, trace shells, (Harbor Bottom Deposit).		
								0.6	3D/A: Grey, wet, very soft, organic SILT, strong organic odor, (Harbor Bottom Deposit). Similar to 3D/A		
	4D	24/7	12.0 - 14.0	WOH/WOH/1(12")			-		Note: Advanced NW casing after washing ahead to 15.0 ft.		
							21				
15	5D/A	24/3	15.0 - 17.0	WOH/3/5/5	8	12	33	-4.1	5D: Grey, wet, medium dense, Silty GRAVEL, trace fine to coarse sand, (Harbor Bottom Deposit).		
									5D/A: Grey, wet, loose, Silty GRAVEL, some fine to coarse sand, (Harbor Bottom Deposit).		
									Note: 5D: recovered wash sample, redrove spoon to collect 5D/A.		
20	6D/A	24/18	20.0 - 22.0	3/2/4/8	6	9	-	-8.3	6D: Grey, wet, Silty CLAY, trace fine gravel, trace fine to medium sand, (Marine Deposit).		
								-9.3	6D/A: Grey, wet, fine to medium SAND, some silt, little coarse sand, trace fine gravel, (Glacial Till).		
25								-12.8			

Remarks:

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.

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Boring No.: BB-SOT-202

Maine Department of Transportation Soil/Rock Exploration Log US CUSTOMARY UNITS				Project: Oceanville Road Over Oceanville Thoroughfare, Bridge No. 3696 Location: Stonington, Maine				Boring No.: BB-SOT-202 WIN: 22630.00																						
Driller: New England Boring Contractors				Elevation (ft.): 11.7				Auger ID/OD: --																						
Operator: M. Porter				Datum: NAVD 88				Sampler: Standard Split Spoon																						
Logged By: M. Snow/J. Ilunga				Rig Type: Mobile B-53 Track				Hammer Wt./Fall: 140#/30"; 300#/24"																						
Date Start/Finish: 6-8-22/6-9-22				Drilling Method: NW Drive				Core Barrel: NQ-2.0 in. ID																						
Boring Location: 106+35, 5.5 ft Lt.				Casing ID/OD: NW-3.0 in. ID				Water Level*: 9.7 ft																						
Hammer Efficiency Factor: 0.922				Hammer Type: Automatic ☒ Hydraulic ☐ Rope & Cathead ☐																										
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25	R1	48/48	25.0 - 29.0	RQD = 67%			RC NQ CORE				Top of Bedrock El. -12.8 Note: Advanced roller bit to 25.0 ft. R1: Pink to grey, medium to coarse-grained GRANITE. Very hard, fresh to slightly weathered. Joints dipping from moderate to high angles, very close to close, tight to open. Rock Quality=Fair -STONINGTON GRANITE- Recovery=100% R1 Core Times (min:sec): 25.0-26.0' (2:17); 26.0-27.0' (1:54); 27.0-28.0' (1:56); 28.0-29.0' (4:13) R2: Similar to R1, except secondary joints horizontal and vertical, close to moderately close, tight to open, highly fractured zone from approximately 32.1 to 33.0 ft. Rock Quality=Fair -STONINGTON GRANITE- Recovery=100% R2 Core Times (min:sec): 29.0-30.0' (2:55); 30.0-31.0' (1:29); 31.0-32.0' (2:21); 32.0-33.0' (1:37) R3: Similar to R1, except secondary vertical and low angle joints, moderately close, tight to open. Rock Quality=Good -STONINGTON GRANITE- Recovery=97% R3 Core Times (min:sec): 33.0-34.0' (2:16); 34.0-35.0' (2:18); 35.0-36.0' (2:46); 36.0-37.0' (2:15); 37.0-38.0' (2:18)	qp=3,422 ksf (29.07'-29.47')																		
30	R2	48/48	29.0 - 33.0	RQD = 67%																										
35	R3	60/58	33.0 - 38.0	RQD = 88%																										
40	R4	60/60	38.0 - 43.0	RQD = 77%							R4: Similar to R1, except primary joints dipping from moderate to horizontal angles. Secondary high angle joint at approximately 42.5 ft, very close to moderately close, tight to open. Rock Quality=Good -STONINGTON GRANITE- Recovery=100% R4 Core Times (min:sec): 38.0-39.0' (5:21); 39.0-40.0' (2:52); 40.0-41.0' (3:21); 41.0-42.0' (2:22); 42.0-43.0' (2:30)	qp=3,473 ksf (39.44'-39.83')																		
45	R5	60/61	43.0 - 48.0	RQD = 92%							R5: Similar to R1, except joints close to moderately close, tight to open, two high angle healed joints. Secondary vertical joint at approximately 47.5 ft. Rock Quality=Excellent -STONINGTON GRANITE- Recovery=101% R5 Core Times (min:sec): 43.0-44.0' (2:14); 44.0-45.0' (4:28); 45.0-46.0' (1:38); 46.0-47.0' (2:04); 47.0-48.0' (2:30)																			
50	R6	24/14	48.0 - 50.0	RQD = 58%							R6: Similar to R1, except joints dipping at low to high angles, close, open. Rock Quality=Fair -STONINGTON GRANITE-																			
Remarks:																														
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.																														
										Page 2 of 3																				
Boring No.: BB-SOT-202																														

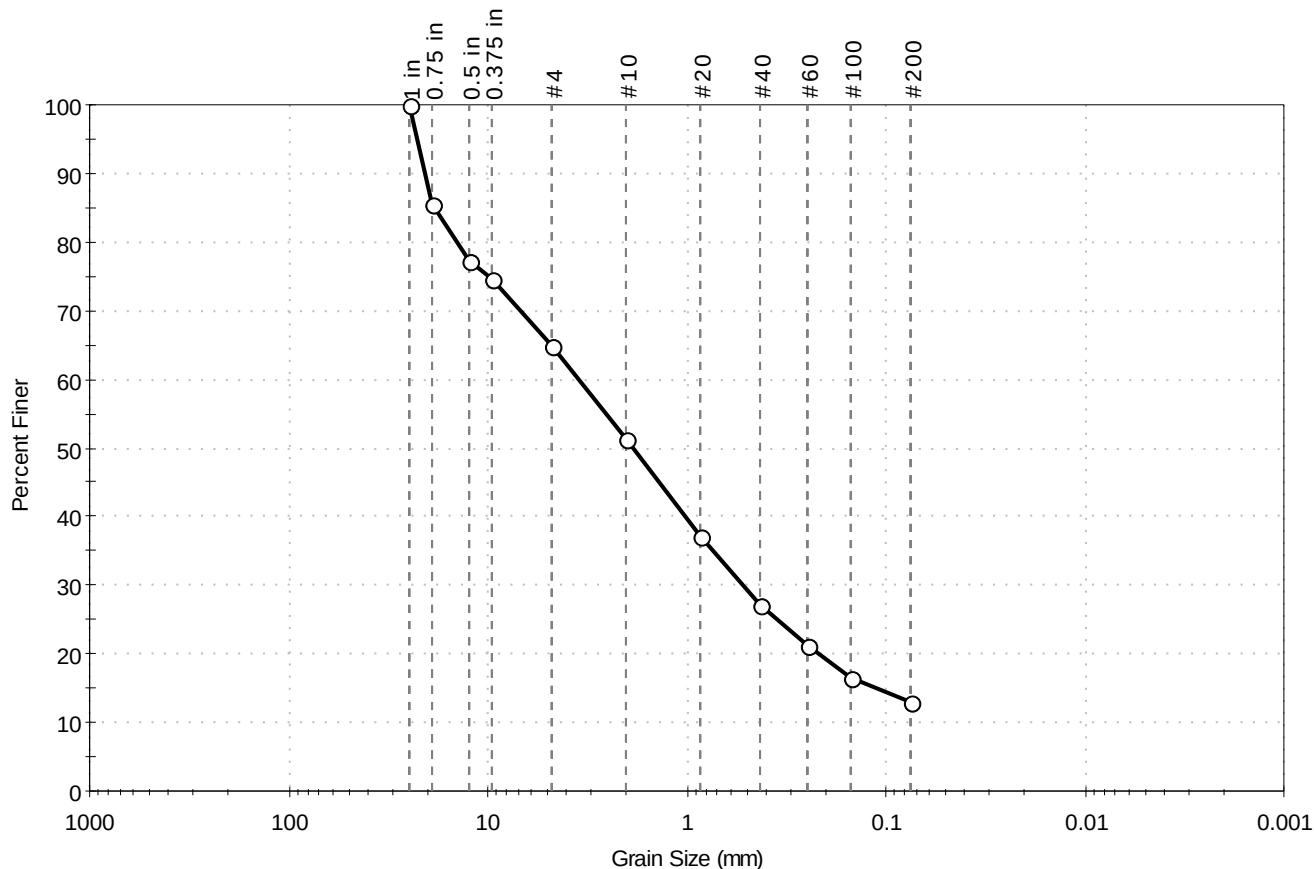
[illegible]

APPENDIX B

Laboratory Test Results

Client:	Haley & Aldrich, Inc.		
Project:	Oceanville Rd. Over Oceanville Thoroughfare		
Location:	Stonington, ME	Project No:	GTX-304692
Boring ID:	BB-SOT-101	Sample Type:	bag
Sample ID:	---	Test Date:	05/10/16
Depth :	3.0-5.0 ft	Test Id:	376434
Test Comment:	---		
Visual Description:	Moist, brown silty sand with gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	35.1	51.9	13.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	85		
0.5 in	12.50	77		
0.375 in	9.50	75		
#4	4.75	65		
#10	2.00	51		
#20	0.85	37		
#40	0.42	27		
#60	0.25	21		
#100	0.15	17		
#200	0.075	13		

Coefficients

D ₈₅ = 18.5663 mm	D ₃₀ = 0.5139 mm
D ₆₀ = 3.4884 mm	D ₁₅ = 0.1096 mm
D ₅₀ = 1.8562 mm	D ₁₀ = N/A
C _u = N/A	C _c = N/A

Classification

ASTM N/A

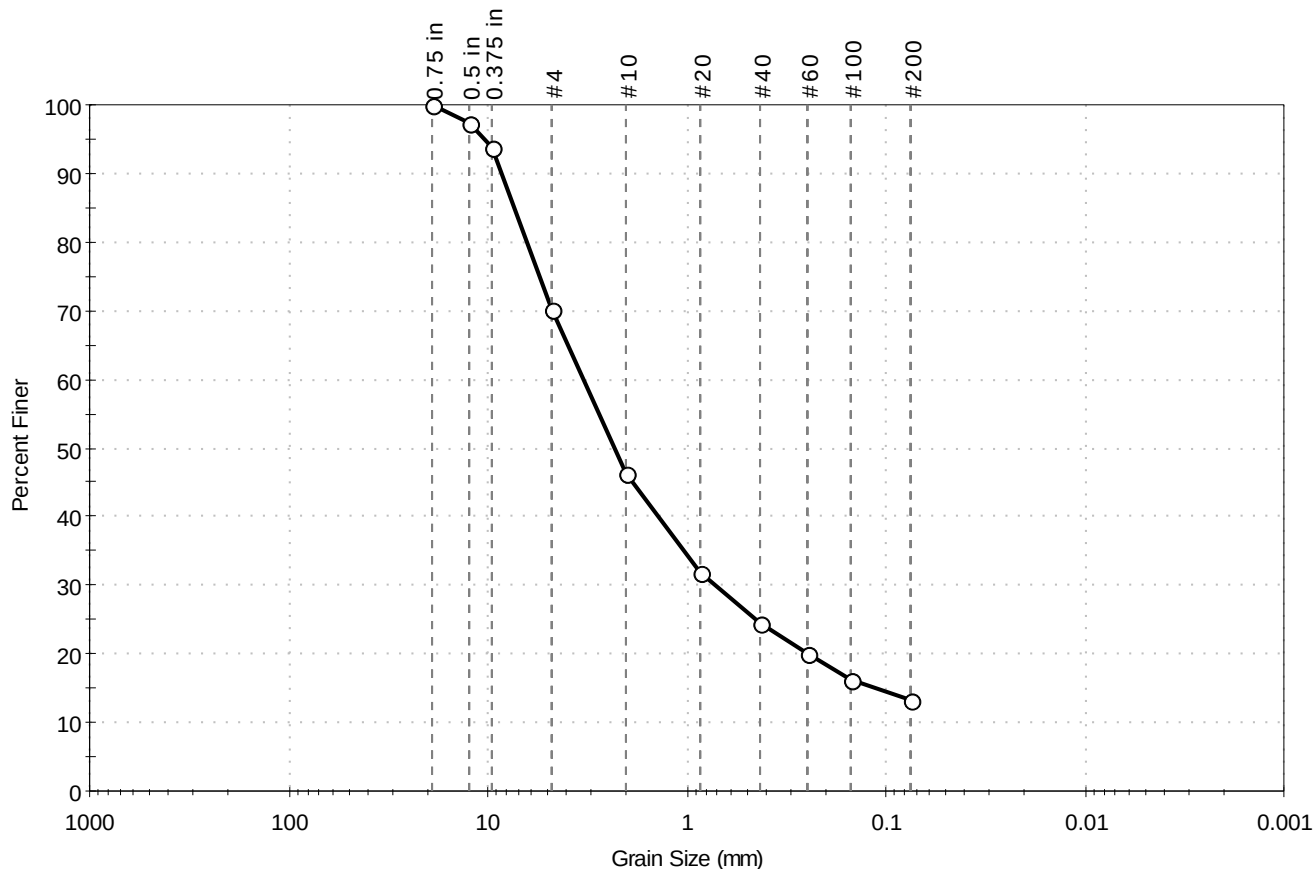
AASHTO Stone Fragments, Gravel and Sand (A-1-b (0))

Sample / Test Description

Sand/Gravel Particle Shape : **ROUNDED**
 Sand/Gravel Hardness : **HARD**

Client:	Haley & Aldrich, Inc.		
Project:	Oceanville Rd. Over Oceanville Thoroughfare		
Location:	Stonington, ME	Project No:	GTX-304692
Boring ID:	BB-SOT-101	Sample Type:	bag
Sample ID:	---	Test Date:	05/10/16
Depth :	10.0-12.0 ft	Test Id:	376435
Test Comment:	---		
Visual Description:	Moist, brown silty sand with gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	29.9	57.0	13.1

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	97		
0.375 in	9.50	94		
#4	4.75	70		
#10	2.00	46		
#20	0.85	32		
#40	0.42	24		
#60	0.25	20		
#100	0.15	16		
#200	0.075	13		

Coefficients

D ₈₅ = 7.3533 mm	D ₃₀ = 0.7171 mm
D ₆₀ = 3.2944 mm	D ₁₅ = 0.1159 mm
D ₅₀ = 2.2945 mm	D ₁₀ = N/A
C _u = N/A	C _c = N/A

Classification

ASTM N/A

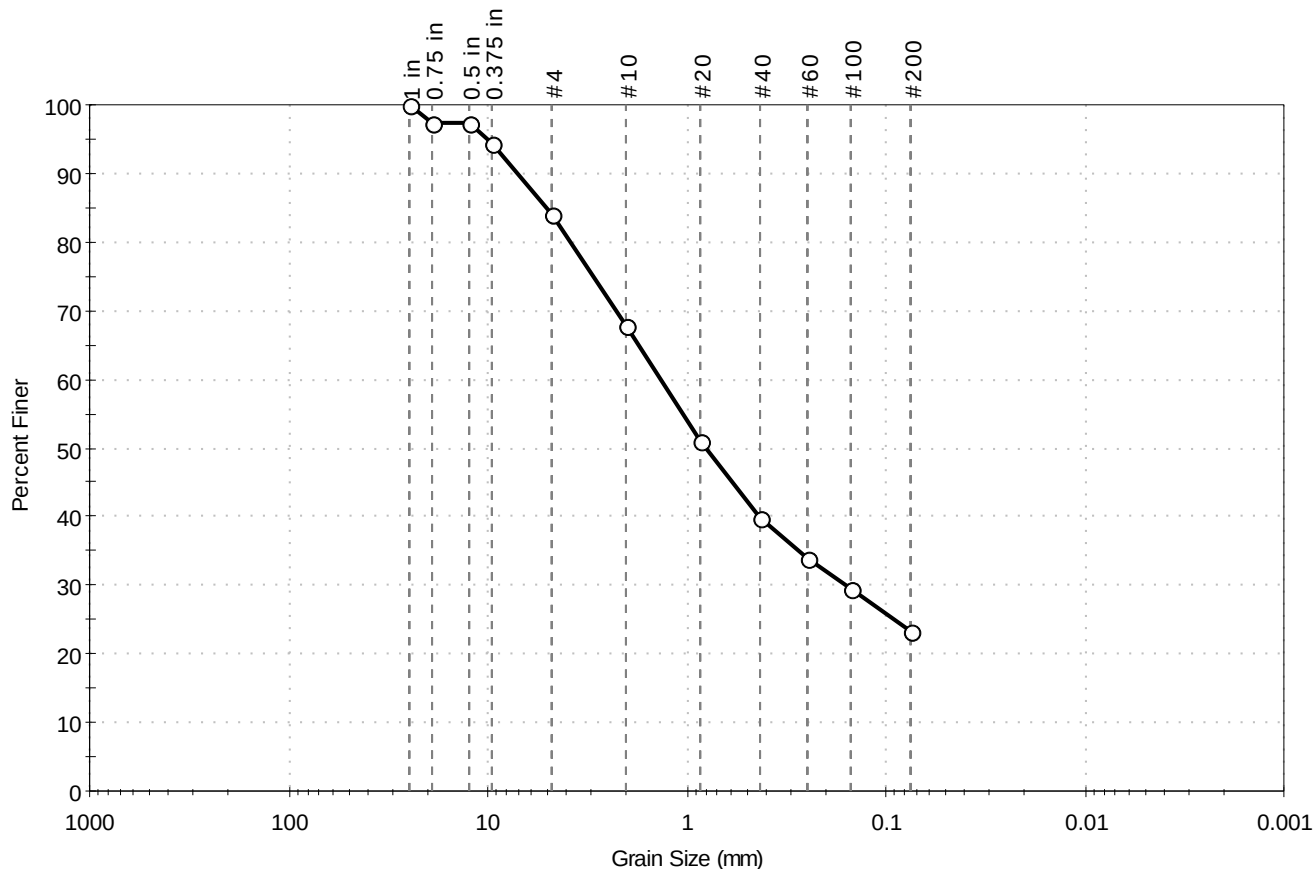
AASHTO Stone Fragments, Gravel and Sand (A-1-a (0))

Sample / Test Description

Sand/Gravel Particle Shape : ANGULAR
Sand/Gravel Hardness : HARD

Client:	Haley & Aldrich, Inc.		
Project:	Oceanville Rd. Over Oceanville Thoroughfare		
Location:	Stonington, ME	Project No:	GTX-304692
Boring ID:	BB-SOT-102	Sample Type:	bag
Sample ID:	---	Test Date:	05/10/16
Depth :	1.0-3.0 ft	Test Id:	376436
Test Comment:	---		
Visual Description:	Moist, brown silty sand with gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	15.9	60.8	23.3

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
1 in	25.00	100		
0.75 in	19.00	97		
0.5 in	12.50	97		
0.375 in	9.50	94		
#4	4.75	84		
#10	2.00	68		
#20	0.85	51		
#40	0.42	40		
#60	0.25	34		
#100	0.15	30		
#200	0.075	23		

Coefficients

D ₈₅ = 5.0506 mm	D ₃₀ = 0.1586 mm
D ₆₀ = 1.3452 mm	D ₁₅ = N/A
D ₅₀ = 0.7999 mm	D ₁₀ = N/A
C _u = N/A	C _c = N/A

Classification

ASTM N/A

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

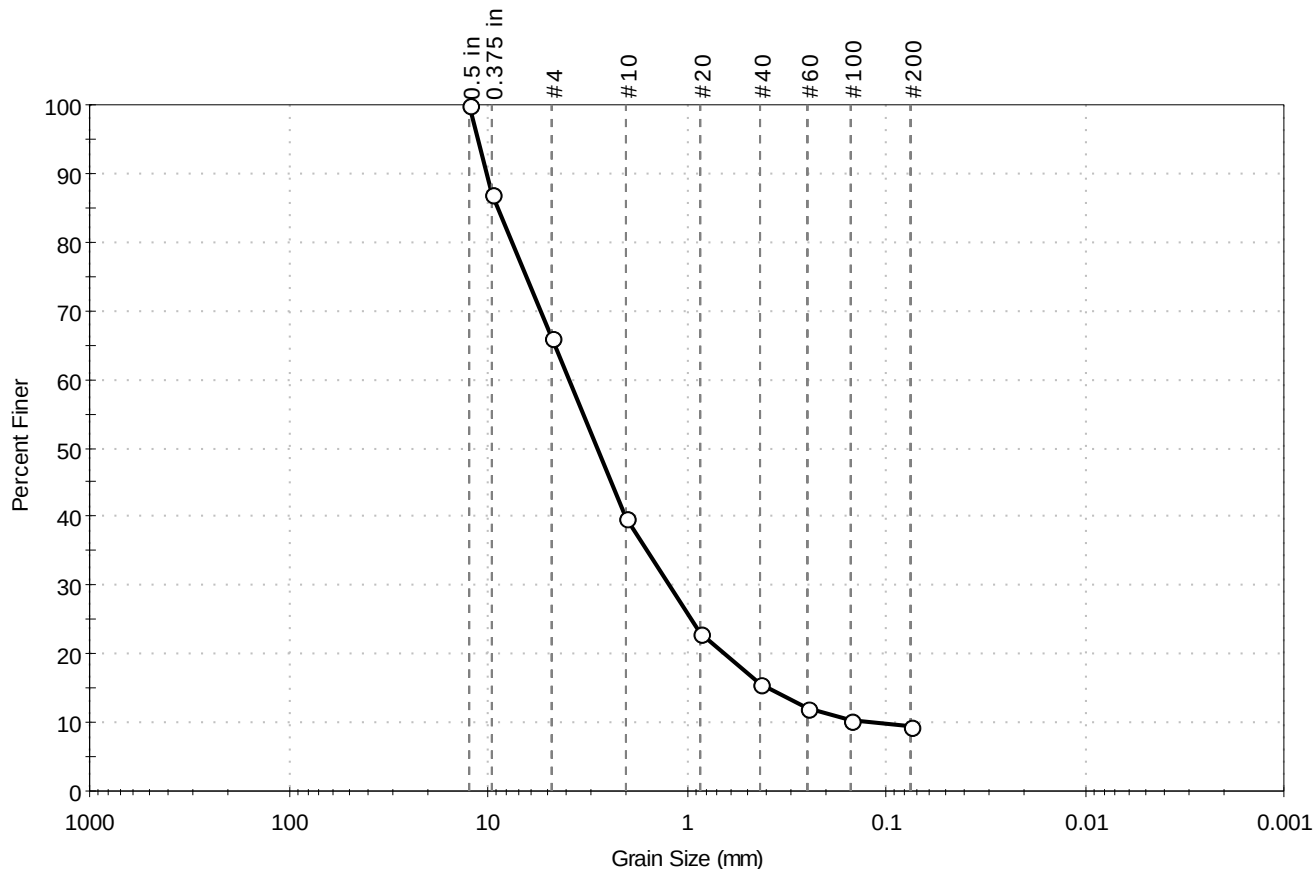
Sample / Test Description

Sand/Gravel Particle Shape : **ROUNDED**
 Sand/Gravel Hardness : **HARD**



Client:	Haley & Aldrich, Inc.		
Project:	Oceanville Rd. Over Oceanville Thoroughfare		
Location:	Stonington, ME	Project No:	GTX-304692
Boring ID:	BB-SOT-102	Sample Type:	bag
Sample ID:	---	Test Date:	05/10/16
Depth :	15.0-17.0 ft	Test Id:	376437
Test Comment:	---		
Visual Description:	Moist, gray sand with silt and gravel		
Sample Comment:	---		

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	33.9	56.7	9.4

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.5 in	12.50	100		
0.375 in	9.50	87		
#4	4.75	66		
#10	2.00	40		
#20	0.85	23		
#40	0.42	16		
#60	0.25	12		
#100	0.15	10		
#200	0.075	9.4		

Coefficients

D ₈₅ = 8.8596 mm	D ₃₀ = 1.2087 mm
D ₆₀ = 3.8803 mm	D ₁₅ = 0.3923 mm
D ₅₀ = 2.7893 mm	D ₁₀ = 0.1108 mm
C _u = 35.021	C _c = 3.398

Classification

ASTM N/A

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

Sample / Test Description

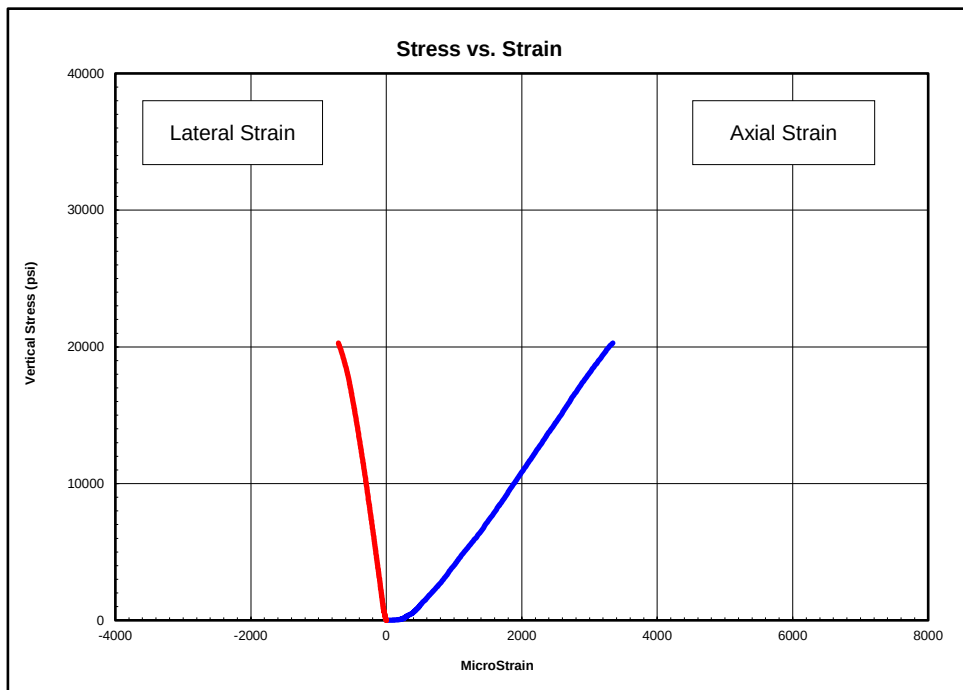
Sand/Gravel Particle Shape : ROUNDED

Sand/Gravel Hardness : HARD



Client:	Haley & Aldrich, Inc.
Project Name:	Oceanville Rd. Over Oceanville Thoroughfare
Project Location:	Stonington, ME
GTX #:	304692
Test Date:	5/10/2016
Tested By:	daa
Checked By:	jsc
Boring ID:	BB-SOT-101
Sample ID:	R1
Depth, ft:	24.24-24.61
Sample Type:	rock core
Sample Description:	See photographs Intact material failure Diameter < 10 times maximum particle size

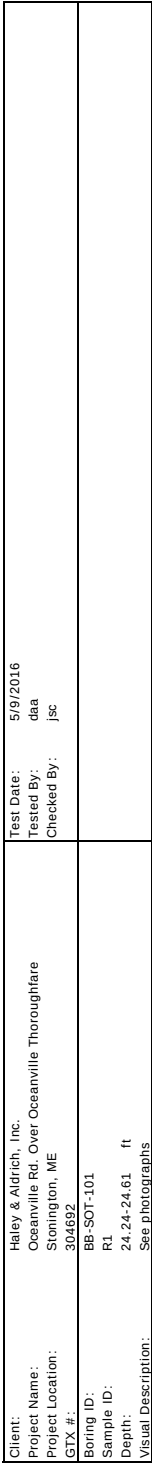
Compressive Strength and Elastic Moduli of Rock by ASTM D7012 - Method D



Peak Compressive Stress: 20,296 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
2000-7400	6,330,000	0.17
7400-12800	7,270,000	0.22
12800-18300	7,310,000	0.26

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.



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	
Specimen Length, in:	4.38	4.38	
Specimen Diameter, in:	1.99	1.99	
Specimen Mass, g:	587.99	587.99	
Bulk Density, lb/ft³	164	164	
Length to Diameter Ratio:	2.2	2.2	
Minimum Diameter Tolerance Met?		YES	
Length to Diameter Ratio Tolerance Met?		YES	
END FLATNESS AND PARALLELISM (Procedure FP1)			
END 1	-0.875	-0.750	
Diameter 1, in	0.00000	0.00000	
Diameter 2, in (rotated 90°)	0.00000	0.00000	
END 2	-0.875	-0.750	
Diameter 1, in	0.00000	0.00000	
Diameter 2, in (rotated 90°)	0.00010	0.00010	
Straightness Tolerance Met?			
Maximum difference must be < 0.020 in.			
Difference between max and min readings, in:			
0° = 0.00000			
90° = 0.00010			
Flatness Tolerance Met?			
Maximum difference must be < 0.0020 in.			
Difference = ± 0.00005			
Perpendicularity Tolerance Met?			
Maximum angle of departure must be ≤ 0.25°			
Perpendicularity Tolerance Met?			
Maximum angle of departure must be ≤ 0.25°			

Client:	Haley & Aldrich, Inc.
Project Name:	Oceanville Rd. Over Oceanville Thoroughfare
Project Location:	Stonington, ME
GTX #:	304692
Test Date:	5/10/2016
Tested By:	daa
Checked By:	jsc
Boring ID:	BB-SOT-101
Sample ID:	R1
Depth, ft:	24.24-24.61



After cutting and grinding

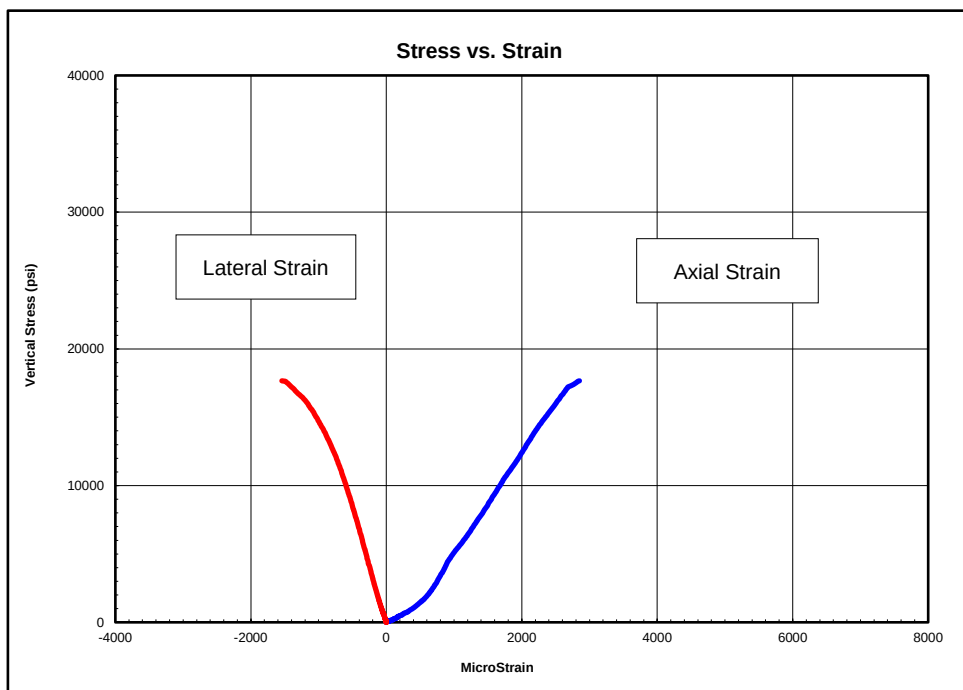


After break



Client:	Haley & Aldrich, Inc.
Project Name:	Oceanville Rd. Over Oceanville Thoroughfare
Project Location:	Stonington, ME
GTX #:	304692
Test Date:	5/10/2016
Tested By:	daa
Checked By:	jsc
Boring ID:	BB-SOT-102
Sample ID:	R2
Depth, ft:	31.30-31.67
Sample Type:	rock core
Sample Description:	See photographs Intact material failure Diameter < 10 times maximum particle size

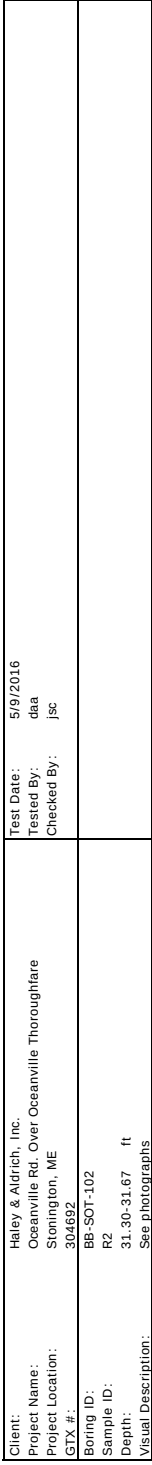
Compressive Strength and Elastic Moduli of Rock by ASTM D7012 - Method D



Peak Compressive Stress: 17,668 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
1800-6500	7,760,000	0.43
6500-11200	7,660,000	0.48
11200-15900	7,360,000	---

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.



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

BULK DENSITY		DEVIATION FROM STRAIGHTNESS (Procedure SI)													
		Maximum gap between side of core and reference surface plate: Is the maximum gap ≤ 0.02 in.? YES													
1	2	Average													
Specimen Length, in:	4.39	4.39													
Specimen Diameter, in:	1.99	1.99													
Specimen Mass, g:	589.36														
Bulk Density, lb/ft ³	164														
Length to Diameter Ratio:	2.2														
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	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Diameter 2, in (rotated 90°)	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
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	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Diameter 2, in (rotated 90°)	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Difference between max and min readings, in:															
0° = 0.00010 90° = 0.00010															
Maximum difference must be < 0.020 in. Difference = ± 0.00005 YES															
Flatness Tolerance Met? YES															
DIAMETER 1															
End 1:															
Slope of Best Fit Line: 0.00006															
Angle of Best Fit Line: 0.00344															
End 2:															
Slope of Best Fit Line: 0.00000															
Angle of Best Fit Line: 0.00000															
Maximum Angular Difference: 0.00344															
Parallelism Tolerance Met? YES															
Spherically Seated															
DIAMETER 2															
End 1:															
Slope of Best Fit Line: 0.00008															
Angle of Best Fit Line: 0.00458															
End 2:															
Slope of Best Fit Line: 0.00007															
Angle of Best Fit Line: 0.00401															
Maximum Angular Difference: 0.00057															
Parallelism Tolerance Met? YES															
Spherically Seated															
Maximum angle of departure must be $\leq 0.25^\circ$															
Perpendicularity 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	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Diameter 2, in (rotated 90°)	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
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	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Diameter 2, in (rotated 90°)	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Difference between max and min readings, in:															
0° = 0.00010 90° = 0.00010															
Maximum difference must be < 0.020 in. Difference = ± 0.00005 YES															
Flatness Tolerance Met? YES															
DIAMETER 1															
End 1:															
Slope of Best Fit Line: 0.00006															
Angle of Best Fit Line: 0.00344															
End 2:															
Slope of Best Fit Line: 0.00000															
Angle of Best Fit Line: 0.00000															
Maximum Angular Difference: 0.00344															
Parallelism Tolerance Met? YES															
Spherically Seated															
DIAMETER 2															
End 1:															
Slope of Best Fit Line: 0.00008															
Angle of Best Fit Line: 0.00458															
End 2:															
Slope of Best Fit Line: 0.00007															
Angle of Best Fit Line: 0.00401															
Maximum Angular Difference: 0.00057															
Parallelism Tolerance Met? YES															
Spherically Seated															
Maximum angle of departure must be $\leq 0.25^\circ$															
Perpendicularity 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	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Diameter 2, in (rotated 90°)	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
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	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Diameter 2, in (rotated 90°)	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Difference between max and min readings, in:															
0° = 0.00010 90° = 0.00010															
Maximum difference must be < 0.020 in. Difference = ± 0.00005 YES															
Flatness Tolerance Met? YES															
DIAMETER 1															
End 1:															
Slope of Best Fit Line: 0.00006															
Angle of Best Fit Line: 0.00344															
End 2:															
Slope of Best Fit Line: 0.00000															
Angle of Best Fit Line: 0.00000															
Maximum Angular Difference: 0.00344															
Parallelism Tolerance Met? YES															
Spherically Seated															
DIAMETER 2															
End 1:															
Slope of Best Fit Line: 0.00008															
Angle of Best Fit Line: 0.00458															
End 2:															
Slope of Best Fit Line: 0.00007															
Angle of Best Fit Line: 0.00401															
Maximum Angular Difference: 0.00057															
Parallelism Tolerance Met? YES															
Spherically Seated															
Maximum angle of departure must be $\leq 0.25^\circ$															
Perpendicularity 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	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Diameter 2, in (rotated 90°)	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
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	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Diameter 2, in (rotated 90°)	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Difference between max and min readings, in:															
0° = 0.00010 90° = 0.00010															
Maximum difference must be < 0.020 in. Difference = ± 0.00005 YES															
Flatness Tolerance Met? YES															
DIAMETER 1															
End 1:															
Slope of Best Fit Line: 0.00006															
Angle of Best Fit Line: 0.00344															
End 2:															
Slope of Best Fit Line: 0.00000															
Angle of Best Fit Line: 0.00000															
Maximum Angular Difference: 0.00344															
Parallelism Tolerance Met? YES															
Spherically Seated															
DIAMETER 2															
End 1:															
Slope of Best Fit Line: 0.00008															
Angle of Best Fit Line: 0.00458															
End 2:															
Slope of Best Fit Line: 0.00007															
Angle of Best Fit Line: 0.00401															
Maximum Angular Difference: 0.00057															
Parallelism Tolerance Met? YES															
Spherically Seated															
Maximum angle of departure must be $\leq 0.25^\circ$															
Perpendicularity 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	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Diameter 2, in (rotated 90°)	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
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	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Diameter 2, in (rotated 90°)	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Difference between max and min readings, in:															
0° = 0.00010 90° = 0.00010															
Maximum difference must be < 0.020 in. Difference = ± 0.00005 YES															
Flatness Tolerance Met? YES															
DIAMETER 1															
End 1:															
Slope of Best Fit Line: 0.00006															
Angle of Best Fit Line: 0.00344															
End 2:															
Slope of Best Fit Line: 0.00000															
Angle of Best Fit Line: 0.00000															
Maximum Angular Difference: 0.00344															
Parallelism Tolerance Met? YES															
Spherically Seated															
DIAMETER 2															
End 1:															
Slope of Best Fit Line: 0.00008															
Angle of Best Fit Line: 0.00458															
End 2:															
Slope of Best Fit Line: 0.00007															
Angle of Best Fit Line: 0.00401															
Maximum Angular Difference: 0.00057															
Parallelism Tolerance Met? YES															
Spherically Seated															
Maximum angle of departure must be $\leq 0.25^\circ$															
Perpendicularity 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	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Diameter 2, in (rotated 90°)	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
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	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Diameter 2, in (rotated 90°)	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Difference between max and min readings, in:															
0° = 0.00010 90° = 0.00010															
Maximum difference must be < 0.020 in. Difference = ± 0.00005 YES															
Flatness Tolerance Met? YES															
DIAMETER 1															
End 1:															
Slope of Best Fit Line: 0.00006															
Angle of Best Fit Line: 0.00344															
End 2:															
Slope of Best Fit Line: 0.00000															
Angle of Best Fit Line: 0.00000															
Maximum Angular Difference: 0.00344															
Parallelism Tolerance Met? YES															
Spherically Seated															
DIAMETER 2															
End 1:															
Slope of Best Fit Line: 0.00008															
Angle of Best Fit Line: 0.00458															
End 2:															
Slope of Best Fit Line: 0.00007															
Angle of Best Fit Line: 0.00401															
Maximum Angular Difference: 0.00057															
Parallelism Tolerance Met? YES															
Spherically Seated															
Maximum angle of departure must be $\leq 0.25^\circ$															
Perpendicularity 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	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Diameter 2, in (rotated 90°)	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
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	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Diameter 2, in (rotated 90°)	-0.00010	-0.00010	-0.00010	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Difference between max and min readings, in:															
0° = 0.00010 90° = 0.00010															
Maximum difference must be < 0.020 in. Difference = ± 0.00005 YES															
Flatness Tolerance Met? YES															
DIAMETER 1															
End 1:															
Slope of Best Fit Line: 0.0000															

Client:	Haley & Aldrich, Inc.
Project Name:	Oceanville Rd. Over Oceanville Thoroughfare
Project Location:	Stonington, ME
GTX #:	304692
Test Date:	5/10/2016
Tested By:	daa
Checked By:	jsc
Boring ID:	BB-SOT-102
Sample ID:	R2
Depth, ft:	31.30-31.67



After cutting and grinding

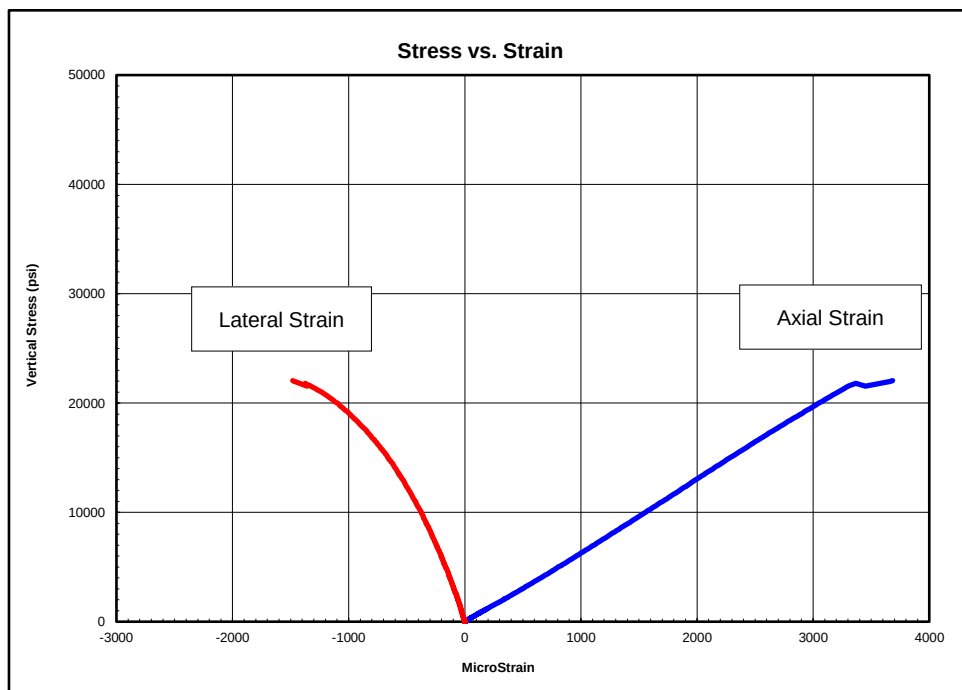


After break



Client:	Haley & Aldrich, Inc.
Project Name:	Oceanville Rd over Oceanville Thr.
Project Location:	Stonington, ME
GTX #:	316597
Test Date:	1/18/2023
Tested By:	bp
Checked By:	jsc
Boring ID:	BB-SOT-201
Sample ID:	R2
Depth, ft:	32.39-32.77
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: 22,053 psi

The strain values recorded within the third stress range 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
2200-8100	6,480,000	0.25
8100-14000	6,820,000	0.35
14000-19800	6,590,000	---

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.

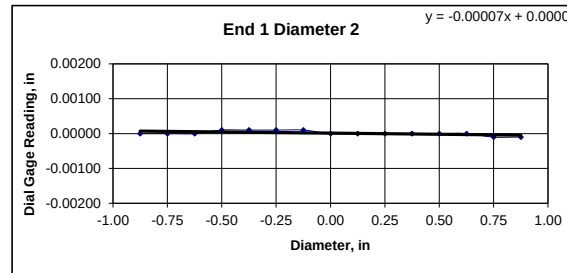
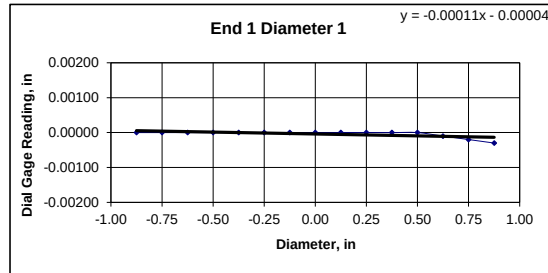


Client:	Haley & Aldrich, Inc.	Test Date:	1/13/2023
Project Name:	Oceanville Rd over Oceanville Thr.	Tested By:	bp/te
Project Location:	Stonington, ME	Checked By:	smd
GTx #:	316597		
Boring ID:	BB-SOT-201		
Sample ID:	R2		
Depth:	32.39-32.77 ft		
Visual Description:	See photographs		

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: Is the maximum gap $\leq$ 0.02 in.? YES	
Specimen Length, in:	4.41	4.42	4.42	Maximum difference must be $<$ 0.020 in. Straightness Tolerance Met? YES	
Specimen Diameter, in:	1.99	1.99	1.99		
Specimen Mass, g:	597.33				
Bulk Density, lb/ft ³	165				
Length to Diameter Ratio:	2.2	Minimum Diameter Tolerance Met? YES	Length to Diameter Ratio Tolerance Met? YES		

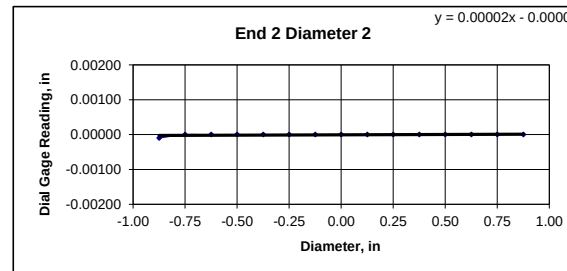
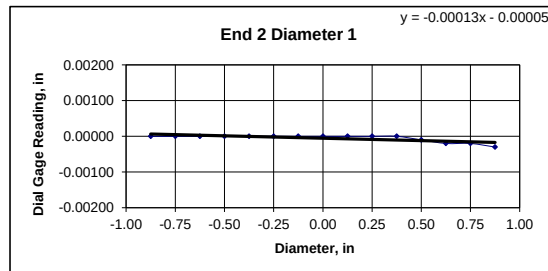
END FLATNESS AND PARALLELISM (Procedure FP1)														
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750
Diameter 1, in	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00030
Diameter 2, in (rotated 90°)	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010
Difference between max and min readings, in: 0° = 0.00030 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
Diameter 1, in	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00030
Diameter 2, in (rotated 90°)	-0.00010	0.00000	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° = 0.0003 90° = 0.0001 Maximum difference must be $<$ 0.0020 in. Difference = $\pm$ 0.00015 Flatness Tolerance Met? YES														



DIAMETER 1

End 1:	Slope of Best Fit Line	0.00011
	Angle of Best Fit Line:	0.00622
End 2:	Slope of Best Fit Line	0.00013
	Angle of Best Fit Line:	0.00769
Maximum Angular Difference:		0.00147

Parallelism Tolerance Met? YES
Spherically Seated



DIAMETER 2

End 1:	Slope of Best Fit Line	0.00007
	Angle of Best Fit Line:	0.00377
End 2:	Slope of Best Fit Line	0.00002
	Angle of Best Fit Line:	0.00115
Maximum Angular Difference:		0.00262

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?	Maximum angle of departure must be $\leq$ 0.25°	
Diameter 1, in	0.00030	1.990	0.00015	0.009	YES	Perpendicularity Tolerance Met? YES	
Diameter 2, in (rotated 90°)	0.00020	1.990	0.00010	0.006	YES		
END 2							
Diameter 1, in	0.00030	1.990	0.00015	0.009	YES		
Diameter 2, in (rotated 90°)	0.00010	1.990	0.00005	0.003	YES		

Client:	Haley & Aldrich, Inc.
Project Name:	Oceanville Rd over Oceanville Thr.
Project Location:	Stonington, ME
GTX #:	316597
Test Date:	1/18/2023
Tested By:	bp
Checked By:	smd
Boring ID:	BB-SOT-201
Sample ID:	R2
Depth, ft:	32.39-32.77



After cutting and grinding

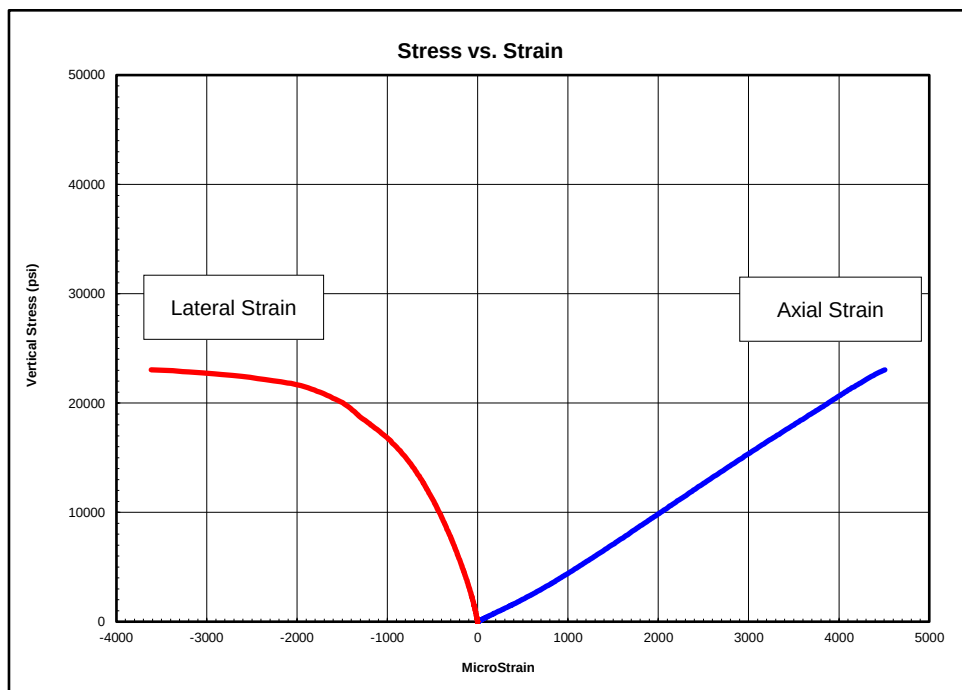


After break



Client:	Haley & Aldrich, Inc.
Project Name:	Oceanville Rd over Oceanville Thr.
Project Location:	Stonington, ME
GTX #:	316597
Test Date:	1/18/2023
Tested By:	bp
Checked By:	jsc
Boring ID:	BB-SOT-201
Sample ID:	R3
Depth, ft:	36.76-37.13
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: 23,037 psi

The strain values recorded within the third stress range 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
2300-8400	5,190,000	0.23
8400-14600	5,540,000	0.37
14600-20700	5,260,000	---

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.

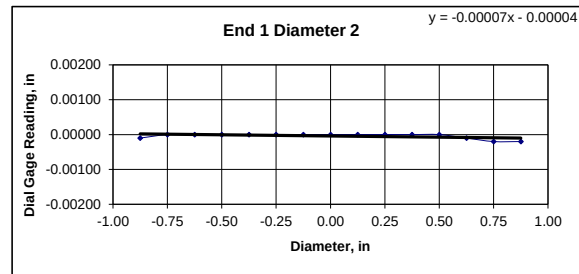
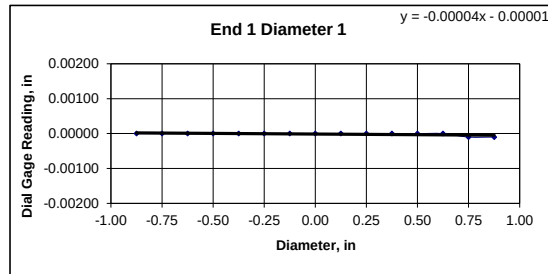


Client:	Haley & Aldrich, Inc.	Test Date:	1/13/2023
Project Name:	Oceanville Rd over Oceanville Thr.	Tested By:	bp/te
Project Location:	Stonington, ME	Checked By:	smd
GTx #:	316597		
Boring ID:	BB-SOT-201		
Sample ID:	R3		
Depth:	36.76-37.13 ft		
Visual Description:	See photographs		

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: Is the maximum gap $\leq$ 0.02 in.? YES	
Specimen Length, in:	4.48	4.48	4.48	Maximum difference must be $<$ 0.020 in.	
Specimen Diameter, in:	1.99	1.99	1.99	Straightness Tolerance Met? YES	
Specimen Mass, g:	603.51				
Bulk Density, lb/ft ³ :	165				
Length to Diameter Ratio:	2.3	Minimum Diameter Tolerance Met? YES			
		Length to Diameter Ratio Tolerance Met? YES			

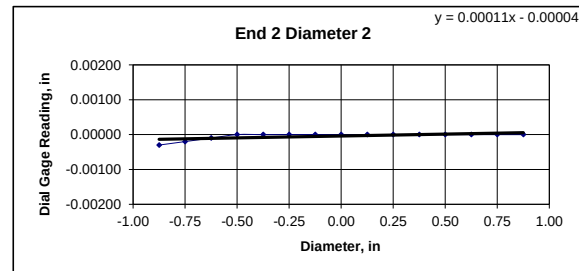
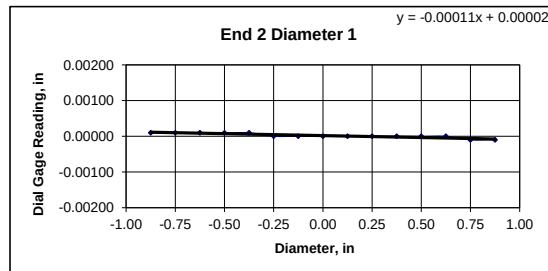
END FLATNESS AND PARALLELISM (Procedure FP1)															
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010
Diameter 2, in (rotated 90°)	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00020
Difference between max and min readings, in: 0° = 0.00010 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	0.00010	0.00010	0.00010	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010
Diameter 2, in (rotated 90°)	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
Difference between max and min readings, in: 0° = 0.0002 90° = 0.0003 Maximum difference must be < 0.0020 in. Difference = ± 0.00015															
Flatness Tolerance Met? YES															



DIAMETER 1

End 1:	Slope of Best Fit Line	0.00004
	Angle of Best Fit Line:	0.00213
End 2:	Slope of Best Fit Line	0.00011
	Angle of Best Fit Line:	0.00622
Maximum Angular Difference:		0.00409

Parallelism Tolerance Met? YES
Spherically Seated



DIAMETER 2

End 1:	Slope of Best Fit Line	0.00007
	Angle of Best Fit Line:	0.00393
End 2:	Slope of Best Fit Line	0.00011
	Angle of Best Fit Line:	0.00622
Maximum Angular Difference:		0.00229

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?	Maximum angle of departure must be $\leq$ 0.25°	
Diameter 1, in	0.00010	1.990	0.00005	0.003	YES	Perpendicularity Tolerance Met? YES	
Diameter 2, in (rotated 90°)	0.00020	1.990	0.00010	0.006	YES		
END 2							
Diameter 1, in	0.00020	1.990	0.00010	0.006	YES		
Diameter 2, in (rotated 90°)	0.00030	1.990	0.00015	0.009	YES		

Client:	Haley & Aldrich, Inc.
Project Name:	Oceanville Rd over Oceanville Thr.
Project Location:	Stonington, ME
GTX #:	316597
Test Date:	1/18/2023
Tested By:	bp
Checked By:	smd
Boring ID:	BB-SOT-201
Sample ID:	R3
Depth, ft:	36.76-37.13



After cutting and grinding

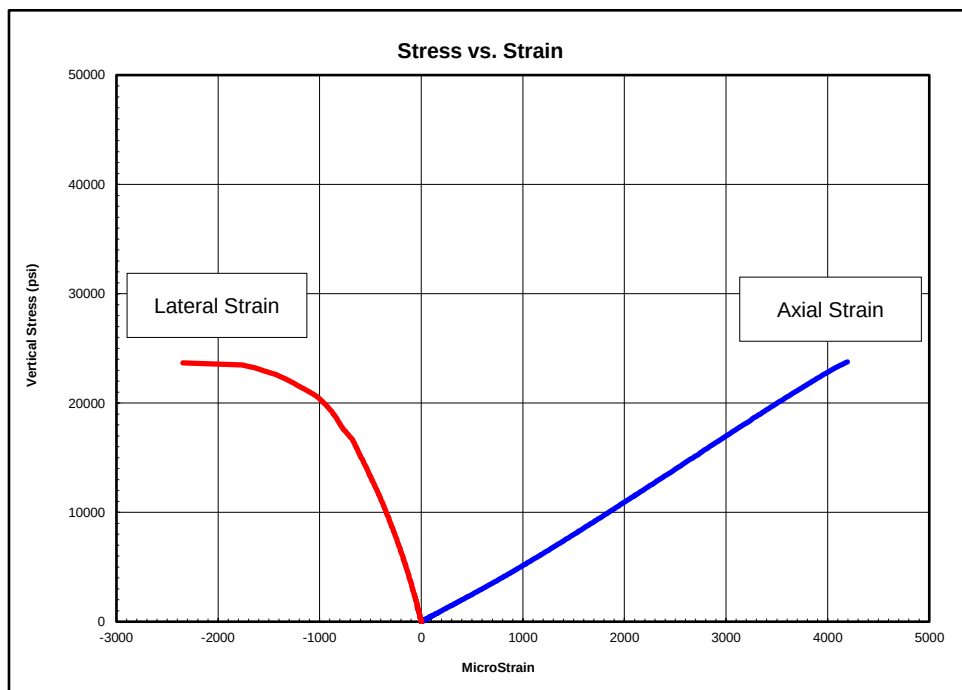


After break



Client:	Haley & Aldrich, Inc.
Project Name:	Oceanville Rd over Oceanville Thr.
Project Location:	Stonington, ME
GTX #:	316597
Test Date:	1/18/2023
Tested By:	bp
Checked By:	jsc
Boring ID:	BB-SOT-202
Sample ID:	R2
Depth, ft:	29.07-29.47
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: 23,764 psi

The strain values recorded within the third stress range 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
2400-8700	5,500,000	0.19
8700-15100	6,000,000	0.29
15100-21400	6,000,000	---

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.

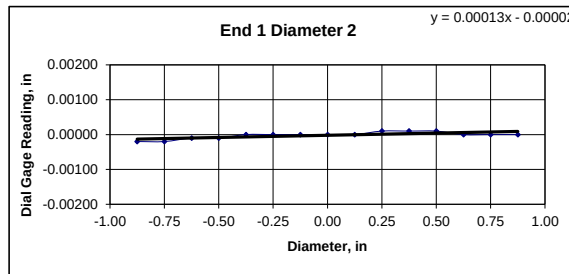
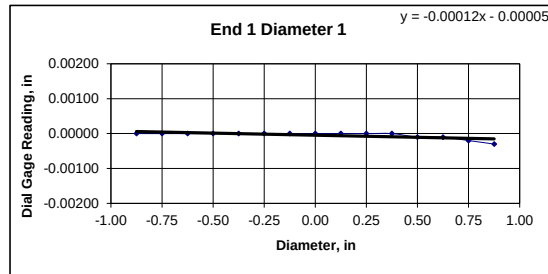


Client:	Haley & Aldrich, Inc.	Test Date:	1/13/2023
Project Name:	Oceanville Rd over Oceanville Thr.	Tested By:	bp/te
Project Location:	Stonington, ME	Checked By:	smd
GTx #:	316597		
Boring ID:	BB-SOT-202		
Sample ID:	R2		
Depth:	29.07-29.47 ft		
Visual Description:	See photographs		

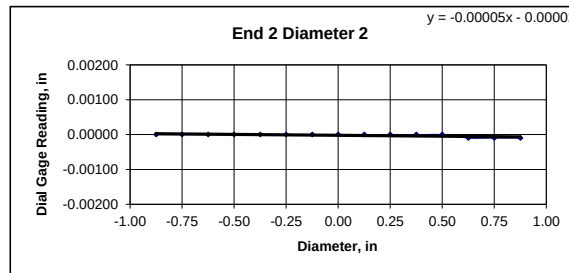
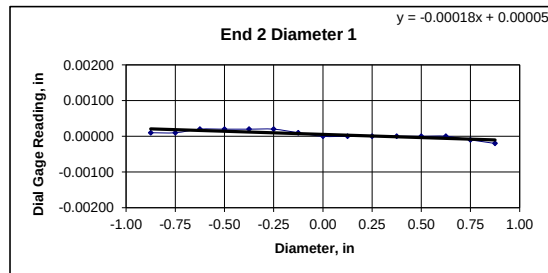
UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

BULK DENSITY				DEVIATION FROM STRAIGHTNESS (Procedure S1)	
	1	2	Average	Maximum gap between side of core and reference surface plate: Is the maximum gap $\leq$ 0.02 in.? YES	
Specimen Length, in:	4.35	4.35	4.35	Maximum difference must be < 0.020 in.	
Specimen Diameter, in:	1.99	1.99	1.99	Straightness Tolerance Met? YES	
Specimen Mass, g:	588.33				
Bulk Density, lb/ft ³ :	165				
Length to Diameter Ratio:	2.2	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	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00020	-0.00030
Diameter 2, in (rotated 90°)	-0.00020	-0.00020	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010	0.00000	0.00000	0.00000
Difference between max and min readings, in:															
0° = 0.00030 90° = 0.00030															
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00010	0.00010	0.00020	0.00020	0.00020	0.00020	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-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	0.00000	0.00000	0.00000	-0.00010	-0.00010	-0.00010
Difference between max and min readings, in:															
0° = 0.0004 90° = 0.0001															
Maximum difference must be < 0.0020 in. Difference = ± 0.00020															
Flatness Tolerance Met? YES															



DIAMETER 1	
End 1:	
Slope of Best Fit Line	0.00012
Angle of Best Fit Line:	0.00688
End 2:	
Slope of Best Fit Line	0.00018
Angle of Best Fit Line:	0.01015
Maximum Angular Difference:	0.00327
Parallelism Tolerance Met? Spherically Seated	YES



DIAMETER 2	
End 1:	
Slope of Best Fit Line	0.00013
Angle of Best Fit Line:	0.00720
End 2:	
Slope of Best Fit Line	0.00005
Angle of Best Fit Line:	0.00295
Maximum Angular Difference:	0.00426
Parallelism Tolerance Met? Spherically Seated	YES

PERPENDICULARITY (Procedure P1)						Maximum angle of departure must be $\leq 0.25^\circ$	
END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?		
Diameter 1, in	0.00030	1.990	0.00015	0.009	YES		
Diameter 2, in (rotated 90°)	0.00030	1.990	0.00015	0.009	YES	Perpendicularity Tolerance Met? YES	
END 2							
Diameter 1, in	0.00040	1.990	0.00020	0.012	YES		
Diameter 2, in (rotated 90°)	0.00010	1.990	0.00005	0.003	YES		

Client:	Haley & Aldrich, Inc.
Project Name:	Oceanville Rd over Oceanville Thr.
Project Location:	Stonington, ME
GTX #:	316597
Test Date:	1/18/2023
Tested By:	bp
Checked By:	smd
Boring ID:	BB-SOT-202
Sample ID:	R2
Depth, ft:	29.07-29.47



After cutting and grinding

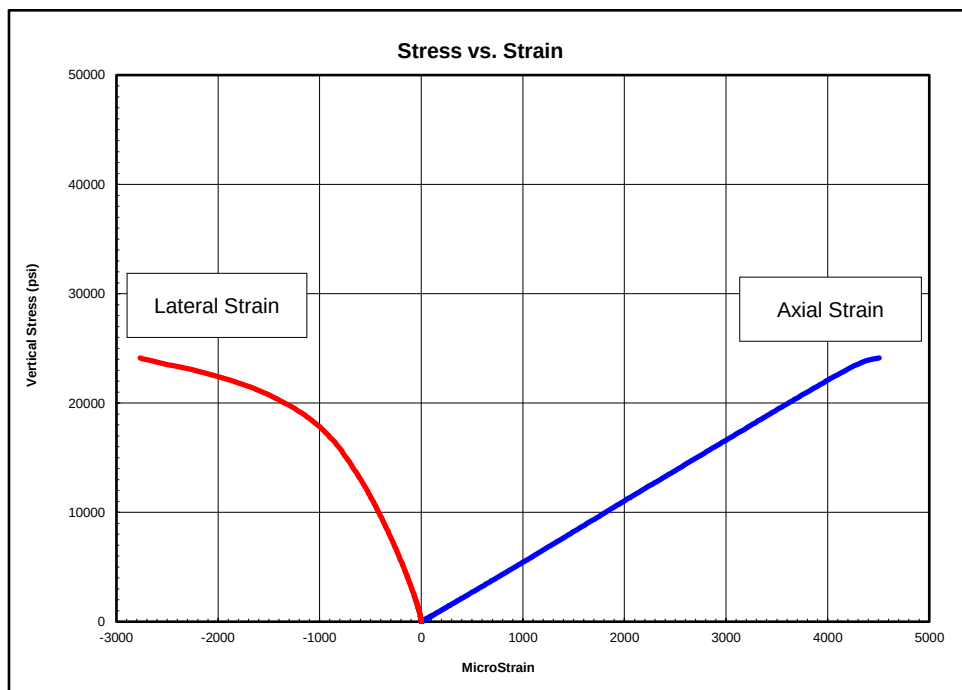


After break



Client:	Haley & Aldrich, Inc.
Project Name:	Oceanville Rd over Oceanville Thr.
Project Location:	Stonington, ME
GTX #:	316597
Test Date:	1/18/2023
Tested By:	bp
Checked By:	jsc
Boring ID:	BB-SOT-202
Sample ID:	R4
Depth, ft:	39.44-39.83
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: 24,125 psi

The strain values recorded within the third stress range 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
2400-8800	5,550,000	0.25
8800-15300	5,580,000	0.34
15300-21700	5,510,000	---

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.

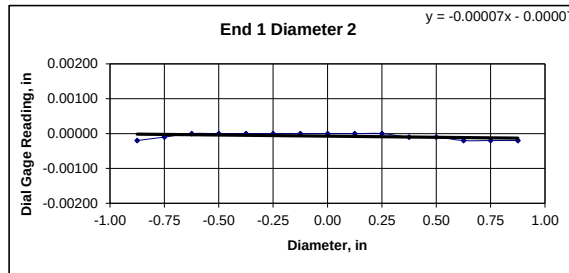
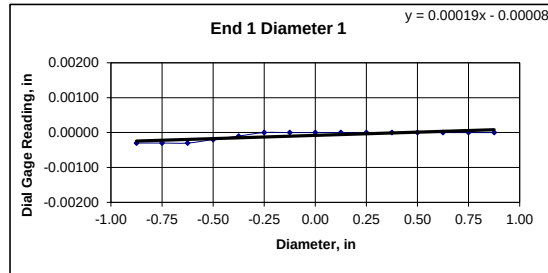


Client:	Haley & Aldrich, Inc.	Test Date:	1/16/2023
Project Name:	Oceanville Rd over Oceanville Thr.	Tested By:	bp/te
Project Location:	Stonington, ME	Checked By:	smd
GTX #:	316597		
Boring ID:	BB-SOT-202		
Sample ID:	R4		
Depth:	39.44-39.83 ft		
Visual Description:	See photographs		

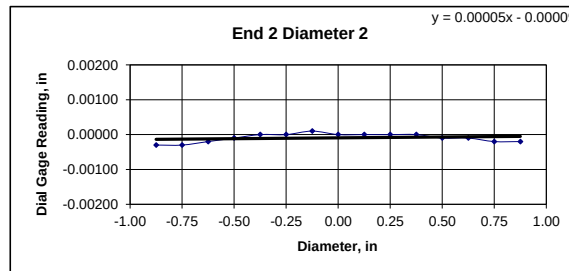
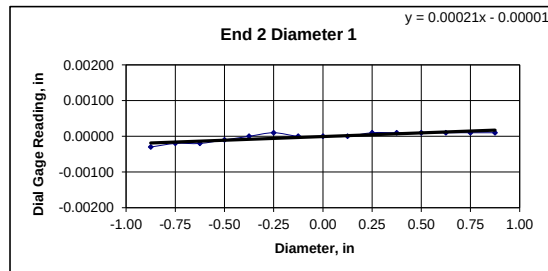
UNIT WEIGHT DETERMINATION AND DIMENSIONAL AND SHAPE TOLERANCES OF ROCK CORE SPECIMENS BY ASTM D4543

BULK DENSITY				DEVIATION FROM STRAIGHTNESS (Procedure S1)	
	1	2	Average	Maximum gap between side of core and reference surface plate: Is the maximum gap $\leq$ 0.02 in.? YES	
Specimen Length, in:	4.41	4.41	4.41	Maximum difference must be $<$ 0.020 in. Straightness Tolerance Met? YES	
Specimen Diameter, in:	1.99	1.99	1.99		
Specimen Mass, g:	595.94				
Bulk Density, lb/ft ³ :	165				
Length to Diameter Ratio:	2.2	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
Diameter 1, in	-0.00030	-0.00030	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000
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	-0.00010	-0.00010	-0.00020	-0.00020
Difference between max and min readings, in: 0° = 0.00030 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
Diameter 1, in	-0.00030	-0.00020	-0.00020	-0.00010	0.00000	0.00010	0.00000	0.00000	0.00000	0.00010	0.00010	0.00010	0.00010	0.00010
Diameter 2, in (rotated 90°)	-0.00030	-0.00030	-0.00020	-0.00010	0.00000	0.00000	0.00010	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00020	-0.00020
Difference between max and min readings, in: 0° = 0.0004 90° = 0.0004 Maximum difference must be $<$ 0.0020 in. Difference = $\pm$ 0.00020 Flatness Tolerance Met? YES														



DIAMETER 1	
End 1:	
Slope of Best Fit Line	0.00019
Angle of Best Fit Line:	0.01064
End 2:	
Slope of Best Fit Line	0.00021
Angle of Best Fit Line:	0.01179
Maximum Angular Difference:	0.00115
Parallelism Tolerance Met? Spherically Seated	YES



DIAMETER 2	
End 1:	
Slope of Best Fit Line	0.00007
Angle of Best Fit Line:	0.00377
End 2:	
Slope of Best Fit Line	0.00005
Angle of Best Fit Line:	0.00278
Maximum Angular Difference:	0.00098
Parallelism Tolerance Met? Spherically Seated	YES

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	0.00030	1.990	0.00015	0.009	YES	Perpendicularity Tolerance Met? YES	
Diameter 2, in (rotated 90°)	0.00020	1.990	0.00010	0.006	YES		
END 2							
Diameter 1, in	0.00040	1.990	0.00020	0.012	YES		
Diameter 2, in (rotated 90°)	0.00040	1.990	0.00020	0.012	YES		

Client:	Haley & Aldrich, Inc.
Project Name:	Oceanville Rd over Oceanville Thr.
Project Location:	Stonington, ME
GTX #:	316597
Test Date:	1/18/2023
Tested By:	bp
Checked By:	smd
Boring ID:	BB-SOT-202
Sample ID:	R4
Depth, ft:	39.44-39.83



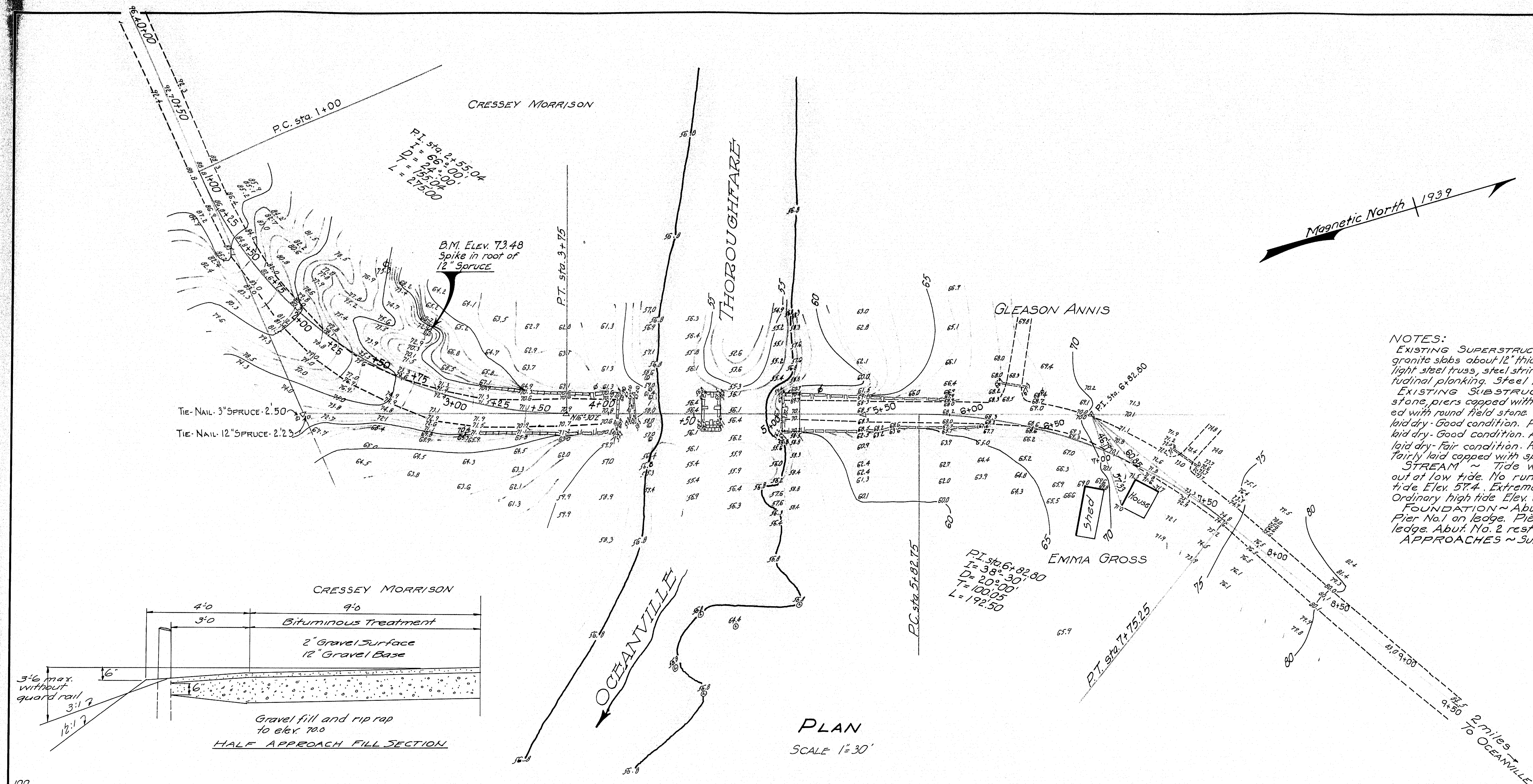
After cutting and grinding



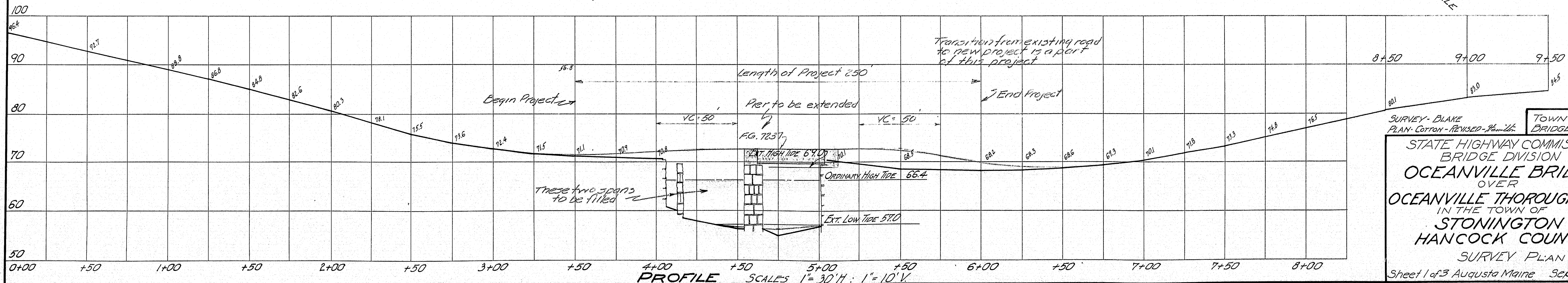
After break

APPENDIX C

Historic Bridge Drawings

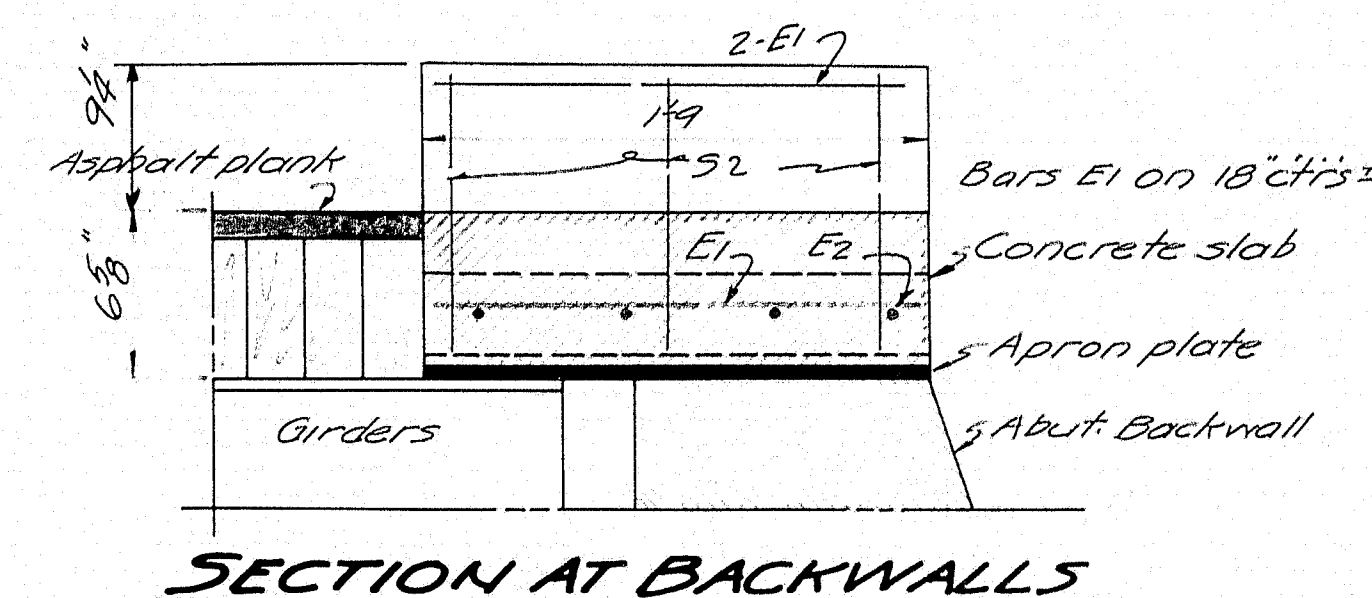
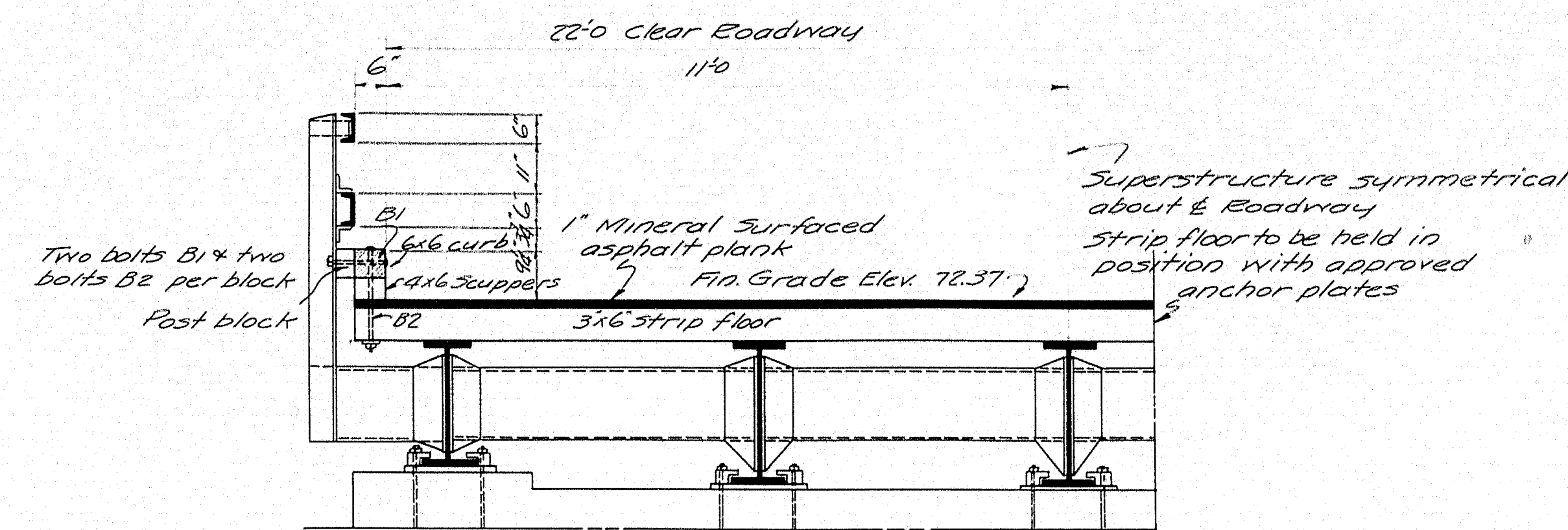
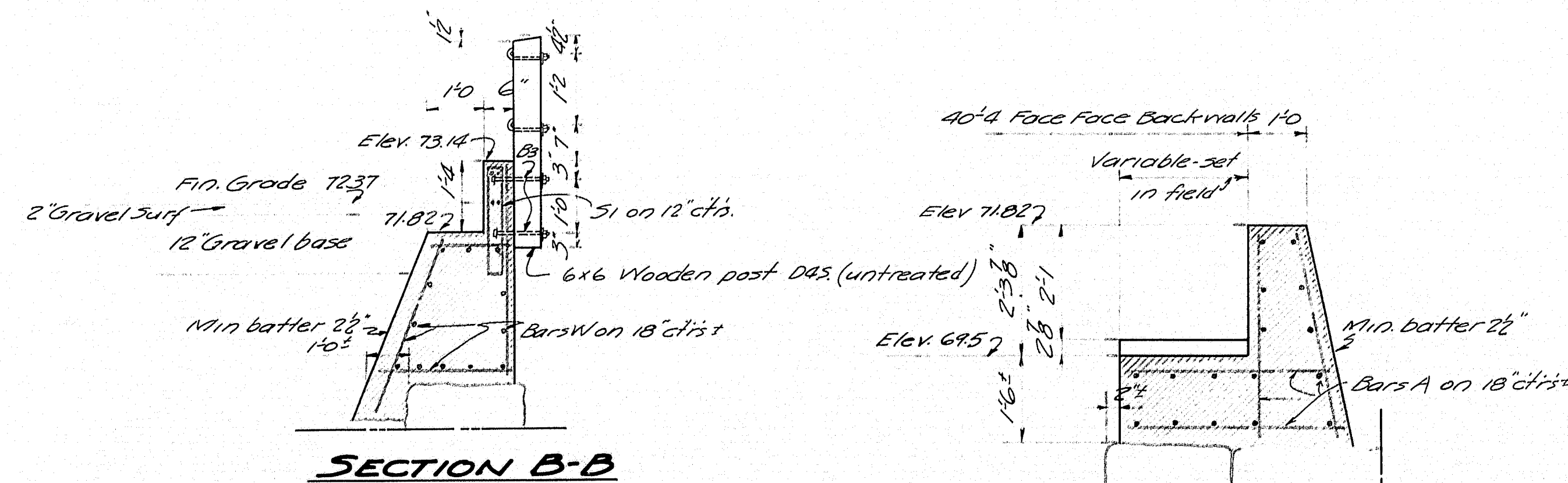
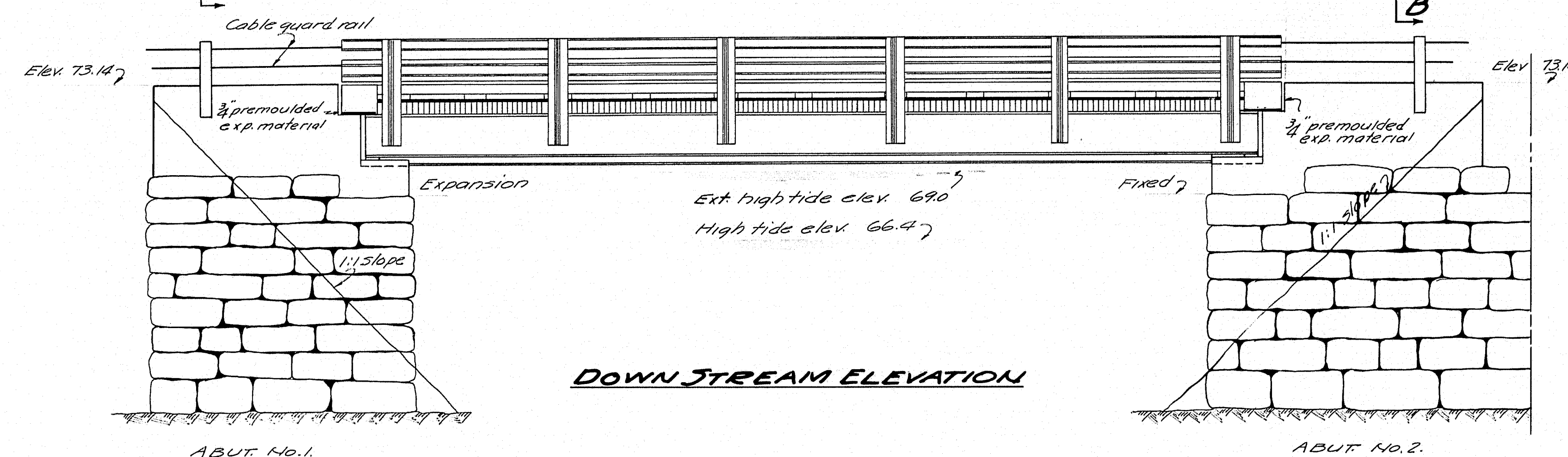
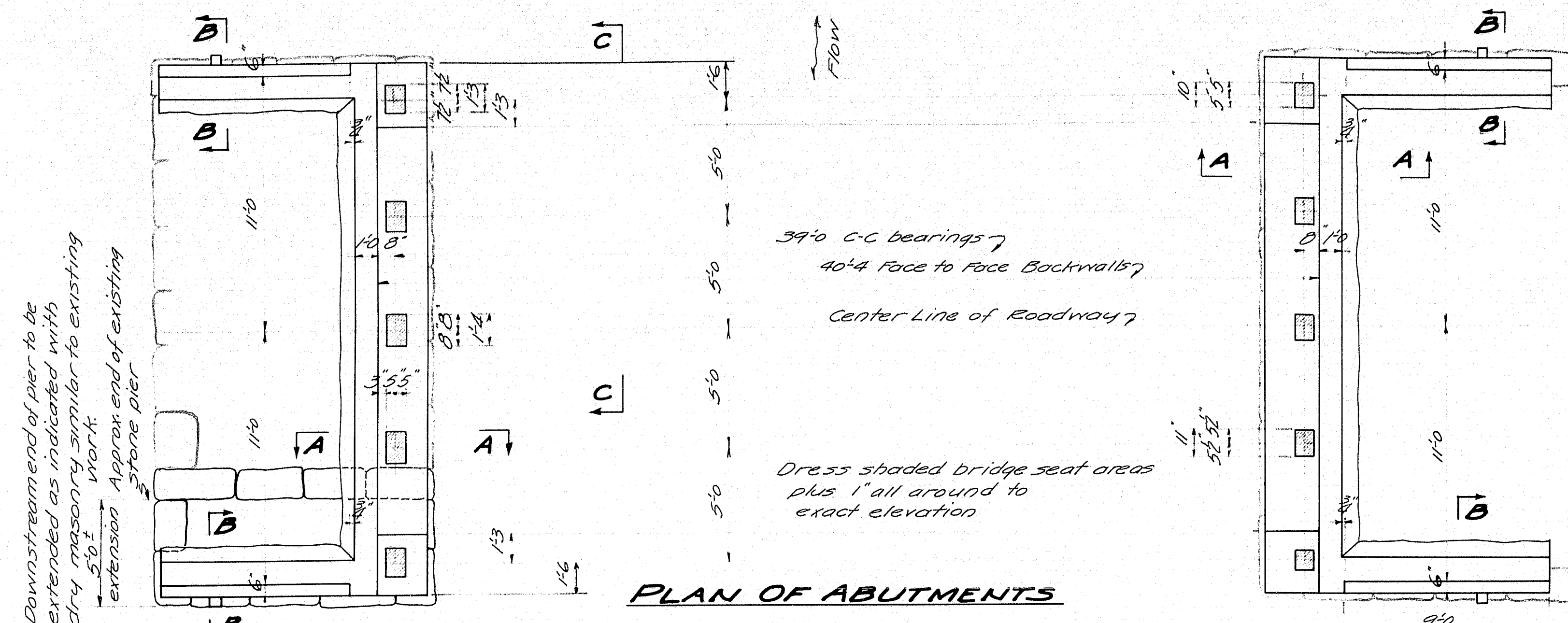


NOTES:
 EXISTING SUPERSTRUCTURE ~ Span No. 1 split granite slabs about 12" thick. Span No. 2 and span No. 3 light steel truss, steel stringers, 2" transverse and 1" longitudinal planking. Steel rusted badly built by Canton.
 EXISTING SUBSTRUCTURE ~ Abut. No. 1 built of stone piers capped with stone slabs. D.S. corner extended with round field stone. Pier No. 1 roughly split granite laid dry. Good condition. Pier No. 2 roughly split granite laid dry. Fair condition. Retaining walls round stone fairly laid capped with split stone.
 STREAM ~ Tide water. Water drains practically out at low tide. No running water. Extreme low tide Elev. 57.4. Extreme high tide Elev. 69.0. Ordinary high tide Elev. 66.4.
 FOUNDATION ~ Abut. No. 1 probably rests on ledge. Pier No. 1 on ledge. Pier No. 2 probably not far from ledge. Abut. No. 2 rests on ledge.
 APPROACHES ~ Surface treated gravel.



SURVEY - BLAKE
 PLAN - COTTON - REVISED - 1939
 TOWN 05-35
 BRIDGE 3596
 STATE HIGHWAY COMMISSION
 BRIDGE DIVISION
 OCEANVILLE BRIDGE
 OVER
 OCEANVILLE THOROUGHFARE
 IN THE TOWN OF
 STONINGTON
 HANCOCK COUNTY
 SURVEY PLAN
 Sheet 1 of 3 Augusta Maine Sept. 1939.

Note: center line of roadway to be set so that new structure will fit existing abutment face of concrete bridge seat to follow existing stone.



TIMBER SCHEDULE	
Strip Floor 3x6 Dns 28'x58' Fin. Size	Required 180 pieces 23'0 long.
Curb 6x6 Dns 58'x38' Fin. Size	Required 4 pieces 14'5 long.
Scuppers 4x6 Dns 38'x58' Fin. Size	Required 22 pieces 14'0 long.
Post Blocks 4x6 Dns 38'x58' Fin. Size	Required 12 pieces 0'7 1/2 long.

All timber to be pressure treated.

BOLT SCHEDULE	
Required 24 bolts B1 5/8" x 0'11"	
Required 44 bolts B2 5/8" x 1'5"	
Equipped with one cast washer each.	
Reqd. 8 bolts B3 3/4" x 1'0" Equipped with one 3x3 plate washer & one cast washer each.	

Bolts to be carriage type with hex nuts.

REINFORCING STEEL	
Bars A 5/8" x 20'0	Reqd. 60 Abutments.
Bar E1 1/2" x 1'6"	Reqd. 40 Superstructure.
Bars E2 1/2" x 22'9"	Reqd. 8 Superstructure.
Bars W 5/8" x 12'0	Reqd. 60 Abutments.

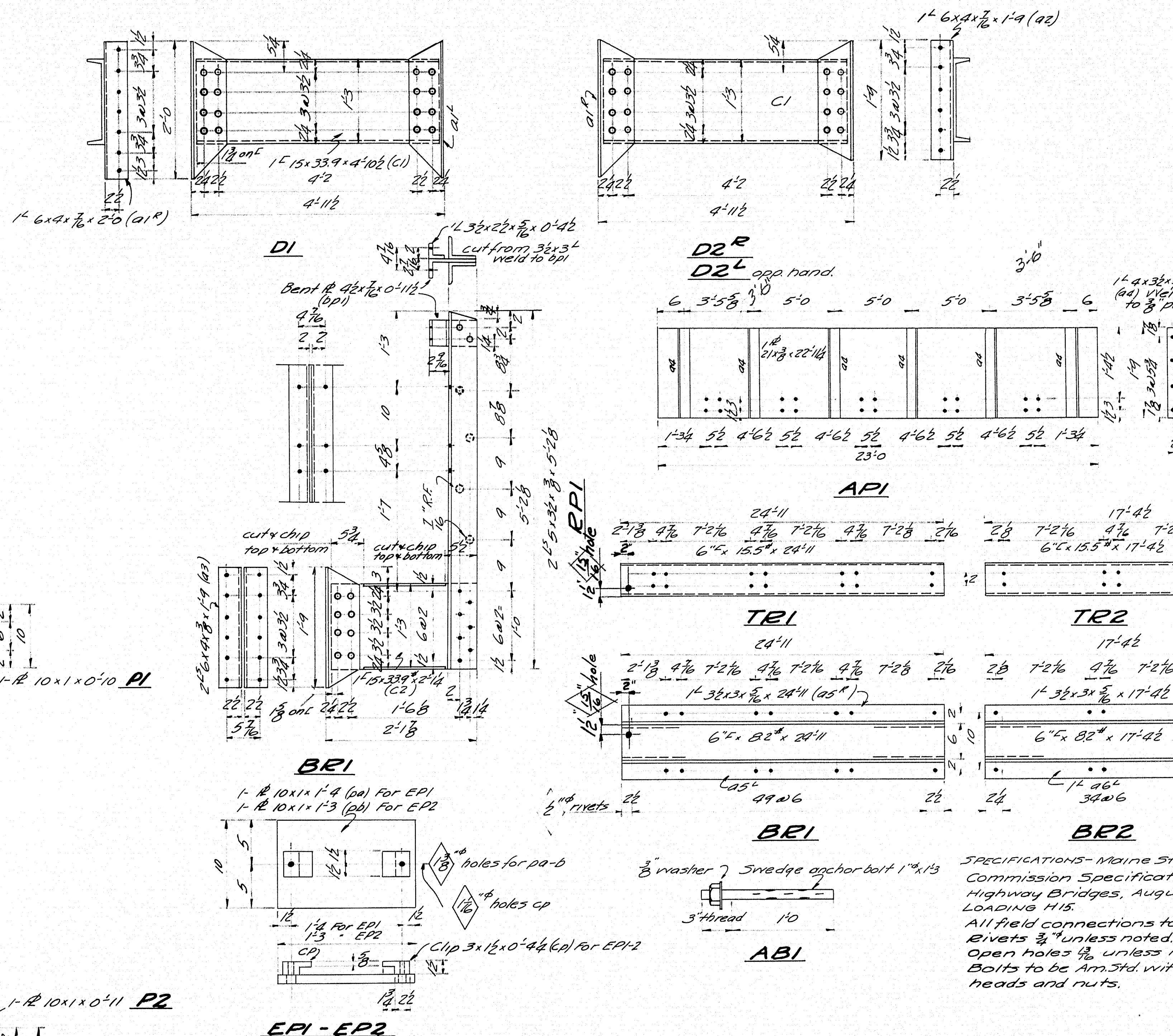
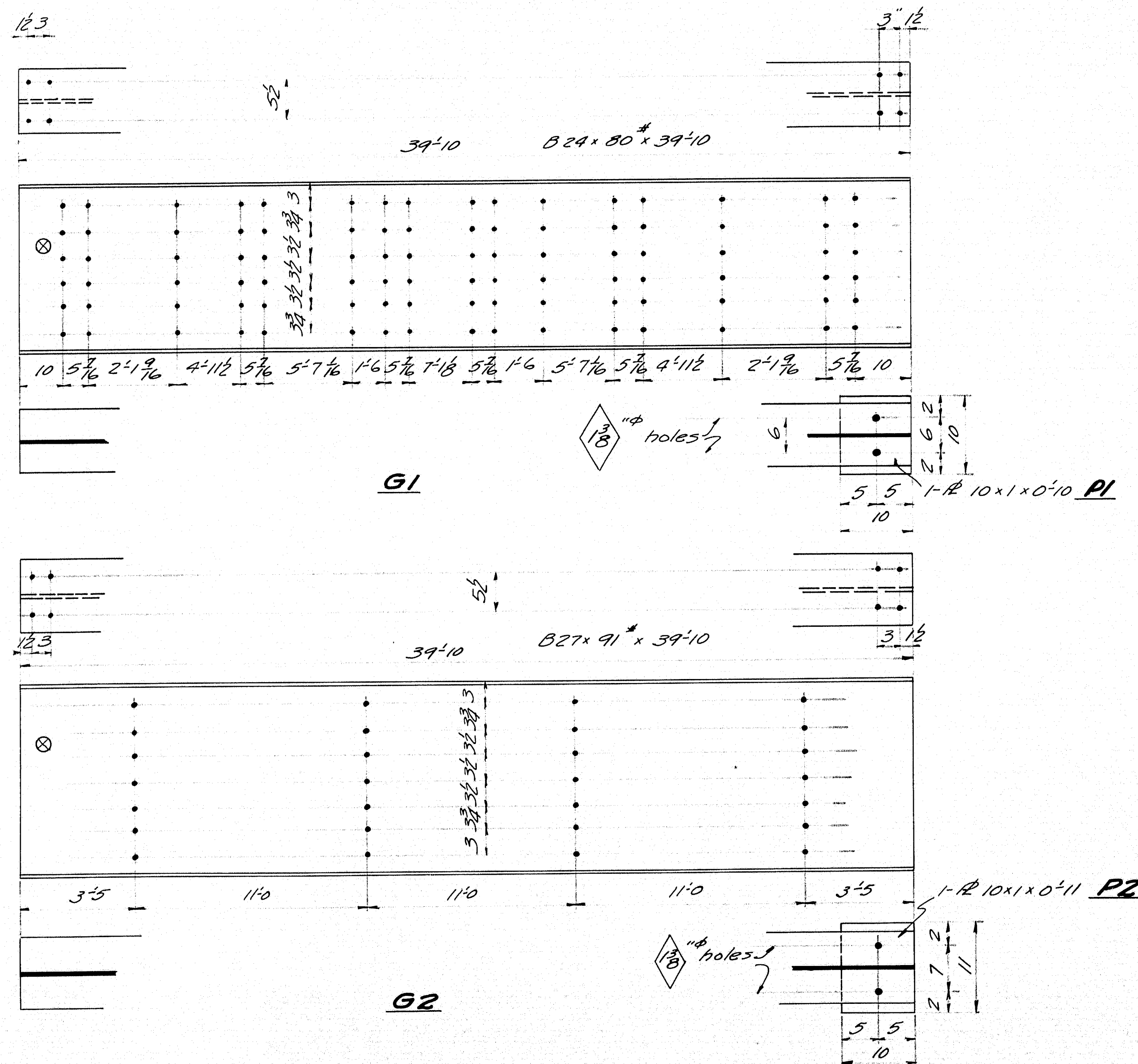
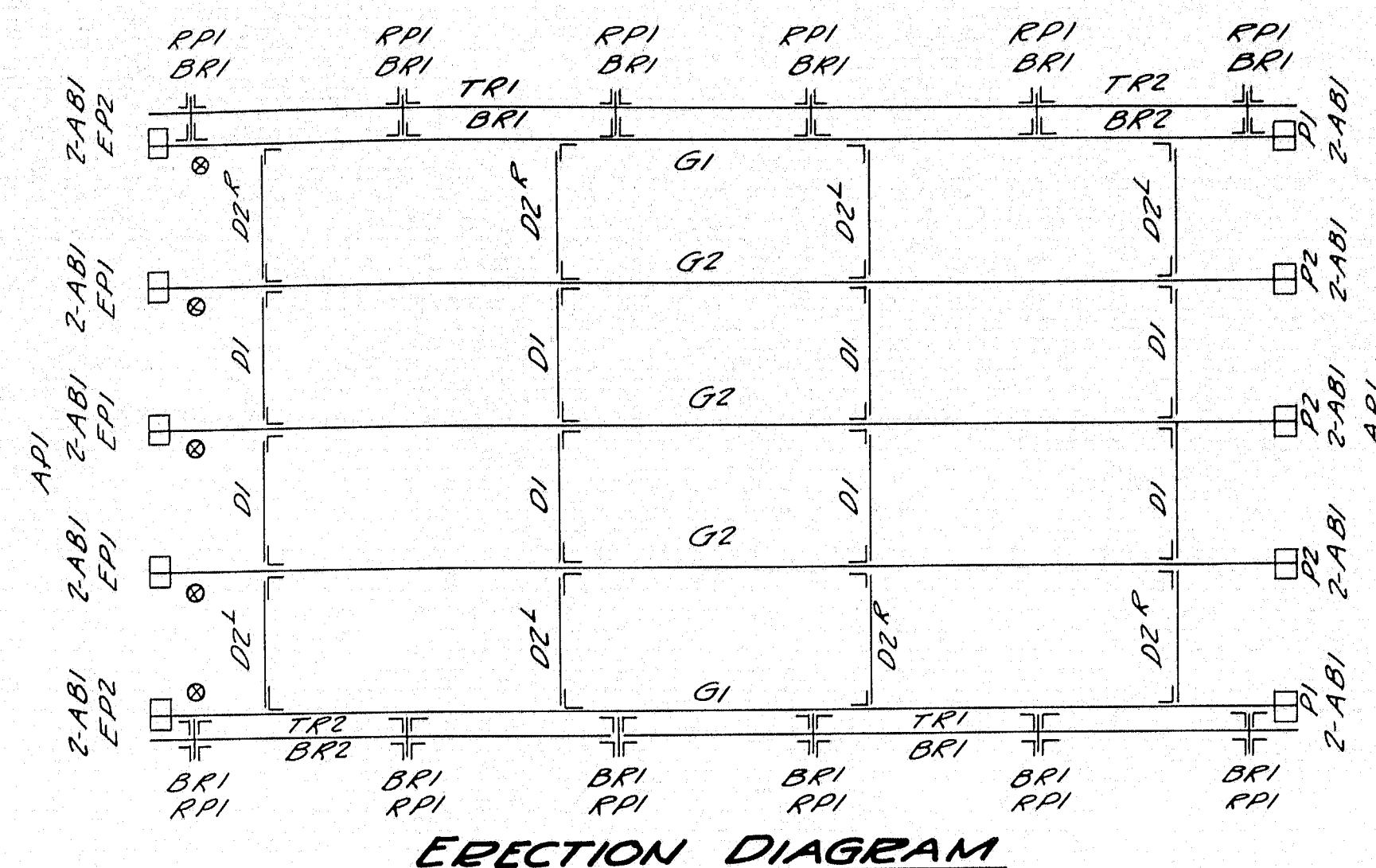
Required 900' 1" mineral surfaced asphalt plank.

Cut bars "A" & "W" to required lengths in field.

Plan by V.
Checked by E.B.

TOWN 0535
Bridge 3696

STATE HIGHWAY COMMISSION
BRIDGE DIVISION
OCEANVILLE BRIDGE
over
OCEANVILLE THOROUGHFARE
in the town of
STONINGTON
HANCOCK COUNTY
DETAILS
Sheet 2 of 3 Augusta, ME. NOV. 1939.



MATERIAL LIST		
Mat. Req'd	Remarks	
G1	2	Girders
G2	3	do
D1	8	Diaphragms
D2R	4	do
D2L	4	do
BRI	12	Brackets
RPI	12	Posts
PI	2	Masonry Plates
P2	3	do
EPI	3	Expansion Plates
EP2	2	do
AP1	2	Apron Plates
TR1	2	Rail Bars
TR2	2	do
BRI	2	do
BR2	2	do
ABI	20	Anchor Bolts

Bolt Schedule		
No	Size	Length
48	3/4"	1 1/2"
240	3/4"	2"
124	3/4"	2 1/2"
84	3/4"	2 1/2"

SPECIFICATIONS - Maine State Highway Commission Specifications for Steel Highway Bridges, August 1937. LOADING H15. All field connections to be bolted. Rivets 3/4" unless noted. Open holes 1/2" unless noted. Bolts to be Am. Std. with hex heads and nuts.

Plan by E. B. Checked by E. B. TOWN 0535 Bridge 3696

STATE HIGHWAY COMMISSION
BRIDGE DIVISION
OCEANVILLE BRIDGE
over
OCEANVILLE THOROUGHFARE
in the town of
STONINGTON
HANCOCK COUNTY
STEEL DETAILS
Sheet 3 of 3 Augusta Me., Nov. 1939

APPENDIX D

Geotechnical Design Calculations

Seismic Site Class

Client Fuss & O'Neill

Project Oceanville Road over Oceanville Thoroughfare, WIN 22630, Bridge No. 3696

Subject Seismic Site Class Evaluation

PROBLEM STATEMENT & OBJECTIVE

Determine the Seismic Site Class using SPT N-values and assumed S_u values from borings drilled approximately near the proposed substructures.

EXECUTIVE SUMMARY

Based on the subsurface conditions encountered at the two borings near the proposed substructures (BB-BSB-101 and BB-BSB-102), recommend a **Seismic Site Class D**.

REFERENCES

1. MaineDOT Bridge Design Guide, August 2003, with interim revisions through March 2014
2. AASHTO LRFD Bridge Design Specifications, 9th edition, 2020

AVAILABLE INFORMATION

1. Boring logs.
2. Elevations reference the North American Vertical Datum of 1988 (NAVD 88).

ASSUMPTIONS

1. Where SPT N-value was available to depths less than 100 ft, the subsurface profile was extended to 100 ft. The SPT N-values for the extended profile were then assumed based on the available information.
2. WOH/WOR = SPT N-value of 1.
3. Used Method C and assumed S_u values per MaineDOT Field Identification Card.

PROCEDURE

1. Check the site against the three categories of Site Class F (see attached Table 3.4.2.1-1), requiring site-specific ground motion response evaluation. If the site corresponds to any of these categories, classify the site as Site Class F and conduct a site-specific ground motion response evaluation.
2. Categorize the site using one of the following three methods (Method A, B, or C).

Method A

Average shear wave velocity for the upper 100 ft of the soil profile:

$$\bar{V}_s = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{V_{si}}}$$

where

V_{si} = shear wave velocity of i th soil (ft/s).

d_i = thickness of i th soil layer (ft).

n = total number of distinctive soil layers in the upper 100 ft of the site profile.

i = any one of the layers between 1 and n .

Client Fuss & O'Neill

Project Oceanville Road over Oceanville Thoroughfare, WIN 22630, Bridge No. 3696

Subject Seismic Site Class Evaluation

PROCEDURE**Method B**

Average standard penetration test (SPT) for the upper 100 ft of the soil profile:

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

where

N_i = standard penetration resistance as measured directly in the field, uncorrected blow count, of i th soil layer not to exceed 100 ft (blows/ft).

d_i = thickness of i th soil layer (ft).

n = total number of distinctive soil layers in the upper 100 ft of the site profile.

i = any one of the layers between 1 and n .

Method C

Average standard penetration test (SPT) for the cohesionless layers in the upper 100 ft of the soil profile:

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

where

N_i = standard penetration resistance as measured directly in the field, uncorrected blow count, of i th cohesionless soil layer (blows/ft).

d_i = thickness of i th cohesionless soil layer (ft).

m = total number of distinctive cohesionless soil layers in the upper 100 ft of the site profile.

i = any one of the layers between 1 and m .

Average undrained shear strength for the cohesive layers in the upper 100 ft of the soil profile:

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

where

s_{ui} = undrained shear strength of i th cohesive soil layer (psf), not to exceed 5000 psf

d_i = thickness of i th cohesive soil layer (ft).

k = total number of distinctive cohesive soil layers in the upper 100 ft of the site profile.

i = any one of the layers between 1 and k .

Based on the available information, Method C will be used for the seismic Site Class evaluation.

Client Fuss & O'Neill

Date 4-Oct-23

Project Oceanville Road over Oceanville Thoroughfare, WIN 22630, Bridge No. 3696

Computed by EMH

Subject Seismic Site Class Evaluation

Checked by MMB

SITE CLASS DEFINITIONS

(Table from AASHTO Guide Specifications for LRFD Seismic Bridge Design, 2nd Edition, 2011 (with 2012 Interim Revisions)).

Table 3.4.2.1-1—Site Class Definitions

Site Class	Soil Type and Profile
A	Hard rock with measured shear wave velocity, $\bar{v}_s > 5000$ ft/sec
B	Rock with $2500 \text{ ft/sec} < \bar{v}_s < 5000 \text{ ft/sec}$
C	Very dense soil and soil rock with $1200 \text{ ft/sec} < \bar{v}_s < 2500 \text{ ft/sec}$, or with either $\bar{N} > 50$ blows/ft or $\bar{s}_u > 2.0$ ksf
D	Stiff soil with $600 \text{ ft/sec} < \bar{v}_s < 1200 \text{ ft/sec}$, or with either $15 \text{ blows/ft} < \bar{N} < 50 \text{ blows/ft}$ or $1.0 \text{ ksf} < \bar{s}_u < 2.0 \text{ ksf}$
E	Soil profile with $\bar{v}_s < 600 \text{ ft/sec}$, or with either $\bar{N} < 15 \text{ blows/ft}$ or $\bar{s}_u < 1.0 \text{ ksf}$, or any profile with more than 10 ft of soft clay defined as soil with $PI > 20$, $w > 40\%$, and $\bar{s}_u < 0.5 \text{ ksf}$
F	Soils requiring site-specific ground motion response evaluations, such as: - Peats or highly organic clays ($H > 10$ ft of peat or highly organic clay, where H = thickness of soil) - Very high plasticity clays ($H > 25$ ft with $PI > 75$) - Very thick soft/medium stiff clays ($H > 120$ ft)

Exceptions:

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

where:

$\bar{v}_s$ = average shear wave velocity for the upper 100 ft of the soil profile as defined in Article 3.4.2.2

$\bar{N}$ = average standard penetration test (SPT) blow count (blows/ft) (ASTM D 1586) for the upper 100 ft of the soil profile as defined in Article 3.4.2.2

$\bar{s}_u$ = average undrained shear strength in ksf (ASTM D 2166 or D 2850) for the upper 100 ft of the soil profile as defined in Article 3.4.2.2

PI = plasticity index (ASTM D 4318)

w = moisture content (ASTM D 2216)

CALCULATIONS

File No. 42835-200
 Sheet 4 of 7
 Date 4-Oct-23
 Computed by EMH
 Checked by MMB

Client Fuss & O'Neill
 Project Oceanville Road over Oceanville Thoroughfare, WIN 22630, Bridge No. 3696
 Subject Seismic Site Class Evaluation

CALCULATIONS - METHOD C

Boring No. BB-SOT-101
 Est. Ground Surface El.: 12.4

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	Cohesionless		Cohesive	
					SPT N (blows/ft)	d/N	Su (psf)	d/Su
1D	1	11.4	SAND (Fill)	3.0	13	0.231		
2D	3	9.4	SAND (Fill)	2.0	11	0.182		
3D	5	7.4	GRAVEL (Fill)	3.0	10	0.300		
4D	8	4.4	SAND (Fill)	2.0	28	0.071		
5D	10	2.4	SAND (Fill)	3.7	13	0.285		
6D	14	-1.3	SAND (Fill)	5.7	35	0.163		
7D	20	-7.6	SAND (Glacial Till)	3.6	31	0.116		
R1/R2	23	-10.6	BEDROCK	77.0	100	0.770		
					$\Sigma d/N =$	2.118	$\Sigma d/Su =$	0.000
Total Thickness of Cohesionless (ft) =					100.0	N_{ch-bar} (blows/ft) =	47.2	S_{u-bar} (psf) = #DIV/0!
Total Thickness of Cohesive (ft) =					0.0	Site Class _N =	D	Site Class _{Su} = #DIV/0!
Total Thickness (ft) =					100.0	Site Class =	D	

CALCULATIONS

File No. 42835-200
 Sheet 5 of 7
 Date 4-Oct-23
 Computed by EMH
 Checked by MMB

Client Fuss & O'Neill
 Project Oceanville Road over Oceanville Thoroughfare, WIN 22630, Bridge No. 3696
 Subject Seismic Site Class Evaluation

CALCULATIONS - METHOD C

Boring No. BB-SOT-102
 Est. Ground Surface El.: 12.2

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	Cohesionless		Cohesive	
					SPT N (blows/ft)	d/N	Su (psf)	d/Su
1D	1	11.2	SAND (Fill)	3.0	17	0.176		
2D	5	7.2	SAND (Fill)	6.0	22	0.273		
MD	9	3.2	(Fill)	6.0	75	0.080		
3D	15	-2.8	SAND (Harbor Bottom Deposit)	3.6	21	0.171		
4D	20	-7.8	Silty CLAY (Marine Deposit)	2.9			400	0.007
5D	24	-11.5	Silty SAND (Glacial Till)	3.0	100	0.030		
R1-R3	25	-12.3	BEDROCK	75.5	100	0.755		
					$\Sigma d/N =$	1.486	$\Sigma d/Su =$	0.007
Total Thickness of Cohesionless (ft) =				97.1	N_{ch-bar} (blows/ft) =	65.4	S_{u-bar} (psf) =	400
Total Thickness of Cohesive (ft) =				2.9	Site Class _N =	C	Site Class _{Su} =	E
Total Thickness (ft) =				100.0	Site Class =	D		

CALCULATIONS

File No.	42835-200
Sheet	6 of 7
Date	4-Oct-23
Computed by	EMH
Checked by	MMB

Client	Fuss & O'Neill
Project	Oceanville Road over Oceanville Thoroughfare, WIN 22630, Bridge No. 3696
Subject	Seismic Site Class Evaluation

CALCULATIONS - METHOD C

Boring No.	BB-SOT-201
Est. Ground Surface El.:	11.8

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	Cohesionless		Cohesive	
					SPT N (blows/ft)	d/N	Su (psf)	d/Su
1D	1	10.8	SAND (Fill)	5.0	10	0.500		
2D	5	6.8	SAND (Fill)	5.0	25	0.200		
3D	10	1.8	GRAVEL (Fill)	3.5	20	0.175		
4D	15	-3.2	Silty CLAY (Marine Deposit)	2.0			1000	0.002
5D/A	17	-5.2	Silty CLAY (Marine Deposit)	2.9			1000	0.003
6D	20	-8.2	SAND (Glacial Till)	6.6	16	0.413		
7D	25	-13.2	SAND (Glacial Till)	0.3	100	0.003		
R1-R5	25	-13.5	BEDROCK	74.7	100	0.747		
					$\Sigma d/N =$	2.038	$\Sigma d/Su =$	0.005
Total Thickness of Cohesionless (ft) =				95.1	N_{ch-bar} (blows/ft) =	46.7	S_{u-bar} (psf) =	1000
Total Thickness of Cohesive (ft) =				4.9	Site Class _N =	D	Site Class _{Su} =	D
Total Thickness (ft) =				100.0	Site Class =	D		

CALCULATIONS

File No.	42835-200
Sheet	7 of 7
Date	4-Oct-23
Computed by	EMH
Checked by	MMB

Client	Fuss & O'Neill
Project	Oceanville Road over Oceanville Thoroughfare, WIN 22630, Bridge No. 3696
Subject	Seismic Site Class Evaluation

CALCULATIONS - METHOD C

Boring No.	BB-SOT-202
Est. Ground Surface El.:	11.7

Sample Number	Depth (ft)	Elevation (ft)	Description	d (ft)	Cohesionless		Cohesive	
					SPT N (blows/ft)	d/N	Su (psf)	d/Su
1D	1	10.7	SAND (Fill)	4.0	19	0.211		
2D	5	6.7	SAND (Fill)	5.7	15	0.380		
3D/A	10	1.7	Silty SAND (Harbor Bottom Deposit)	1.4	1	1.400		
4D	12	-0.3	Organic SILT (Harbor Bottom Deposit)	4.7			400	0.012
5D/A	15	-3.3	Silty GRAVEL (Harbor Bottom Deposit)	4.2	8	0.525		
6D/A	20	-8.3	Silty CLAY (Marine Deposit)	4.5			1000	0.005
R1-R6	25	-12.8	BEDROCK	75.5	100	0.755		
					$\Sigma d/N =$	3.271	$\Sigma d/Su =$	0.016
Total Thickness of Cohesionless (ft) =				90.8	N_{ch-bar} (blows/ft) =	27.8	S_{u-bar} (psf) =	566
Total Thickness of Cohesive (ft) =				9.2	Site Class _N =	D	Site Class _{Su} =	E
Total Thickness (ft) =				100.0	Site Class =	E		

Liquefaction

PROBLEM STATEMENT & OBJECTIVE

Calculate factor of safety against liquefaction using the procedure in Youd and Idriss (2001) for SPT N data.

EXECUTIVE SUMMARY

The minimum F.S. against liquefaction based on the available information is 2.1; therefore, site soils are not liquefaction susceptible.

REFERENCES

1. Youd and Idriss, "Liquefaction Resistance of Soils: Summary Report for the 1996 NCEER and 1998 NCEER/NSF Workshop on Evaluation of Liquefaction Resistance of Soils," 2001.

AVAILABLE INFORMATION

- 1. Preliminary boring logs BB-SOT-101 and BB-SOT-102 drilled in March 2016 and design phase boring logs BB-SOT-201 and BB-SOT-202 drilled in June 2022.
- 2. The site is Seismic Site Class D, and the site-modified Peak Ground Acceleration, A_s , is 0.083g.
- 3. Deaggregation plot of the site from the United States Geological Survey (USGS) Earthquake Hazards website.
- 4. Assumed groundwater for analysis at El. 5 (Mean Higher High Water from plan set received from Fuss & O'Neill on 12/30/22).
- 5. Geotechnical laboratory test results for sieve gradation by GeoTesting Express.

ASSUMPTIONS

- 1. Elevations reference the North American Vertical Datum of 1988 (NAVD88).
- 2. Fines content for samples from preliminary borings were estimated based on field geologist log descriptions correlated to the Maine Department of Transportation key to soil and rock description and terms.
- 3. Fines content for following samples was determined from sieve gradation laboratory testing:

Boring No.	Sample No.	Depth (ft)	% Fines
BB-SOT-101	2D	3-5	13.0
BB-SOT-101	5D	10-12	13.1
BB-SOT-102	1D	1-3	23.3
BB-SOT-102	3D	15-17	9.4

- 4. Total unit weights used in the analysis were estimated as follows: Fill=115 pcf, Harbor Bottom Depsoits=100 pcf, Marine Deposits=110 pcf, and Glacial Till=130 pcf.
- 5. SPT N_{60} values were provided on the boring logs. The SPT N_{60} values were used in the analysis.
- 6. An SPT value of "1" was assigned to samples with WOR or WOH. An SPT value of "100" was assigned to samples that encountered refusal.
- 7. Groundwater is at El. 5.
- 8. Magnitude $M=5.65$ based on deaggregation plot and corresponding $MSF=2.06$ based on equation (24) in Youd & Idriss (2001).

PROCEDURE

1. Evaluate the Cyclic Stress Ratio:

The Cyclic Stress Ratio (CSR) is defined as $CSR = (\tau_{av}/\sigma'_{vo}) = 0.65(\frac{a_{max}}{g})(\frac{\sigma_{vo}}{\sigma'_{vo}})r_d$

a_{max} = peak horizontal acceleration at the ground surface generated by the earthquake (A_s *g)
g = acceleration of gravity
 σ_{vo} = total vertical overburden stress
 σ'_{vo} = effective vertical overburden stress
 r_d = stress reduction coefficient
 $r_d = 1 - 0.00765z$ for $z \leq 9.15$ m
 $r_d = 1.174 - 0.0267z$ for $9.15 \text{ m} < z \leq 23 \text{ m}$
z = depth below ground surface in meters

2. Evaluate the Cyclic Resistance Ratio:

The Cyclic Resistance Ratio for a magnitude 7.5 earthquake ($CRR_{7.5}$) is defined as $CRR_{7.5} = \frac{1}{34-(N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10*(N_1)_{60} + 45]^2} - \frac{1}{200}$ for clean sands

where $(N_1)_{60}$ = SPT blow count normalized to an overburden pressure of approximately 100 KPa (1 tsf) and a hammer energy ratio or hammer efficiency of 60%.

$(N_1)_{60} = C_N * N_{60}$

$C_N = \frac{P_a^{0.5}}{\sigma'_{vo}} (\leq 1.7)$

The above is valid for $(N_1)_{60} < 30$
for $(N_1)_{60} \geq 30$, clean granular soils are too dense to liquefy and are classed as non-liquefiable (NL)

To consider the influence of fines content (FC), a correction to the $(N_1)_{60}$ to an equivalent clean sand is made $(N_1)_{60\text{cs}} = \alpha + \beta(N_1)_{60}$
where α and β are coefficients determined from the following relationships:

$\alpha = 0$ for $FC \leq 5\%$
 $\alpha = \exp[1.76-(190/FC^2)]$ for $5\% < FC < 35\%$
 $\alpha = 5.0$ for $FC \geq 35\%$
 $\beta = 1.0$ for $FC \leq 5\%$
 $\beta = [0.99 + (FC^{1.5}/1000)]$ for $5\% < FC < 35\%$
 $\beta = 1.2$ for $FC \geq 35\%$

3. Evaluate the Factor of Safety Against Liquefaction:

where $F.S. = \frac{CRR_{7.5}}{CSR} * MSF$

MSF = magnitude scaling factor (equation (24) in Youd & Idriss, 2001)
 M_w = moment magnitude of earthquake

CALCULATIONS

Boring No.	BB-SOT-101	input value
Estimated Ground Surface Elevation	12.4	ft, NAVD 88
Groundwater Elevation	5	ft, NAVD 89
Depth of Groundwater	7.4	ft
Unit Weight of Water	64	pcf
A _s (F _{PGA} *PGA)	0.083	g
Magnitude (based on mean magnitude from deaggregation plot)	5.65	
Magnitude Scaling Factor	2.06	

Sample No.	Depth (ft)	Elevation (ft)	Geologic Unit	USCS Symbol	SPT N ₆₀	Y _t (pcf)	σ _v (psf)	σ _w (psf)	σ' _v (psf)	C _N	(N ₁) ₆₀	Fines Content (%)	α	β	(N ₁) _{60-CS}	CRR _{7.5}	r _d	CSR	FS
1D	1.0	11.4	FILL	SW	12	115	115	0	115	1.70	20	0	0.00	1.00	20	0.220	1.00	0.054	8.5
2D	3.0	9.4	FILL	SW-SM	10	115	345	0	345	1.70	17	13	1.89	1.04	20	0.209	0.99	0.054	8.1
3D	5.0	7.4	FILL	GP	10	115	575	0	575	1.70	17	0	0.00	1.00	17	0.181	0.99	0.053	7.0
4D	8.0	4.4	FILL	SW	27	115	920	38	882	1.54	42	11	1.21	1.03	44	NL	0.98	0.055	NL
5D	10.0	2.4	FILL	SW	12	115	1,150	166	984	1.46	17	13.1	1.92	1.04	20	0.216	0.98	0.062	7.2
6D	13.7	-1.3	FILL	SW	33	115	1,576	403	1,172	1.33	44	0	0.00	1.00	44	NL	0.97	0.070	NL
7D	20.0	-7.6	GLACIAL TILL	SM	29	130	2,600	806	1,794	1.08	31	21	3.78	1.09	38	NL	0.95	0.075	NL

Boring No.	BB-SOT-102	input value
Estimated Ground Surface Elevation	12.2	ft, NAVD 88
Groundwater Elevation	5	ft, NAVD 89
Depth of Groundwater	7.2	ft
Unit Weight of Water	64	pcf
A _s (F _{PGA} *PGA)	0.083	g
Magnitude (based on mean magnitude from deaggregation plot)	5.65	
Magnitude Scaling Factor	2.06	

Sample No.	Depth (ft)	Elevation (ft)	Geologic Unit	USCS Symbol	SPT N ₆₀	Y _t (pcf)	σ _v (psf)	σ _w (psf)	σ' _v (psf)	C _N	(N ₁) ₆₀	Fines Content (%)	α	β	(N ₁) _{60-CS}	CRR _{7.5}	r _d	CSR	FS
1D	1.0	11.4	FILL	SM	16	115	115	0	115	1.70	27	23.3	4.10	1.10	34	NL	1.00	0.054	NL
2D	5.0	7.4	FILL	SW	21	115	575	0	575	1.70	36	11	1.21	1.03	38	NL	0.99	0.053	NL
MD	9.0	3.4	FILL		71	115	1,035	115	920	1.51	107	0	0.00	1.00	107	NL	0.98	0.059	NL
3D	15.0	-2.6	HARBOR BOTTOM DEPOSIT	SW	20	105	1,575	499	1,076	1.39	28	9.4	0.68	1.02	29	0.413	0.97	0.076	11.2
4D	20.0	-7.6	MARINE DEPOSIT	CL	1	110	2,200	819	1,381	1.23	1	50	5.00	1.20	6	0.083	0.95	0.082	2.1
5D	23.7	-11.3	GLACIAL TILL	SM	100	130	3,081	1,056	2,025	1.02	102	36	5.00	1.20	127	NL	0.94	0.078	NL

NL = Not Liquefiable

CALCULATIONS

Boring No.	BB-SOT-201	input value
Estimated Ground Surface Elevation	11.8	ft, NAVD 88
Groundwater Elevation	5	ft, NAVD 89
Depth of Groundwater	6.8	ft
Unit Weight of Water	64	pcf
A _s (F _{PGA} *PGA)	0.083	g
Magnitude (based on mean magnitude from deaggregation plot)	5.65	
Magnitude Scaling Factor	2.06	

Sample No.	Depth (ft)	Elevation (ft)	Geologic Unit	USCS Symbol	SPT N ₆₀	Y _t (pcf)	σ _v (psf)	σ _w (psf)	σ' _v (psf)	C _N	(N ₁) ₆₀	Fines Content (%)	α	β	(N ₁) _{60-CS}	CRR _{7.5}	r _d	CSR	FS
1D	1.0	10.8	FILL	SW	15	115	115	0	115	1.70	26	0	0.00	1.00	26	0.302	1.00	0.054	11.6
2D	5.0	6.8	FILL	SW	38	115	575	0	575	1.70	65	0	0.00	1.00	65	NL	0.99	0.053	NL
3D	10.0	1.8	FILL	GW	31	115	1,150	205	945	1.49	46	11	1.21	1.03	49	NL	0.98	0.064	NL
4D	15.0	-3.2	MARINE DEPOSIT	CL	14	110	1,650	525	1,125	1.36	19	50	5.00	1.20	28	0.366	0.97	0.076	9.9
5D/A	17.0	-5.2	MARINE DEPOSIT/GLACIAL TILL	CL/SP	6	110	1,870	653	1,217	1.31	8	11	1.21	1.03	9	0.107	0.96	0.080	2.8
6D	20.0	-8.2	GLACIAL TILL	SP	25	130	2,600	845	1,755	1.09	27	11	1.21	1.03	29	0.420	0.95	0.076	11.4
7D	25.0	-13.2	GLACIAL TILL	SW	100	130	3,250	1,165	2,085	1.00	100	11	1.21	1.03	104	NL	0.94	0.079	NL

Boring No.	BB-SOT-202	input value
Estimated Ground Surface Elevation	11.7	ft, NAVD 88
Groundwater Elevation	5	ft, NAVD 89
Depth of Groundwater	6.7	ft
Unit Weight of Water	64	pcf
A _s (F _{PGA} *PGA)	0.083	g
Magnitude (based on mean magnitude from deaggregation plot)	5.65	
Magnitude Scaling Factor	2.06	

Sample No.	Depth (ft)	Elevation (ft)	Geologic Unit	USCS Symbol	SPT N ₆₀	Y _t (pcf)	σ _v (psf)	σ _w (psf)	σ' _v (psf)	C _N	(N ₁) ₆₀	Fines Content (%)	α	β	(N ₁) _{60-CS}	CRR _{7.5}	r _d	CSR	FS
1D	1.0	10.8	FILL	SW	29	115	115	0	115	1.70	49	0	0.00	1.00	49	NL	1.00	0.054	NL
2D	5.0	6.8	FILL	SW	23	115	575	0	575	1.70	39	0	0.00	1.00	39	NL	0.99	0.053	NL
3D/A	10.0	1.8	HARBOR BOTTOM DEPOSIT	SM	2	105	1,050	211	839	1.58	3	36	5.00	1.20	9	0.103	0.98	0.066	3.2
4D	12.0	-0.2	HARBOR BOTTOM DEPOSIT	OL	1	105	1,260	339	921	1.51	2	50	5.00	1.20	7	0.086	0.97	0.072	2.5
5D/A	15.0	-3.2	HARBOR BOTTOM DEPOSIT	GM	12	105	1,575	531	1,044	1.41	17	36	5.00	1.20	25	0.299	0.97	0.079	7.9
6D/A	20.0	-8.2	MARINE DEPOSIT/GLACIAL TILL	SM	9	130	2,600	851	1,749	1.09	10	21	3.78	1.09	14	0.155	0.95	0.076	4.2

NL = Not Liquefiable

Global Stability

Client: Fuss & O'Neill

Project: Oveanville Road Bridge over Oceanville Thoroughfare, Bridge No. 3696, Stonington, ME

Subject: Global Stability

PROBLEM STATEMENT AND OBJECTIVE

Calculate the global stability minimum factor of safety for static and pseudostatic analyses.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th Edition, 2020.
2. Slide version 9.0 by RocScience.

AVAILABLE INFORMATION

1. Plan set from Fuss & O'Neill, received on December 30, 2022.
2. Boring logs drilled in March 2016 and June 2022 by New England Boring Contractors.

ASSUMPTIONS

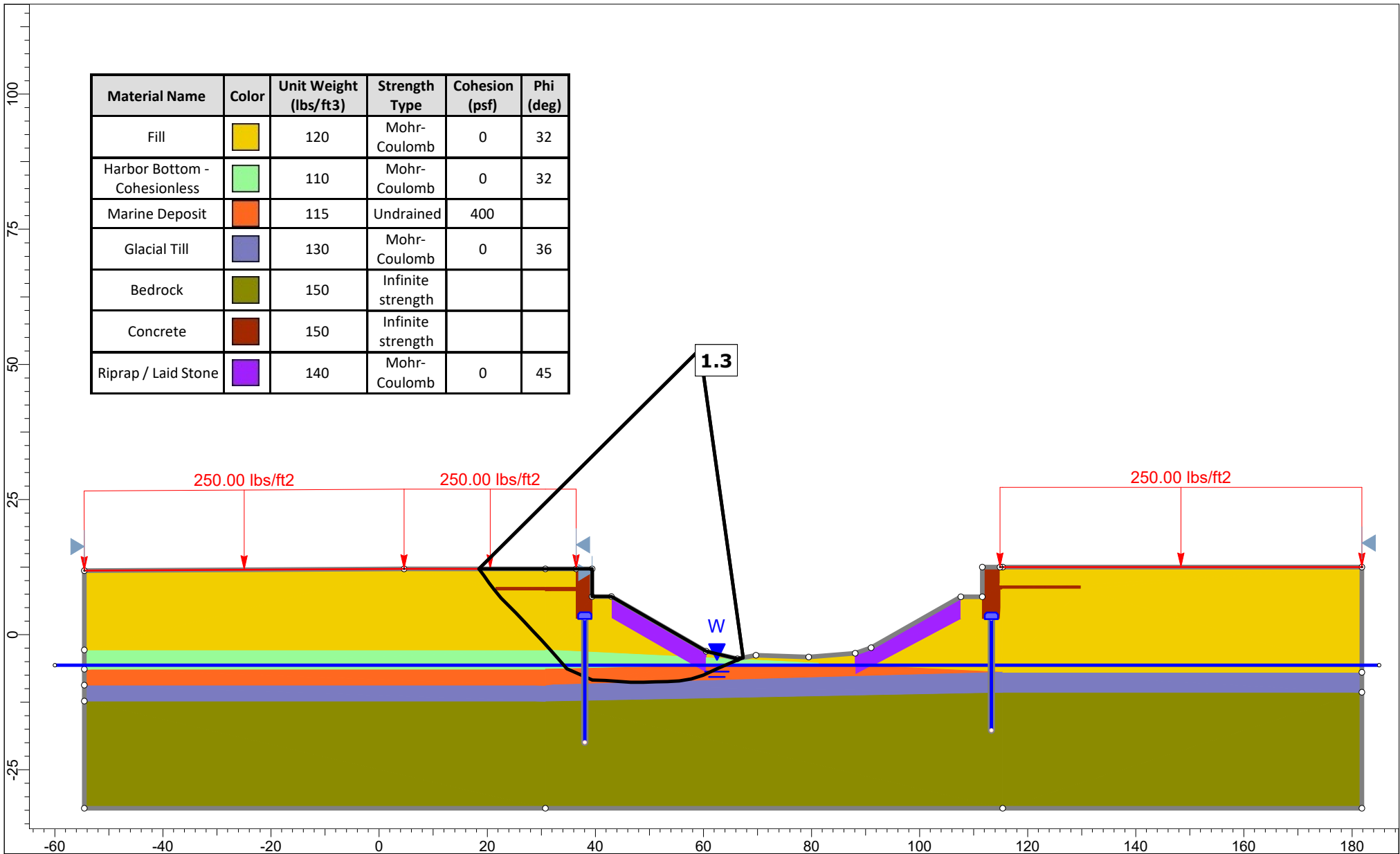
1. Abutments will be modeled with infinite strength with a unit weight of 150 pcf.
2. Water level will be modeled at El. -5.65 (MLLW) based drawings provided by Fuss & O'Neill.
3. Seismic cases will have a seismic force of $A_s/2$ ($0.083g/2$) = 0.0415 g based on the seismic site class calculations.
4. Micropiles supporting the abutments will be modeled with 4.5 ft spacing, per email correspondence with Fuss & O'Neill on July 24, 2023.
5. Micropiles will be modeled with a shear strength of 9 kips, based on the maximum shear in the micropile from an LPile analysis considering 1-inch deflection at the top of the micropile. The following assumptions were made in LPile:
 - a. The micropile will have a 9.625" OD with 0.545" wall thickness, per email correspondence with Fuss & O'Neill on July 24, 2023.
 - b. The micropile will have a #11 bar in the center.
 - c. Refer to the "Lateral Pile Resistance" section in the report text for soil and rock properties used in the LPile analysis. The soil properties for Abutment 1 were modeled, and the rock socket was assumed to extend 10 ft into bedrock.
6. Refer to the Slide outputs on the following pages for material properties used in the global stability analysis.
7. A 250 psf traffic surcharge will be modeled.

RESULTS AND CONCLUSIONS

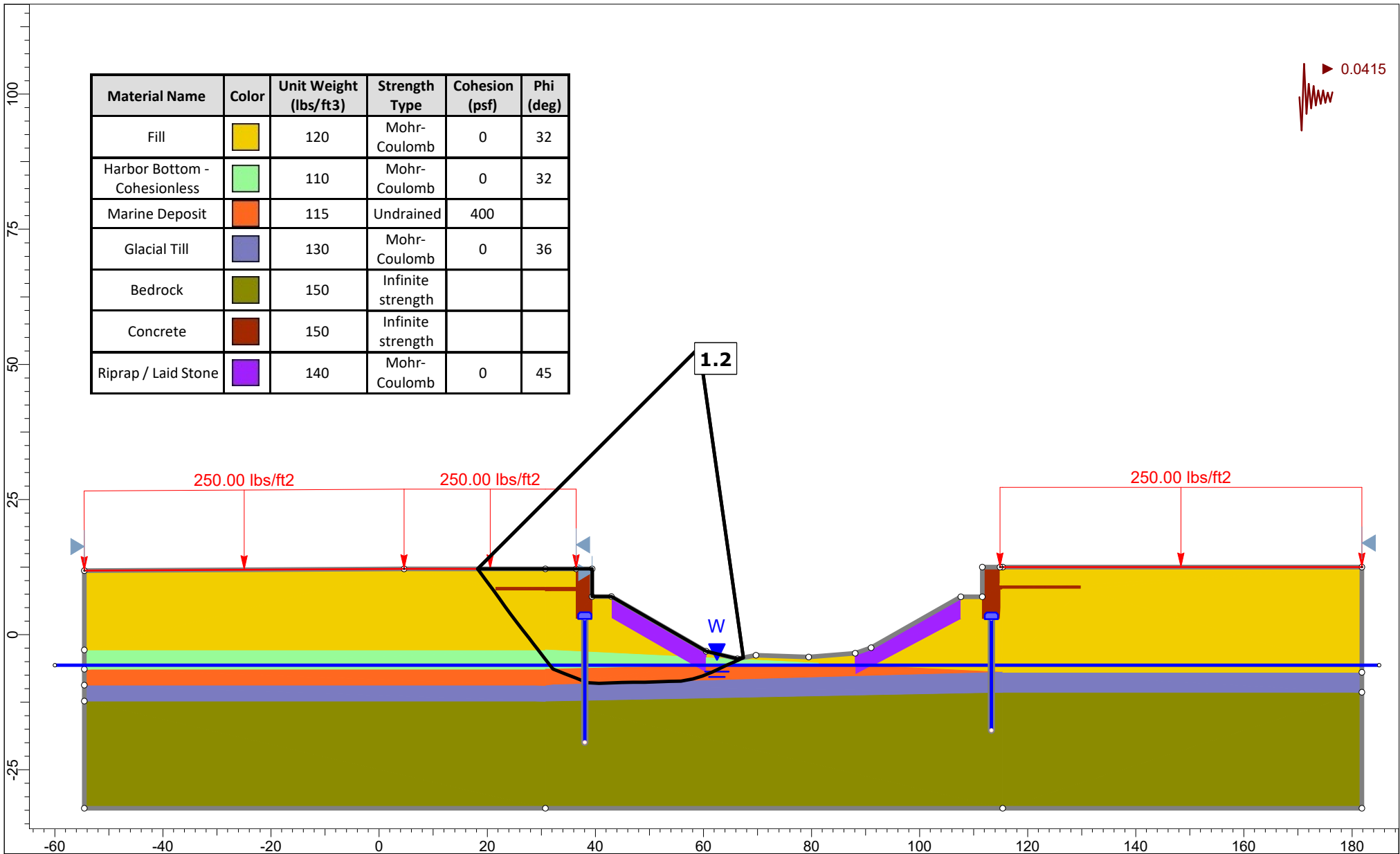
Section	Factor of Safety	
	Static	Pseudostatic
Longitudinal, 100-Series Borings	1.3	1.2
Longitudinal, 200-Series Borings	1.3	1.2
Transverse - Sta. 106+25	1.3	1.1

Based on AASHTO LRFD Section 11.6.3.7, an acceptable resistance factor where the geotechnical parameters and subsurface stratigraphy are well defined is 0.75 ($F.S. = 1/0.75 = 1.3$).

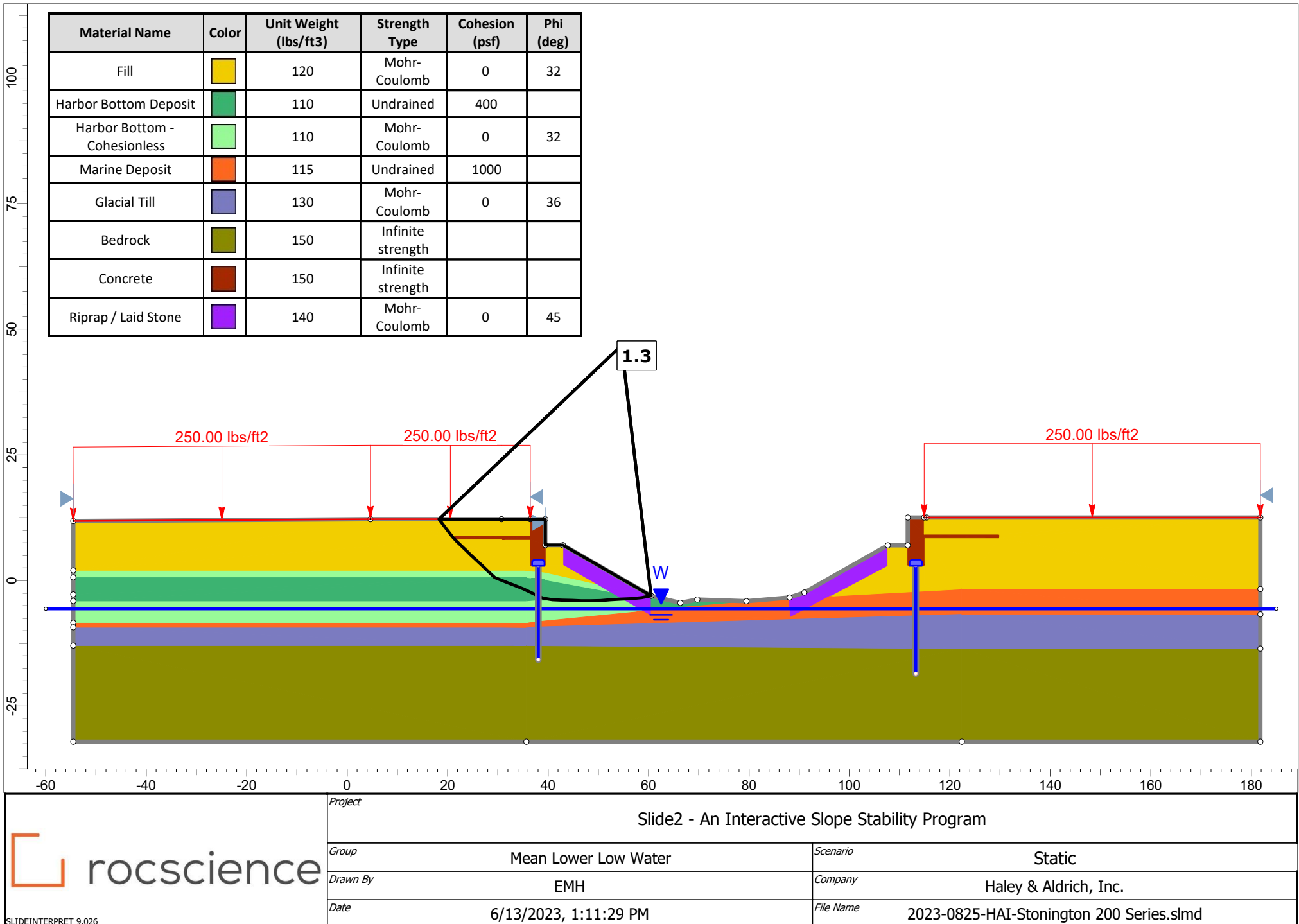
Based on AASHTO LRFD Section 11.5.8, an acceptable resistance factor when earthquake loading is included is 0.9 ($F.S. = 1/0.9 = 1.1$).

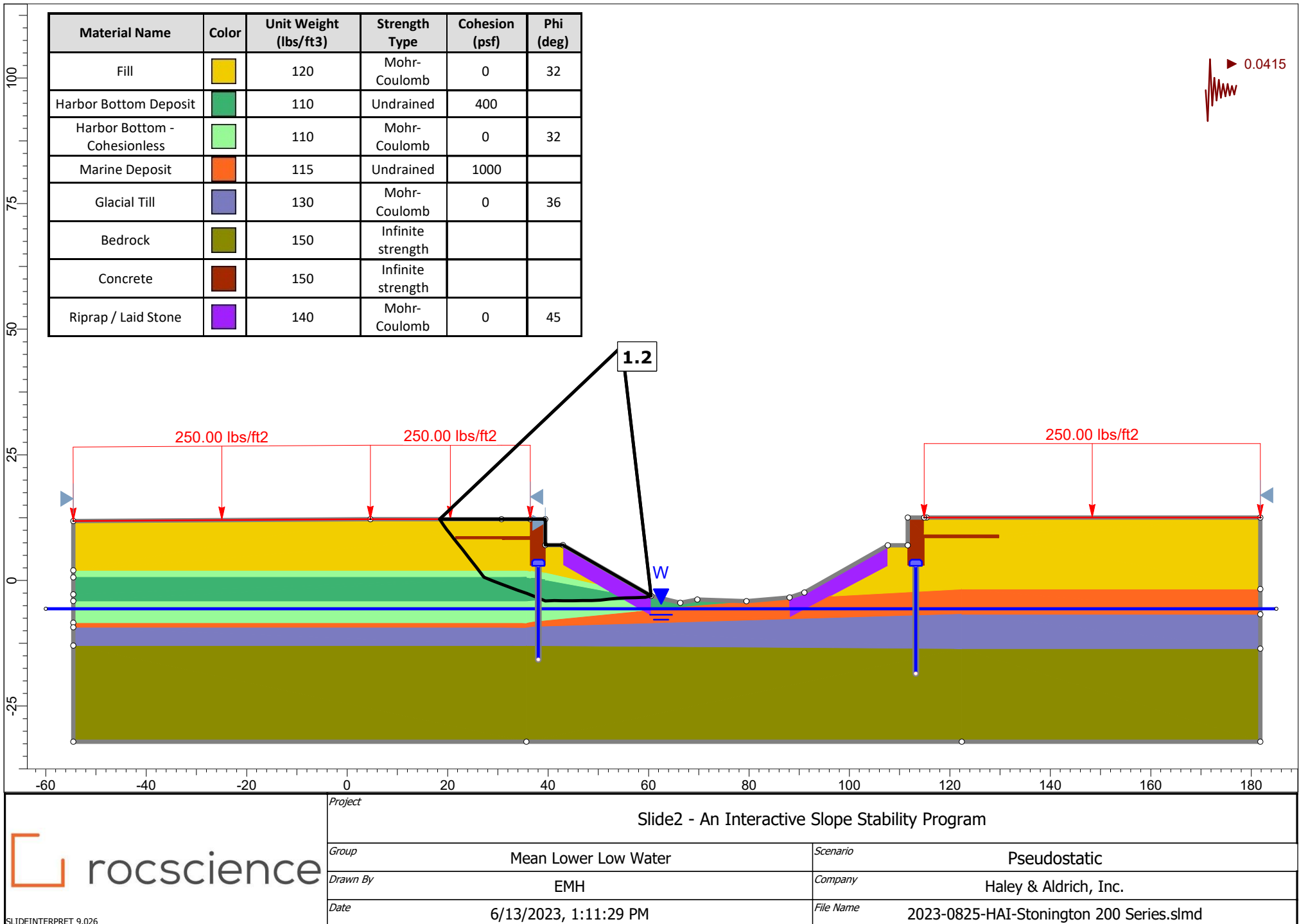


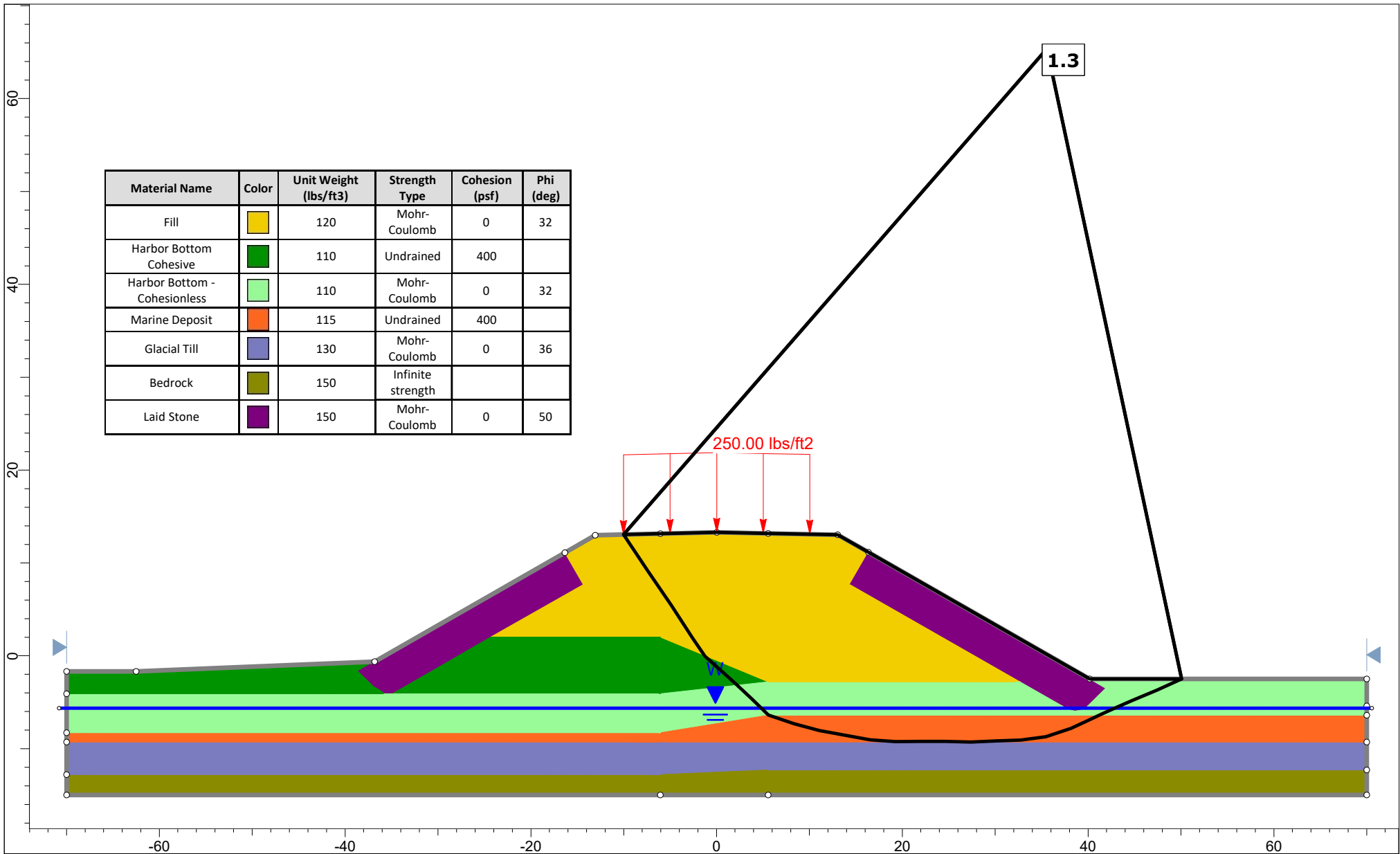
	Project		Slide2 - An Interactive Slope Stability Program	
	Group	Mean Lower Low Water	Scenario	Static
	Drawn By	EMH	Company	Haley & Aldrich, Inc.
	Date	6/13/2023, 1:11:29 PM	File Name	2023-0613-HAI-Stonington 100 Series.slmd
	SLIDEINTERPRET 9.026			



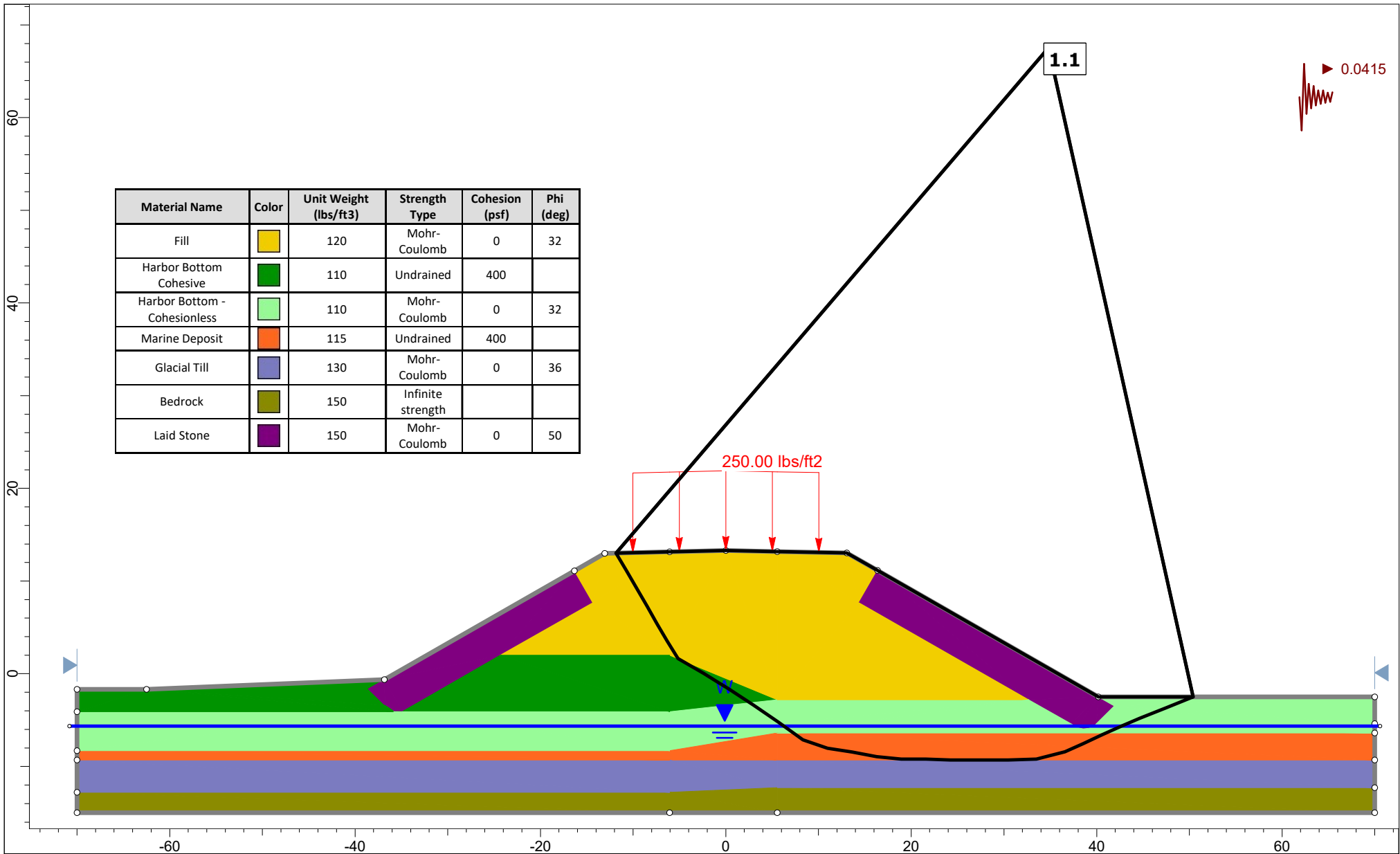
	Project		Slide2 - An Interactive Slope Stability Program	
	Group	Mean Lower Low Water	Scenario	Pseudostatic
	Drawn By	EMH	Company	Haley & Aldrich, Inc.
	Date	6/13/2023, 1:11:29 PM	File Name	2023-0613-HAI-Stonington 100 Series.slmd
	SLIDEINTERPRET 9.026			







	Project		Slide2 - An Interactive Slope Stability Program	
	Group	Mean Lower Low Water	Scenario	Static
	Drawn By	EMH	Company	Haley & Aldrich, Inc.
	Date	3/7/2023, 1:29:33 PM	File Name	2023-0825-HAI-Stonington 106+25.slmd
	SLIDEINTERPRET 9.026			



rocscience

SLIDEINTERPRET 9.026

Project		Slide2 - An Interactive Slope Stability Program	
Group	Mean Lower Low Water	Scenario	Pseudostatic
Drawn By	EMH	Company	Haley & Aldrich, Inc.
Date	3/7/2023, 1:29:33 PM	File Name	2023-0825-HAI-Stonington 106+25.slmd

Micropile Axial Compressive Resistance

Client: Fuss & O'Neill

Project: Oceanville Bridge pver Oceanville Thoroughfare

Subject: LPile Parameters for Rock and Micropile Side Resistance

PROBLEM STATEMENT & OBJECTIVE

Determine p-y curves for bedrock at each abutment. Determine the factored geotechnical unit side resistance for 9" diameter rock socketed micropiles under the strength limit state.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th Edition, 2020.
2. NCHRP Synthesis 360 - Rock Socketed Shafts for Highway Structure Foundations, 2010

AVAILABLE INFORMATION

1. Borings logs drilled in March 2016 and June 2022 by New England Boring Contractors.
2. Rock compressive strength laboratory data from GeoTesting Express.

ASSUMPTIONS

1. Micropile diameter = 9.625 in (OD). Socket diameter below casing = 9.0 in.
2. Axial group effects are not considered.
3. Resistance from soils are not considered. Micropile end-bearing resistance is assumed zero.

CONCLUSION

1. For Abutment 1, we recommend the following customized p-y curve for the bedrock:

y (in)	P (lb/in)
0	0
0.01	18,000
10	18,000

Selected k = 200,000 pci
Selected pL+tmax = 2,000 psi

2. For Abutment 2, we recommend the following customized p-y curve for the bedrock:

y (in)	P (lb/in)
0	0
0.01	36,000
10	36,000

Selected k = 300,000 pci
Selected pL+tmax = 4,000 psi

3. We recommend a factored unit side resistance of **18.6** ksf for the Strength Limit State. See Page 10 of this calculation for factored geotechnical resistance vs. depth.

Client: Fuss & O'Neill

Project: Oceanville Bridge pver Oceanville Thoroughfare

Computed by: EMH

Subject: LPile Parameters for Rock and Micropile Side Resistance

Checked by: MMB

PROCEDURE for ROCK P-Y CURVES**From NCHRP and LRFD 2012:**

The procedure for establishing values of subgrade modulus K_h involves determination of the rock mass modulus (E_m) from PMT measurements. The coefficient of subgrade reaction is then given by:

$$k_h = \frac{0.65 E_m}{B(1 - \nu_r^2)} \left[\frac{E_m B^4}{E_s I_s} \right]^{1/2} \quad \left(\text{units: } \frac{F}{l^3} \right) \quad (109)$$

in which B = shaft diameter, E_m = rock mass modulus, ν_r = Poisson's ratio of the rock, and E_s and I_s are modulus and moment of inertia of the shaft, respectively. A procedure is given by Gabr et al. (2002) for establishing the point of rotation of the shaft. For p -y curves above the point of rotation,

From LRFD 2020:Table 10.4.6.5-1—Estimation of E_m Based on GSI

Expression	Notes/Remarks	Reference
$E_m \text{ (GPa)} = \sqrt{\frac{q_u}{100}} \frac{GSI-10}{40}$ for $q_u \leq 100$ MPa	Accounts for rocks with $q_u < 100$ MPa; notes q_u in MPa	Hoek and Brown (1997); Hoek et al. (2002)
$E_m \text{ (GPa)} = 10 \frac{GSI-10}{40}$ for $q_u > 100$ MPa		
$E_m = \frac{E_R}{100} e^{GSI/21.7}$	Reduction factor on intact modulus, based on GSI	Yang (2006)

Notes: E_r = modulus of intact rock, E_m = equivalent rock mass modulus, GSI = geological strength index, q_u = uniaxial compressive strength, and 1 MPa = 20.9 ksf.

TABLE 13
ESTIMATION OF MODULUS RATIO (E_m/E_R)
BASED ON RQD (O'Neill et al. 1996)

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

RQD = rock quality designation.

The second required hyperbolic curve parameter is the asymptote of the p -y curve, which is the ultimate resistance p_{ult} . The proposed expression is given by

$$p_{ult} = (p_L + \tau_{max})B \quad (111)$$

where p_L = limit normal stress and τ_{max} = shearing resistance along the side of the shaft. Gabr et al. adopted the following recommendation of Zhang et al. (2000) for unit side resistance:

$$\tau_{max} = 0.20 \sqrt{q_u \text{ (MPa)}} \quad (112)$$

The limit normal stress is estimated on the basis of Hoek–Brown strength parameters as determined through correlations with RMR and GSI, and is given by

$$p_L = \gamma' z + q_u \left(m_b \frac{\gamma' z}{q_u} + s \right)^a \quad (113)$$

in which γ' = effective unit weight of the rock mass, z = depth from the rock mass surface, and the coefficients m_b , s , and a are the Hoek–Brown coefficients given by Eqs. 12–15 in chapter two.

Client: Fuss & O'Neill

Project: Oceanville Bridge pver Oceanville Thoroughfare

Computed by: EMH

Subject: LPile Parameters for Rock and Micropile Side Resistance

Checked by: MMB

PROCEDURE for MICROPILE SIDE RESISTANCE

From LRFD:

GEOLOGICAL STRENGTH INDEX FOR JOINTED ROCKS (Hoek and Marinos, 2000)		SURFACE CONDITIONS				
From the lithology, structure and surface conditions of the discontinuities, estimate the average value of GSI. Do not try to be too precise. Quoting a range from 33 to 37 is more realistic than stating that GSI = 35. Note that the table does not apply to structurally controlled failures. Where weak planar structural planes are present in an unfavourable orientation with respect to the excavation face, these will dominate the rock mass behaviour. The shear strength of surfaces in rocks that are prone to deterioration as a result of changes in moisture content will be reduced if water is present. When working with rocks in the fair to very poor categories, a shift to the right may be made for wet conditions. Water pressure is dealt with by effective stress analysis. STRUCTURE DECREASING SURFACE QUALITY → DECREASING INTERLOCKING OF ROCK PIECES ↓		VERY GOOD Very rough, fresh unweathered surfaces	GOOD Rough, slightly weathered, iron stained surfaces	FAIR Smooth, moderately weathered and altered surfaces	POOR Slackens, highly weathered surfaces with compact coatings or fillings or angular fragments	VERY POOR Slackens, highly weathered surfaces with soft clay coatings or fillings
		INTACT OR MASSIVE - intact rock specimens or massive in situ rock with few widely spaced discontinuities	90		N/A	N/A
		BLOCKY - well interlocked undisturbed rock mass consisting of cubical blocks formed by three intersecting discontinuity sets	80	70		
		VERY BLOCKY - interlocked, partially disturbed mass with multi-faceted angular blocks formed by 4 or more joint sets		60	50	
		BLOCKY/DISTURBED/SEAMY - folded with angular blocks formed by many intersecting discontinuity sets. Persistence of bedding planes or schistosity			40	30
		DISINTEGRATED - poorly interlocked, heavily broken rock mass with mixture of angular and rounded rock pieces				20
		LAMINATED/SHEARED - Lack of blockiness due to close spacing of weak schistosity or shear planes	N/A	N/A		10

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

The strength of jointed rock masses should be evaluated using the Hoek-Brown failure criterion (Hoek et al., 2002). This nonlinear strength criterion is expressed in its general form as:

$$\sigma'_1 = \sigma'_3 + q_u \left(m_b \frac{\sigma'_1 - \sigma'_3}{q_u} + s \right)^a \quad (10.4.6.4-1)$$

in which:

$$s = e^{\left(\frac{GSI - 100}{9 - 3D} \right)} \quad (10.4.6.4-2)$$

$$a = \frac{1}{2} + \frac{1}{6} \left(e^{\frac{-GSI}{15}} - e^{\frac{-20}{3}} \right) \quad (10.4.6.4-3)$$

where:

- e = 2.718 (natural or Napierian log base)
- D = disturbance factor (dim)
- σ'_1 and σ'_3 = principal effective stresses (ksf)
- q_u = average unconfined compressive strength of rock core (ksf)
- m_b , s , and a = empirically determined parameters

The value of the constant m_b should be estimated from Table 10.4.6.4-1 based on lithology. Relationships between GSI and the parameters m_b , s , and a , according to Hoek et al. (2002) are as follows:

$$m_b = m_i e^{\left(\frac{GSI - 100}{28 - 14D} \right)} \quad (10.4.6.4-4)$$

Table 10.4.6.4-1—Values of the Constant m_b by Rock Group (after Marinos and Hoek 2000; with updated values from Rescension, Inc., 2007)

Rock type	Class	Group	Texture			
			Coarse	Medium	Fine	Very fine
SEDIMENTARY	Clastic		Conglomerate (21 ± 3)	Sandstone 17 ± 4	Siltstone 7 ± 2	Claystone 4 ± 2
			Breccia (19 ± 5)		Greywacke (18 ± 3)	Shale (6 ± 2)
					Marl (7 ± 2)	
	Non-Clastic	Carbonates	Crystalline Limestone (12 ± 3)	Sparitic Limestone (10 ± 5)	Micritic Limestone (8 ± 3)	Dolomite (9 ± 3)
		Evaporites		Gypsum 10 ± 2	Anhydrite 12 ± 2	
		Organic				Chalk 7 ± 2
METAMORPHIC	Non Foliated		Marble 9 ± 3	Hornfels (19 ± 4)	Quartzite 20 ± 3	
				Metasandstone (19 ± 3)		
	Slightly foliated		Migmatite (29 ± 3)	Amphibolite 26 ± 6	Gneiss 28 ± 5	
		Foliated*		Schist (10 ± 3)	Phyllite (7 ± 3)	Slate 7 ± 4
IGNEOUS	Plutonic	Light	Granite 32 ± 3	Diorite 25 ± 5		
				Monzonite (29 ± 3)		
		Dark	Gabbro 27 ± 3	Dolerite (16 ± 5)		
	Hypabyssal		Norite 20 ± 5			
			Porphyries (20 ± 5)		Diorite (15 ± 5)	Peridotite (25 ± 5)
		Volcanic		Rhyolite (25 ± 5)	Dacite (25 ± 3)	Basalt (25 ± 5)
		Pyroclastic	Agglomerate (19 ± 3)	Volcanic breccia (19 ± 5)	Tuff (13 ± 5)	

* These values are for intact rock specimens tested normal to bedding or foliation. The value of m_b will be significantly different if failure occurs along a weakness plane.

Disturbance to the foundation excavation caused by the rock removal methodology should be considered through the disturbance factor, D , in Eqs. 10.4.6.4-2 through 10.4.6.4-4.

The disturbance factor, D , ranges from 0.0 (undisturbed) to 1.0 (highly disturbed), and is an adjustment for the rock mass disturbance induced by the excavation method. Suggested values for various tunnel and slope excavations can be found in Hoek et al. (2002).

GEOLOGICAL STRENGTH INDEX (GSI) EVALUATION

Boring No.	Core Run Number	Depth (ft) from	Depth (ft) to	Elevation from	Elevation to	RQD (%)	Rock Type	ROCK STRUCTURE						ROCK SURFACE CONDITION					ROCK JOINTS (%)			ROCK JOINTS			GSI
								Intact or Massive	Blocky	Very Blocky	Block/ Disturbed/ Seamy	Disintegrated	Laminated/ Sheared	Very Good	Good	Fair	Poor	Very Poor	Open	Closed	Both	Open	Closed	Both	
BB-SOT-101	R1	23.8	28.8	-11.4	-16.4	90	GRANITE		X						X				10	90		0.1	0.9		70
BB-SOT-101	R2	28.8	33.8	-16.4	-21.4	97	GRANITE		X						X				30	70		0.3	0.7		70
BB-SOT-102	R1	25	30	-12.8	-17.8	42	GRANITE			X					X				70	30		0.7	0.3		55
BB-SOT-102	R2	30	35	-17.8	-22.8	75	GRANITE		X						X				20	80		0.2	0.8		65
BB-SOT-102	R3	35	40	-22.8	-27.8	22	GRANITE			X						X			40	60		0.4	0.6		40
BB-SOT-201	R1	25.5	30.5	-13.7	-18.7	65	GRANITE			X					X				80	20		0.8	0.2		70
BB-SOT-201	R2	30.5	35.5	-18.7	-23.7	82	GRANITE		X						X				50	50		0.5	0.5		80
BB-SOT-201	R3	35.5	40.5	-23.7	-28.7	73	GRANITE		X						X				70	30		0.7	0.3		80
BB-SOT-201	R4	40.5	45.5	-28.7	-33.7	83	GRANITE		X						X				70	30		0.7	0.3		80
BB-SOT-201	R5	45.5	50.5	-33.7	-38.7	87	GRANITE		X							X			90	10		0.9	0.1		45
BB-SOT-202	R1	25	29	-13.3	-17.3	67	GRANITE			X						X			90	10		0.9	0.1		50
BB-SOT-202	R2	29	33	-17.3	-21.3	67	GRANITE			X					X				90	10		0.9	0.1		50
BB-SOT-202	R3	33	38	-21.3	-26.3	88	GRANITE		X						X				90	10		0.9	0.1		85
BB-SOT-202	R4	38	43	-26.3	-31.3	77	GRANITE			X					X				60	40		0.6	0.4		65
BB-SOT-202	R5	43	48	-31.3	-36.3	92	GRANITE			X					X				70	30		0.7	0.3		65
BB-SOT-202	R6	48	50	-36.3	-38.3	58	GRANITE			X					X				90	10		0.9	0.1		55

SUMMARY OF LABORATORY TEST RESULTS

Boring No.	Core Run Number	GSI	Ave. Depth (ft)	Depth to Top of Rock (ft)	Depth from Top of Rock (ft)	Laboratory Test Results								
						Peak qu (psi)	Ei1 (psi)	Ei2 (psi)	Ei3 (psi)	Ei ave. (psi)	v1	v2	v3	vave
BB-SOT-101	R1	70	26.3	23.8	2.5	20,296	6,330,000	7,270,000	7,310,000	6,970,000	0.17	0.22	0.26	0.22
BB-SOT-102	R2	65	32.5	30	2.5	17,668	7,760,000	7,660,000	7,360,000	7,593,333	0.43	0.48	--	0.46
BB-SOT-201	R2	80	33.0	30.5	2.5	22,053	6,480,000	6,820,000	6,590,000	6,630,000	0.25	0.35	--	0.30
BB-SOT-201	R3	80	38.0	35.5	2.5	23,037	5,190,000	5,540,000	5,260,000	5,330,000	0.23	0.37	--	0.30
BB-SOT-202	R2	50	31	29	2	23,764	5,500,000	6,000,000	6,000,000	5,833,333	0.19	0.29	--	0.24
BB-SOT-202	R4	65	40.5	38	2.5	24,125	5,550,000	5,580,000	5,510,000	5,546,667	0.25	0.34	--	0.30

Overall Average for qu = 21,824

Overall Average for Ei = 6,317,222

Overall Average for v = 0.30

St. Dev. = 2,457

St. Dev. = 869,907

Mean-1SD = 19,367

Mean-1SD = 5,447,315

Mean+1SD = 24,281

Mean+1SD = 7,187,130

Client: Fuss & O'Neill

Computed by: EMH

Checked by: MMB

Sheet: 4 of 10

ROCK P-y CALCULATIONS

Boring No.	Core Run Number	Em/Ei Closed	Em/Ei Open	Mid-Depth (ft)	Depth to Top of Rock (ft)	Core Mid-Depth from Top of Rock (ft)	Tag for VLOOKUP	Proposed Selected qu (psi)	Proposed Selected Ei (psi)	Em Closed (psi)	Em Open (psi)	Rock Socket Diameter (ft)	Rock Socket Diameter (in)	Ic=Is (in^4)	f'c (psi)	Ec=Es (psi)	Rock v
BB-SOT-101	R1	0.90	0.43	26.3	23	3.3	BB-SOT-101 R1	20296	6970000	6,273,000	3,020,333	0.750	9	322.06233	5000	4,030,509	0.30
BB-SOT-101	R2	0.97	0.55	31.3	23	8.3	BB-SOT-101 R2	17668	5330000	5,170,100	2,931,500	0.750	9	322.06233	5000	4,030,509	0.30
BB-SOT-102	R1	0.12	0.09	27.5	24.5	3.0	BB-SOT-102 R1	17668	5330000	657,367	461,933	0.750	9	322.06233	5000	4,030,509	0.30
BB-SOT-102	R2	0.75	0.18	32.5	24.5	8.0	BB-SOT-102 R2	17668	7593333	5,695,000	1,392,111	0.750	9	322.06233	5000	4,030,509	0.30
BB-SOT-102	R3	0.06	0.05	37.5	24.5	13.0	BB-SOT-102 R3	17668	5330000	302,033	284,267	0.750	9	322.06233	5000	4,030,509	0.30
BB-SOT-201	R1	0.56	0.10	28.0	25.3	2.7	BB-SOT-201 R1	17668	5330000	2,998,125	533,000	0.750	9	322.06233	5000	4,030,509	0.30
BB-SOT-201	R2	0.82	0.30	33.0	25.3	7.7	BB-SOT-201 R2	22053	6630000	5,436,600	1,989,000	0.750	9	322.06233	5000	4,030,509	0.30
BB-SOT-201	R3	0.73	0.15	38.0	25.3	12.7	BB-SOT-201 R3	23037	5330000	3,890,900	799,500	0.750	9	322.06233	5000	4,030,509	0.30
BB-SOT-201	R4	0.83	0.32	43.0	25.3	17.7	BB-SOT-201 R4	17668	5330000	4,423,900	1,687,833	0.750	9	322.06233	5000	4,030,509	0.30
BB-SOT-201	R5	0.87	0.38	48.0	25.3	22.7	BB-SOT-201 R5	17668	5330000	4,637,100	2,043,167	0.750	9	322.06233	5000	4,030,509	0.30
BB-SOT-202	R1	0.62	0.10	27.0	24.5	2.5	BB-SOT-202 R1	17668	5330000	3,291,275	533,000	0.750	9	322.06233	5000	4,030,509	0.30
BB-SOT-202	R2	0.62	0.10	31.0	24.5	6.5	BB-SOT-202 R2	23764	5833333	3,602,083	583,333	0.750	9	322.06233	5000	4,030,509	0.30
BB-SOT-202	R3	0.88	0.40	35.5	24.5	11.0	BB-SOT-202 R3	17668	5330000	4,690,400	2,132,000	0.750	9	322.06233	5000	4,030,509	0.30
BB-SOT-202	R4	0.77	0.22	40.5	24.5	16.0	BB-SOT-202 R4	24125	5546667	4,270,933	1,201,778	0.750	9	322.06233	5000	4,030,509	0.30
BB-SOT-202	R5	0.92	0.47	45.5	24.5	21.0	BB-SOT-202 R5	17668	5330000	4,903,600	2,487,333	0.750	9	322.06233	5000	4,030,509	0.30
BB-SOT-202	R6	0.37	0.10	49.0	24.5	24.5	BB-SOT-202 R6	17668	5330000	1,972,100	533,000	0.750	9	322.06233	5000	4,030,509	0.30

ROCK P-y CALCULATIONS

Boring No.	Core Run Number	AASHTO 2012 (O'NEILL and REESE 1999)							AASHTO 2020 EQN. 1 (YANG 2006)					AASHTO 2020 EQN. 2 (HOEK et. al. 2002)			
		k Closed Joints (pci)	k Open Joints (pci)	k=E/B Closed Joints (pci)	k=E/B Open Joints (pci)	Weighted k closed (pci)	Weighted k open (pci)	Weighted k Ave (pci)	Em/Ei = f(GSI)	Em from GSI (psi)	k1 from Em=f(GSI) (pci)	k=Em/B from Em=f(GSI) (pci)	k average from Em=f(GSI) (pci)	Em (psi)	k1 from Em (pci)	k=Em/B (pci)	k average (pci)
BB-SOT-101	R1	664,537	301,056	697,000	335,593	612,692	31,832	644,524	0.252	1,754,619	167,155	194,958	181,056	4,586,504	473,361	509,612	491,486
BB-SOT-101	R2	538,945	291,475	574,456	325,722	389,690	92,580	482,270	0.252	1,341,767	124,998	149,085	137,042	4,586,504	473,361	509,612	491,486
BB-SOT-102	R1	57,705	39,375	73,041	51,326	19,612	31,745	51,357	0.126	672,161	59,113	74,685	66,899	1,934,113	185,756	214,901	200,329
BB-SOT-102	R2	598,465	130,087	632,778	154,679	492,497	28,477	520,974	0.200	1,518,150	142,893	168,683	155,788	3,439,393	346,558	382,155	364,356
BB-SOT-102	R3	24,849	23,270	33,559	31,585	17,523	10,971	28,494	0.063	336,721	27,955	37,413	32,684	815,609	72,894	90,623	81,759
BB-SOT-201	R1	298,658	45,977	333,125	59,222	63,178	42,080	105,258	0.252	1,341,767	124,998	149,085	137,042	4,586,504	473,361	509,612	491,486
BB-SOT-201	R2	569,105	191,473	604,067	221,000	293,293	103,118	396,411	0.399	2,646,057	260,857	294,006	277,432	8,156,086	883,133	906,232	894,682
BB-SOT-201	R3	396,103	71,336	432,322	88,833	124,264	56,059	180,323	0.399	2,127,222	205,929	236,358	221,144	8,156,086	883,133	906,232	894,682
BB-SOT-201	R4	455,208	160,273	491,544	187,537	142,013	121,734	263,746	0.399	2,127,222	205,929	236,358	221,144	8,156,086	883,133	906,232	894,682
BB-SOT-201	R5	479,021	197,129	515,233	227,019	49,713	190,866	240,579	0.080	423,973	35,881	47,108	41,495	1,087,632	99,566	120,848	110,207
BB-SOT-202	R1	330,419	45,977	365,697	59,222	34,806	47,340	82,146	0.100	533,833	46,055	59,315	52,685	1,450,380	135,996	161,153	148,575
BB-SOT-202	R2	364,352	50,699	400,231	64,815	38,229	51,981	90,210	0.100	584,245	50,785	64,916	57,850	1,450,380	135,996	161,153	148,575
BB-SOT-202	R3	484,989	206,430	521,156	236,889	50,307	199,494	249,801	0.503	2,678,433	264,317	297,604	280,960	10,876,316	1,206,265	1,208,480	1,207,372
BB-SOT-202	R4	438,181	110,934	474,548	133,531	182,546	73,339	255,885	0.200	1,108,956	101,682	123,217	112,450	3,439,393	346,558	382,155	364,356
BB-SOT-202	R5	508,915	243,949	544,844	276,370	158,064	182,112	340,176	0.200	1,065,637	97,386	118,404	107,895	3,439,393	346,558	382,155	364,356
BB-SOT-202	R6	189,712	45,977	219,122	59,222	20,442	47,340	67,781	0.126	672,161	59,113	74,685	66,899	1,934,113	185,756	214,901	200,329

ROCK P-y CALCULATIONS

Boring No.	Core Run Number	qu (MPa)	tmax (MPa)	tmax (psf)	γb (pcf)	mi	Degree of Disturbance	mb	s	a	qu (psf)	pL (psf)	pL+tmax (ksf)	pL+tmax (psi)	pL+tmax/qu used
BB-SOT-101	R1	140	2.37	49430	100	33	0	11.3	0.0357	0.5014	2922624	559625	609	4230	21%
BB-SOT-101	R2	122	2.21	46119	100	33	0	11.3	0.0357	0.5014	2544192	503383	550	3816	22%
BB-SOT-102	R1	122	2.21	46119	100	33	0	6.6	0.0067	0.5040	2544192	216573	263	1824	10%
BB-SOT-102	R2	122	2.21	46119	100	33	0	9.5	0.0205	0.5020	2544192	387450	434	3011	17%
BB-SOT-102	R3	122	2.21	46119	100	33	0	3.9	0.0013	0.5114	2544192	137214	183	1273	7%
BB-SOT-201	R1	122	2.21	46119	100	33	0	11.3	0.0357	0.5014	2544192	486638	533	3700	21%
BB-SOT-201	R2	152	2.47	51525	100	33	0	16.2	0.1084	0.5006	3175632	1063514	1115	7743	35%
BB-SOT-201	R3	159	2.52	52662	100	33	0	16.2	0.1084	0.5006	3317328	1122600	1175	8162	35%
BB-SOT-201	R4	122	2.21	46119	100	33	0	16.2	0.1084	0.5006	2544192	880553	927	6435	36%
BB-SOT-201	R5	122	2.21	46119	100	33	0	4.6	0.0022	0.5081	2544192	196847	243	1687	10%
BB-SOT-202	R1	122	2.21	46119	100	33	0	5.5	0.0039	0.5057	2544192	164024	210	1459	8%
BB-SOT-202	R2	164	2.56	53487	100	33	0	5.5	0.0039	0.5057	3422016	233403	287	1992	8%
BB-SOT-202	R3	122	2.21	46119	100	33	0	19.3	0.1889	0.5004	2544192	1130310	1176	8170	46%
BB-SOT-202	R4	166	2.58	53891	100	33	0	9.5	0.0205	0.5020	3474000	544954	599	4159	17%
BB-SOT-202	R5	122	2.21	46119	100	33	0	9.5	0.0205	0.5020	2544192	426885	473	3285	19%
BB-SOT-202	R6	122	2.21	46119	100	33	0	6.6	0.0067	0.5040	2544192	288671	335	2325	13%

Abutment	Average k (pci)	Selected k (pci)	Average pL+tmax (psi)	Selected pL+tmax (psi)
Abutment 1	211146	200000	3055	2000
Abutment 2	371342	300000	5110	4000

MICROPILE CALCULATIONS

Side Friction Calculations (Kulhawy et al., 2005)

Boring No.	Core Run Number	pa (ksf)	C	Proposed Selected qu (psi)	Proposed Selected qu (ksf)	f'c (psi)	f'c (ksf)	min(f'c,qu) (ksf)	qs (ksf)
BB-SOT-101	R1	2.12	1	20,296	2,923	5,000	720	720	39.1
BB-SOT-101	R2	2.12	1	17,668	2,544	5,000	720	720	39.1
BB-SOT-102	R1	2.12	1	17,668	2,544	5,000	720	720	39.1
BB-SOT-102	R2	2.12	1	17,668	2,544	5,000	720	720	39.1
BB-SOT-102	R3	2.12	1	17,668	2,544	5,000	720	720	39.1
BB-SOT-201	R1	2.12	1	17,668	2,544	5,000	720	720	39.1
BB-SOT-201	R2	2.12	1	22,053	3,176	5,000	720	720	39.1
BB-SOT-201	R3	2.12	1	23,037	3,317	5,000	720	720	39.1
BB-SOT-201	R4	2.12	1	17,668	2,544	5,000	720	720	39.1
BB-SOT-201	R5	2.12	1	17,668	2,544	5,000	720	720	39.1
BB-SOT-202	R1	2.12	1	17,668	2,544	5,000	720	720	39.1
BB-SOT-202	R2	2.12	1	23,764	3,422	5,000	720	720	39.1
BB-SOT-202	R3	2.12	1	17,668	2,544	5,000	720	720	39.1
BB-SOT-202	R4	2.12	1	24,125	3,474	5,000	720	720	39.1
BB-SOT-202	R5	2.12	1	17,668	2,544	5,000	720	720	39.1
BB-SOT-202	R6	2.12	1	17,668	2,544	5,000	720	720	39.1

Side Friction Calculations (O'Neill and Reese, 1999)

α E Closed	α E Open	qs Closed (ksf)	qs Open (ksf)	Weighted qs Closed (ksf)	Weighted qs Open (ksf)	Weighted qs (ksf)
0.95	0.75	48.6	38.4	43.7	3.8	47.6
0.99	0.82	47.0	39.1	32.9	11.7	44.7
0.56	0.53	26.7	25.3	8.0	17.7	25.7
0.88	0.60	41.8	28.6	33.4	5.7	39.1
0.46	0.46	22.0	22.0	13.2	8.8	22.0
0.79	0.55	37.6	26.3	7.5	21.0	28.5
0.91	0.67	48.5	35.7	24.3	17.9	42.1
0.87	0.58	47.2	31.6	14.1	22.1	36.3
0.92	0.68	43.7	32.5	13.1	22.7	35.8
0.94	0.72	44.6	34.4	4.5	30.9	35.4
0.81	0.55	38.8	26.3	3.9	23.6	27.5
0.81	0.55	45.0	30.4	4.5	27.4	31.9
0.94	0.73	44.9	34.8	4.5	31.4	35.9
0.89	0.62	49.4	34.6	19.7	20.8	40.5
0.96	0.77	45.8	36.8	13.7	25.7	39.5
0.70	0.55	33.4	26.3	3.3	23.6	27.0

Selected Unit Side Friction Resistance, qs

Selected qs (ksf)
39.1
39.1
25.7
39.1
22.0
28.5
39.1
36.3
35.8
35.4
27.5
31.9
35.9
39.1
27.0

Average= 33.8 ksf

Client: Fuss & O'Neill
Project: Oceanville Bridge pver Oceanville Thoroughfare
Subject: Micropile Geotechnical Resistance

CALCULATIONS
AASHTO LRFD STR Limit State Resistance Factors for Micropiles
Table 10.5.5.2.5-1—Resistance Factors for Geotechnical Resistance of Axially Loaded Micropiles

Limit State	Method/ Ground Condition	Resistance Factor
Compression Resistance of Single Micropile, ϕ_{stat}	Side Resistance (Bond Resistance): Presumptive Values	0.55 ⁽¹⁾
	Tip Resistance on Rock O'Neill and Reese (1999)	0.50
	Side Resistance and Tip Resistance Load Test	Values in Table 10.5.5.2.3-1, but no greater than 0.70
Block Failure, ϕ_{bl}	Clay	0.60
Uplift Resistance of Single Micropile, ϕ_{up}	Presumptive Values	0.55 ⁽¹⁾
	Tension Load Test	Values in Table 10.5.5.2.3-1, but no greater than 0.70
Group Uplift Resistance, ϕ_{ug}	Sand & Clay	0.50

⁽¹⁾ Apply to presumptive grout-to-ground bond values for preliminary design only in Article C10.9.3.5.2.

Table 10.5.5.2.4-1—Resistance Factors for Geotechnical Resistance of Drilled Shafts

	Method/Soil/Condition	Resistance Factor
Nominal Axial Compressive Resistance of Single-Drilled Shafts, ϕ_{stat}	Side resistance in clay α -method (Brown et al., 2010)	0.45
	Tip resistance in clay Total Stress (Brown et al., 2010)	0.40
	Side resistance in sand β -method (Brown et al., 2010)	0.55
	Tip resistance in sand Brown et al. (2010)	0.50
	Side resistance in cohesive IGMs	Brown et al. (2010) 0.60
	Tip resistance in cohesive IGMe	Brown et al. (2010) 0.55
	Side resistance in rock Kulhawy et al. (2005) Brown et al. (2010)	0.55
	Side resistance in rock Carter and Kulhawy (1988)	0.50
	Tip resistance in rock Canadian Geotechnical Society (1985) Pressuremeter Method (Canadian Geotechnical Society, 1985) Brown et al. (2010)	0.50
	Block Failure, ϕ_{bl}	Clay 0.55
Uplift Resistance of Single-Drilled Shafts, ϕ_{up}	Clay α -method (Brown et al., 2010)	0.35
	Sand β -method (Brown et al., 2010)	0.45
	Rock Kulhawy et al. (2005) Brown et al. (2010)	0.40
Group Uplift Resistance, ϕ_{ug}	Sand and clay	0.45
Horizontal Geotechnical Resistance of Single Shaft or Shaft Group	All materials	1.0
Static Load Test (compression), ϕ_{load}	All Materials	0.70
Static Load Test (uplift), ϕ_{upload}	All Materials	0.60

Bedrock Nominal Unit Resistance
33.8 ksf
Resistance Factor (Load Test)
0.55
Bedrock Factored Unit Resistance
18.6 ksf

Client: Fuss & O'Neill

Date: 4-Oct-2023

Project: Oceanville Bridge pver Oceanville Thoroughfare

Computed by: EMH

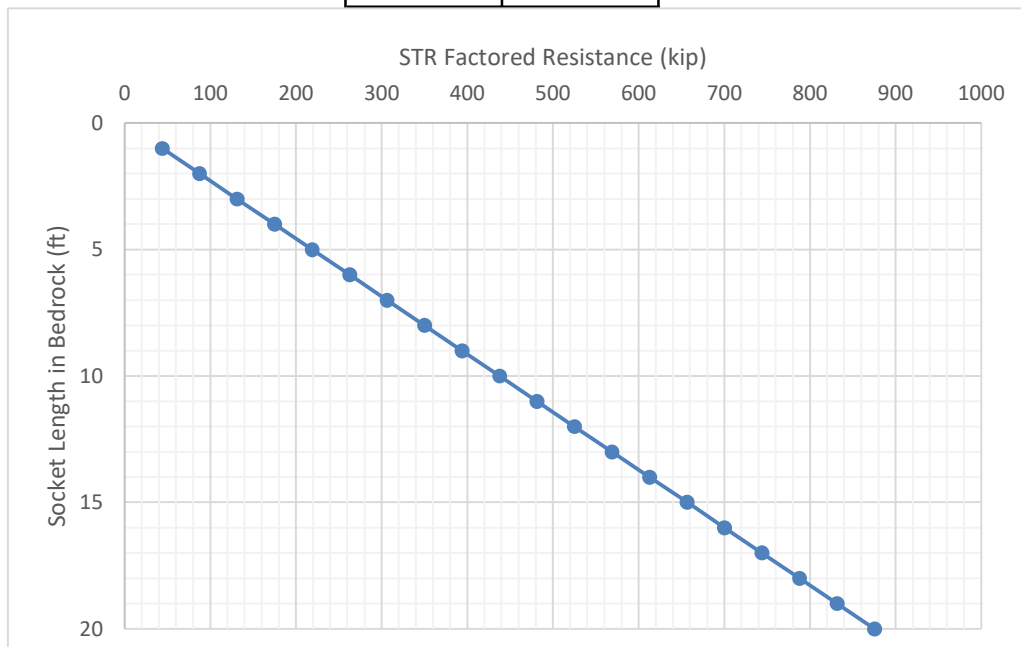
Subject: Micropile Geotechnical Resistance

Checked by: MMB

CALCULATIONS

Rock Socket Diameter (in) = 9
 Rock Socket Perimeter (ft) = 2.4
 Rock Socket Side Area (ft²/ft) = 2.4
 Bedrock STR Factored Unit Side Resistance (ksf) = 18.6
 Bedrock STR Factored Resistance per foot (kip/ft) = 43.8

Rock Socket Length in Bedrock (ft)	Bedrock Factored Geotechnical Resistance (kip)
1	44
2	88
3	131
4	175
5	219
6	263
7	306
8	350
9	394
10	438
11	481
12	525
13	569
14	613
15	656
16	700
17	744
18	788
19	832
20	875



Micropile Elastic Compression

Client: Fuss & O'Neill

Date: 23-Aug-2023

Project: Oceanville Road Over Oceanville Thoroughfare

Computed by: EMH

Subject: Elastic Deformation of Abutment Micropiles

Checked by: MMB

PROBLEM STATEMENT & OBJECTIVE

Determine the elastic compression for micropiles with various rock lengths.

AVAILABLE INFORMATION

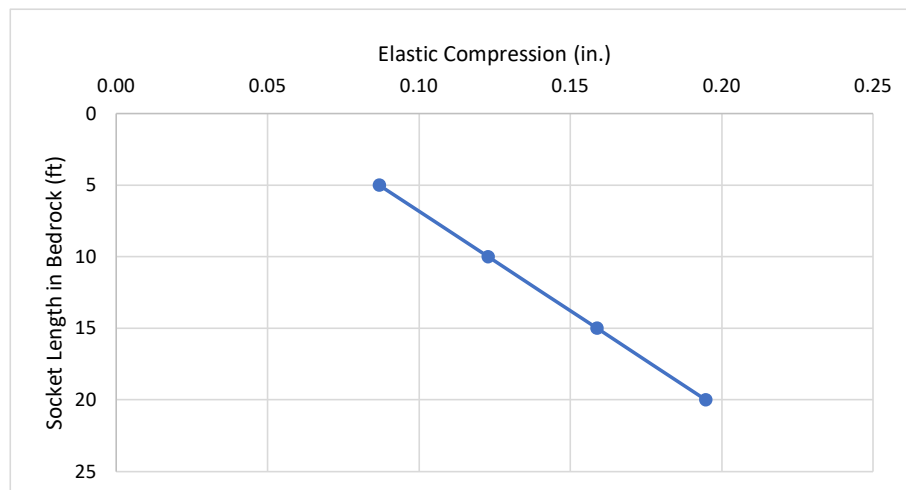
1. Borings logs drilled in March 2016 and June 2022 by New England Boring Contractors.

ASSUMPTIONS

1. Micropile diameter = 9.625 in (OD) with 0.545 in casing. Socket diameter below casing = 9.0 in.
2. Corrosion assumed to reduce casing thickness by 1/16 inch.
3. Micropiles will have a single #11 rebar in the center.
4. The elastic modulus for concrete and steel rebar is 3,600 ksi and 29,000 ksi, respectively.
5. Maximum compression loads on the micropiles are 160 kips, based on email correspondence from Fuss & O'Neill on 25 July 2023.
6. Bottom of proposed abutments is at El. 3.0. Deepest encountered bedrock is at approximately El. -14 (BB-SOT-201).

Pile Section	AE Total	Length	AE/L	Effective Vertical Stiffness, k_v	Max Compression Load	Total Elastic Compression
	(kip)	(ft)	(kip/in.)	(kip/in)	(kip)	(in)
Cased Zone	640360	17	3139			
Rock Socket	267049	5	4451	1841	160	0.1
		10	2225	1302	160	0.1
		15	1484	1007	160	0.2
		20	1113	822	160	0.2

Note: Total Elastic Compression values include compression of the cased zone.


Notes:

1. AE total is the sum of the steel AE and grout AE for each section.
2. The effective vertical stiffness is calculated as $k_v = 1 / (1/k_{v1} + 1/k_{v2})$ where k_{v1} and k_{v2} are the AE/L of the sections making up the micropile.

LPile Parameters

CALCULATIONS

File No.:	42835-002
Sheet:	1 of 2
Date:	2-Aug-2023
Computed by:	EMH
Checked by:	MMB

Client: Fuss & O'Neill

Project: Oceanville Road Bridge, Bridge No. 3696, Stonington, ME

Subject: LPile Soil Parameters - Mean High Water

PROBLEM STATEMENT & OBJECTIVE

Estimate soil parameters for use in LPile evaluations at both abutments.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 2020.
2. LPile Software Manual by Ensoft.

AVAILABLE INFORMATION

1. Boring logs BB-SOT-101, BB-SOT-102, BB-SOT-201, and BB-SOT-202
2. Info from Fuss & O'Neill indicating proposed micropile size is 9.625-in. dia. O.D.

ASSUMPTIONS

1. NAVD 88 elevation datum.
2. Mean High Water (MHW) El. 4.6, Mean Low Water (MLW) El. -5.3, per Fuss & O'Neill PIC Plans.
3. If piles are closer than 5 times the pile dimension, consider lateral group effects (see AASHTO LRFD for factors).
4. Bottom of pile cap at El. 3. Micropile rock socket diameter is 9-in. dia.

LPILE SOIL PARAMETERS

Abutment No. 1 (based on boring BB-SOT-202)

Soil Layer	Soil Model	Top Elevation (ft)	Effective Unit Weight, γ' (pcf)	Undrained Shear Strength, S_u (psf)	Internal Angle of Friction, ϕ' (degrees)	Soil Modulus, k (pci)	Strain Factor, ϵ_{50}
Fill	Sand (Reese)	3.0	56.0	N/A	32	60	N/A
Harbor Bottom Cohesive	Soft Clay (Matlock)	2.0	46.0	400	N/A	N/A	0.020
Harbor Bottom Cohesionless	Sand (Reese)	-4.1	46.0	N/A	32	60	N/A
Glacial Till	Sand (Reese)	-9.3	66.0	N/A	36	125	N/A
Bedrock	See Below	-12.8	N/A	N/A	N/A	N/A	N/A

Rock Layer	Rock Model	Top Elevation (ft)	Bottom Elevation (ft)	Effective Unit Weight (pcf)	γ (in)	P (lb/in)
Bedrock	User Input P-y Curves	-12.8	bottom of micropile	100	0	0
					0.01	18,000
					10	18,000

Abutment No. 2 (based on boring BB-SOT-201)

Soil Layer	Soil Model	Top Elevation (ft)	Effective Unit Weight, γ' (pcf)	Undrained Shear Strength, S_u (psf)	Internal Angle of Friction, ϕ' (degrees)	Soil Modulus, k (pci)	Strain Factor, ϵ_{50}
Fill	Sand (Reese)	3.0	56.0	N/A	32	60	N/A
Marine Deposit	Soft Clay (Matlock)	-1.7	51.0	1000	N/A	N/A	0.010
Glacial Till	Sand (Reese)	-6.6	66.0	N/A	36	125	N/A
Bedrock	See Below	-13.5	N/A	N/A	N/A	N/A	N/A

Rock Layer	Rock Model	Top Elevation (ft)	Bottom Elevation (ft)	Effective Unit Weight (pcf)	γ (in)	P (lb/in)
Bedrock	User Input P-y Curves	-13.5	bottom of micropile	100	0	0
					0.01	36,000
					10	36,000

CALCULATIONS

File No.:	42835-002
Sheet:	2 of 2
Date:	2-Aug-2023
Computed by:	EMH
Checked by:	MMB

Client: Fuss & O'Neill
 Project: Oceanville Road Bridge, Bridge No. 3696, Stonington, ME
 Subject: LPile Soil Parameters - Mean Low Water

PROBLEM STATEMENT & OBJECTIVE

Estimate soil parameters for use in LPile evaluations at both abutments.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 2020.
2. LPile Software Manual by Ensoft.

AVAILABLE INFORMATION

1. Boring logs BB-SOT-101, BB-SOT-102, BB-SOT-201, and BB-SOT-202
2. Info from Fuss & O'Neill indicating proposed micropile size is 9.625-in. dia. O.D.

ASSUMPTIONS

1. NAVD 88 elevation datum.
2. Mean High Water (MHW) El. 4.6, Mean Low Water (MLW) El. -5.3, per Fuss & O'Neill PIC Plans.
3. If piles are closer than 5 times the pile dimension, consider lateral group effects (see AASHTO LRFD for factors).
4. Bottom of pile cap at El. 3. Micropile rock socket diameter is 9-in. dia.

LPILE SOIL PARAMETERS

Abutment No. 1 (based on boring BB-SOT-202)

Soil Layer	Soil Model	Top Elevation (ft)	Effective Unit Weight, γ' (pcf)	Undrained Shear Strength, S_u (psf)	Internal Angle of Friction, ϕ' (degrees)	Soil Modulus, k (pci)	Strain Factor, ϵ_{50}
Fill	Sand (Reese)	3.0	120.0	N/A	32	90	N/A
Harbor Bottom Cohesive	Soft Clay (Matlock)	2.0	110.0	400	N/A	N/A	0.020
Harbor Bottom Cohesionless	Sand (Reese)	-4.1	46.0	N/A	32	60	N/A
Glacial Till	Sand (Reese)	-9.3	66.0	N/A	36	125	N/A
Bedrock	See Below	-12.8	N/A	N/A	N/A	N/A	N/A

Rock Layer	Rock Model	Top Elevation (ft)	Bottom Elevation (ft)	Effective Unit Weight (pcf)	γ (in)	P (lb/in)
Bedrock	User Input P-y Curves	-12.8	bottom of micropile	100	0	0
					0.01	18,000
					10	18,000

Abutment No. 2 (based on boring BB-SOT-201)

Soil Layer	Soil Model	Top Elevation (ft)	Effective Unit Weight, γ' (pcf)	Undrained Shear Strength, S_u (psf)	Internal Angle of Friction, ϕ' (degrees)	Soil Modulus, k (pci)	Strain Factor, ϵ_{50}
Fill	Sand (Reese)	3.0	120.0	N/A	32	90	N/A
Marine Deposit	Soft Clay (Matlock)	-1.7	115.0	1000	N/A	N/A	0.010
Glacial Till	Sand (Reese)	-6.6	66.0	N/A	36	125	N/A
Bedrock	See Below	-13.5	N/A	N/A	N/A	N/A	N/A

Rock Layer	Rock Model	Top Elevation (ft)	Bottom Elevation (ft)	Effective Unit Weight (pcf)	γ (in)	P (lb/in)
Bedrock	User Input P-y Curves	-13.5	bottom of micropile	100	0	0
					0.01	36,000
					10	36,000

Lateral Earth Pressure

Client: Fuss & O'Neill

Date: 4-Oct-2023

Project: Oceanville Bridge over Oceanville Thoroughfare, Bridge No. 3696 Stonington, Maine

Computed by: EMH

Subject: Lateral Earth Pressure Coefficient Calculation

Checked by: MMB

PROBLEM STATEMENT & OBJECTIVE

Determine the passive lateral earth pressure coefficient for the integral abutments in accordance with AASHTO LRFD.

REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 9th Edition, 2020.

CALCULATIONS

ϕ'_f (deg)	θ (deg)	δ (deg)	$-\delta/\phi'_f$	k_p	R	$R \cdot k_p$
32	90	24	-0.75	7.7	0.86	6.6

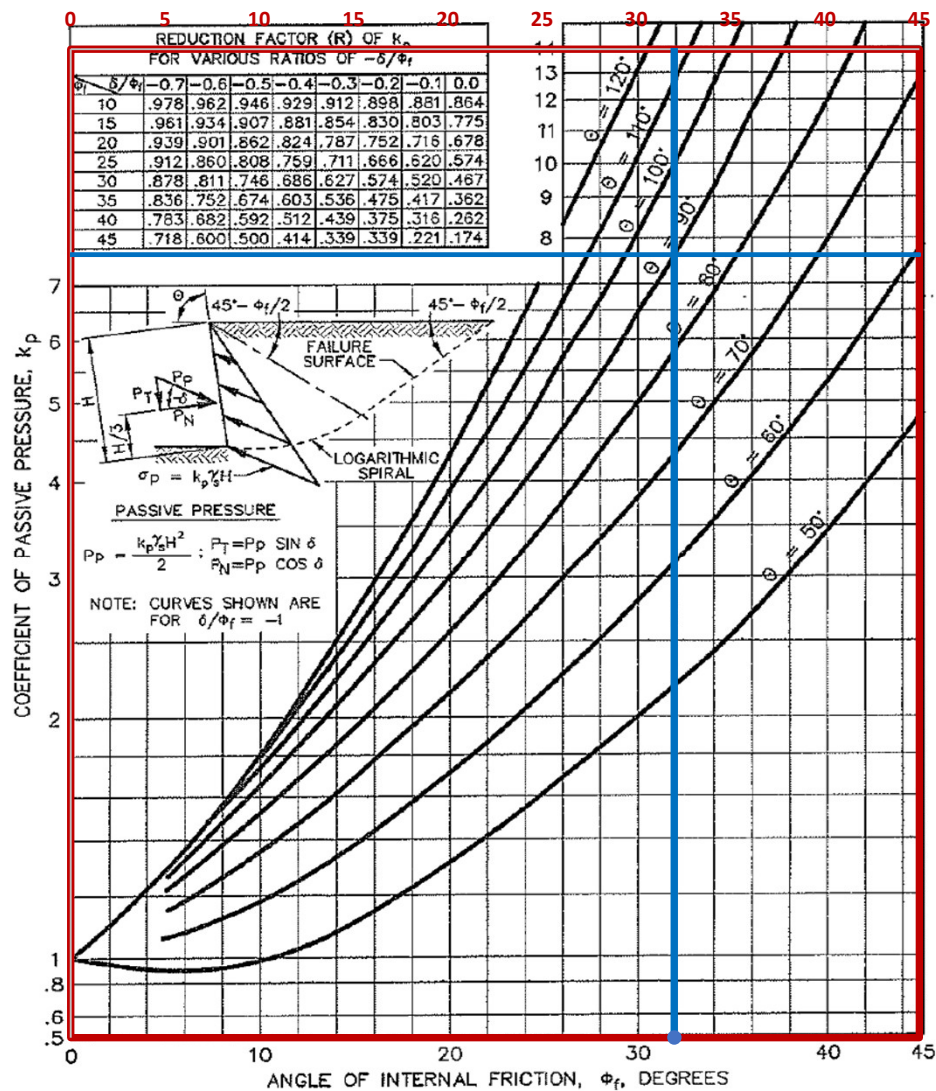
Note: $R \cdot k_p$ should only be applied to effective earth pressures (i.e., do not apply to hydrostatic pressures).

Figure 3.11.5.4-1—Computational Procedures for Passive Earth Pressures for Vertical and Sloping Walls with Horizontal Backfill (U.S. Department of the Navy, 1982a)

Frost Evaluation

Client: Fuss & O'Neill

Date: 4-Oct-2023

Project: Oceanville Road Bridge over Oceanville Thoroughfare No. 3696 - WIN 22630

Computed by: EMH

Subject: Frost Penetration Depth Evaluation

Checked by: MMB

OBJECTIVE:

Evaluate depth of frost penetration based on soil and groundwater conditions, as well as geographic site location.

REFERENCES:

1. MaineDOT Bridge Design Guide, August 2003, with June 2018 updates
2. MaineDOT test borings BB-SOT-101, BB-SOT-102, BB-SOT-201, and BB-SOT-202.
3. MaineDOT drawing set prepared by Fuss & O'Neill, dated November, 2022.

EVALUATION:

1. Gather relevant information from test borings performed near proposed bridge abutment locations:

STRUCTURE	TEST BORING NO. ¹	GROUND SURFACE	GROUND WATER	SAMPLE No. AND ELEVATION	AASHTO/USCS	FINES CONTENT
ABUTMENT NO. 1	BB-SOT-102	El. 12.2	Not Observed	1D El. 11.2 - 9.2	A-1-b(0)/SM	23.3
				3D El. -2.8 - -4.8	A-1-a(1)/SW-SM	9.4
ABUTMENT NO. 2	BB-SOT-101	El. 12.4	Not Observed	2D El. 9.4 - 7.4	A-1-b(0)/SM	13
				5D El. 2.4 - 0.4	A-1-a(0)/SM	13.1

¹ Borings BB-SOT-201 and BB-SOT-202 were excluded because samples were not tested in the laboratory.

2. Water levels at the site are as follows:

Q1.1 (MLW) El. -5.3

Q1.1 (MHW) El. 4.6

Q100 El. 9.3

3. Proposed bottom of abutment elevations are as follows:

Abutment No. 1 El. 3.0

Abutment No. 2 El. 3.0

4. Per MaineDOT Bridge Design Guide Figure 5-1, the design freezing index for the site is approximately 1050°F - days.

5. Estimate range in frost penetration using MaineDOT Bridge Design Guide Table 5-1, the design freezing index above and an assumed range in water content since site-specific laboratory test data does not exist

From Table 5-1, frost penetration depths vary between approximately 4.1 ft (w=30%), 4.7 ft (20%) and 5.7 ft (w=10%) for coarse-grained soils.

Recommend 4.7 ft (w=20%)

Client: Fuss & O'Neill

Date: 4-Oct-2023

Project: Oceanville Road Bridge over Oceanville Thoroughfare No. 3696 - WIN 22630

Computed by: EMH

Subject: Frost Penetration Depth Evaluation

Checked by: MMB

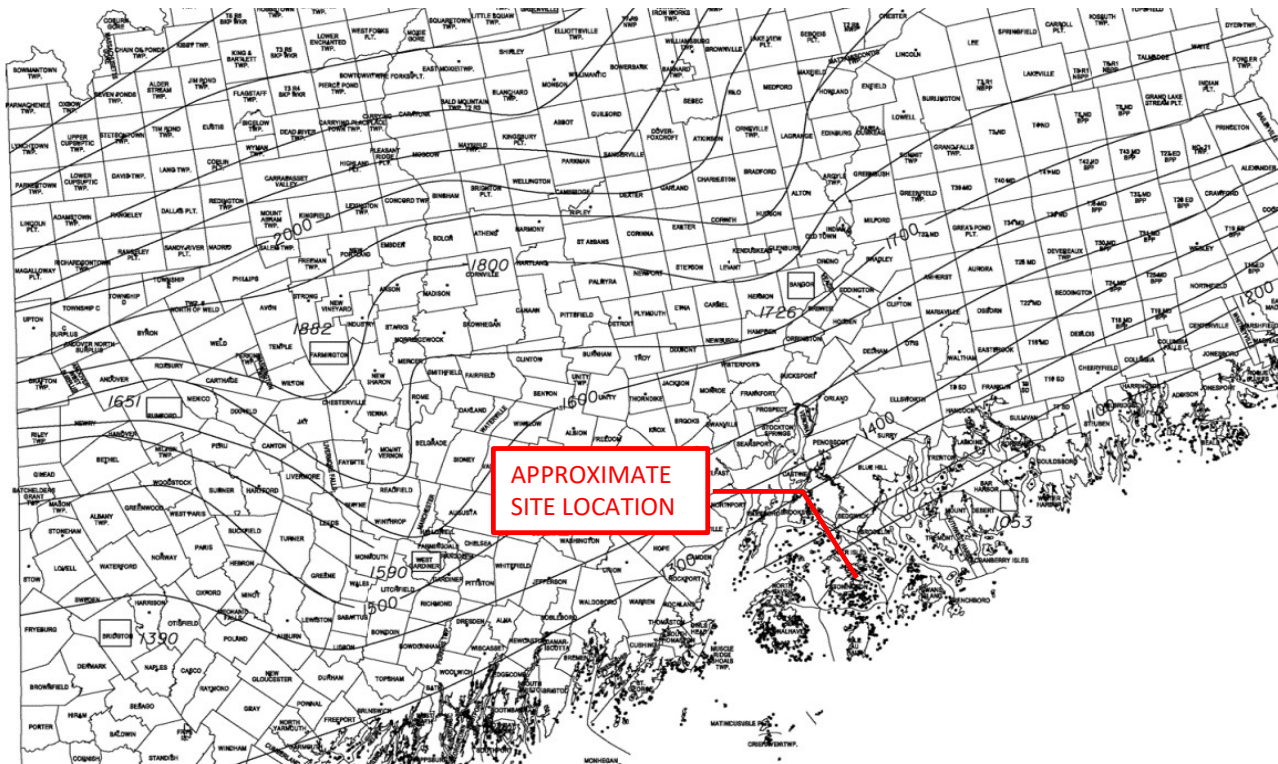


Table 5-1 Depth of Frost Penetration

Design Freezing Index	Frost Penetration (in)					
	Coarse Grained			Fine Grained		
	w=10%	w=20%	w=30%	w=10%	w=20%	w=30%
1000	66.3	55.0	47.5	47.1	40.7	36.9
1100	69.8	57.8	49.8	49.6	42.7	38.7
1200	73.1	60.4	52.0	51.9	44.7	40.5
1300	76.3	63.0	54.3	54.2	46.6	42.2
1400	79.2	65.5	56.4	56.3	48.5	43.9
1500	82.1	67.9	58.4	58.3	50.2	45.4
1600	84.8	70.2	60.3	60.2	51.9	46.9
1700	87.5	72.4	62.2	62.2	53.5	48.4
1800	90.1	74.5	64.0	64.0	55.1	49.8
1900	92.6	76.6	65.7	65.8	56.7	51.1
2000	95.1	78.7	67.5	67.6	58.2	52.5
2100	97.6	80.7	69.2	69.3	59.7	53.8
2200	100.0	82.6	70.8	71.0	61.1	55.1
2300	102.3	84.5	72.4	72.7	62.5	56.4
2400	104.6	86.4	74.0	74.3	63.9	57.6
2500	106.9	88.2	75.6	75.9	65.2	58.8
2600	109.1	89.9	77.1	77.5	66.5	60.0