REPORT

GEOTECHNICAL DESIGN REPORT

Marsh Bridge Replacement
Tremont and Southwest Harbor, Maine
Maine DOT WIN 21702.00

Submitted To: Vanasse Hangen Brustlin, Inc.
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December 11, 2017

Project No.: 1668741

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December 11, 2017

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Mr. Robert Blunt, P.E.
Project Manager
Vanasse Hangen Brustlin, Inc.
500 Southborough Drive, Suite 105B
South Portland, ME 04106

RE: GEOTECHNICAL DESIGN REPORT
MARSH BRIDGE REPLACEMENT
TREMONT AND SOUTHWEST HARBOR, MAINE
MAINE DOT WIN 21702.00

Dear Mr. Blunt:

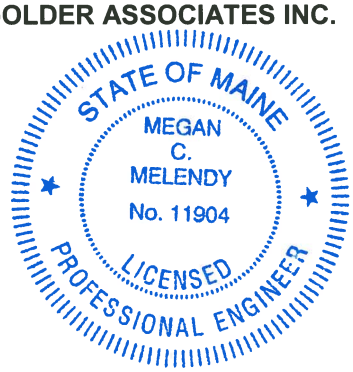
Golder Associates Inc. (Golder) is pleased to submit this Geotechnical Design Report to Vanasse Hangen Brustlin, Inc. (VHB) for the proposed Marsh Bridge Replacement on the town-line of Tremont and Southwest Harbor, Maine. Field explorations were completed in two phases for this project: an initial program was completed in May 2017 (Phase I) and a supplemental program was performed in September 2017 (Phase II). This report presents the soils information and field testing data obtained during both the Phase I & II subsurface investigations, the results of geotechnical laboratory tests, and final geotechnical recommendations for the proposed bridge replacement.

Our Phase I work was conducted in accordance with the scope, schedule and budget described in our proposals dated January 5, 2017 and revised on March 2, 2017. Our Phase II work was completed under a separate proposal dated September 7, 2017. The terms and conditions governing the work are stated in our General Consulting Agreement (GCA) with MaineDOT and the provisions of our Subconsultant Terms and Conditions Agreement with VHB executed March 15, 2012.

It has been a pleasure working with VHB during this interesting project. Please contact us if you have any questions concerning our report or require additional information.

Sincerely,

GOLDER ASSOCIATES INC.



Megan C. Melendy, P.E.
Senior Engineer

Mark S. Peterson, P.E.
Principal

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1.0 INTRODUCTION

This report summarizes the results of Golder Associates Inc.'s (Golder's) Phase I & II geotechnical investigations, evaluations, and recommendations for the replacement of Marsh Bridge carrying Maine State Route 102 over Marshall Brook at the Southwest Harbor and Tremont town line (see Figure 1). The purpose of this report is to present soils and bedrock information obtained from subsurface investigations and laboratory tests; discuss geotechnical considerations for the bridge foundations and embankment widening; and present recommended geotechnical parameters for design and construction of the bridge rehabilitation. Our work was completed in accordance with our General Consulting Agreement (GCA) with MaineDOT and the provisions of our Subconsultant Terms and Conditions Agreement with VHB executed March 15, 2012.

The Marsh Bridge carries State Route 102 over Marshall Brook and is less than a mile from Clark Bridge at the Southwest Harbor and Tremont town line. Golder has provided similar geotechnical engineering services to VHB and MaineDOT at the Clark Bridge since 2012. Marsh Bridge was constructed in 1931 and is a 20-foot clear span cast-in-place concrete superstructure that rests on concrete capped granite blocked abutments and wing walls, with concrete footings supported by piles. The roadway is 22 feet (ft) wide and is flanked by steel guardrail for the approaches which are both approximately 100 ft long.

According to the 1931 design drawings[1], the existing abutments are supported on 48 piles with a capacity of 12 tons each. The piles are assumed to be timber piles driven to bedrock. The approach fill is indicated to be retained by logs and boulders and the drawings report frequent settlements observed over the east approach. In addition, Golder reviewed pile installation field notes from the 1931 construction field book[2] that indicated pile installation lengths for the west and east abutments and wingwalls between approximately 6 and 19.5 feet.

We understand that MaineDOT plans to replace the existing bridge deck and reuse the existing substructure (foundation, abutments, and retaining walls) that are supported on timber piles. In addition, the design cross section drawings provided by VHB[3] include widening the approach fills and the addition of four embankment retaining walls located adjacent to the existing bridge abutment retaining walls needed to maintain the widened slope fills within the right of way. The majority of the embankment fill occurs within approximately 40 ft of each abutment and is thickest at the widened top of slope (i.e., approximately 2 to 3.5 ft over the western approach and approximately 1 to 4 ft over the eastern approach.

[1] *Maine Highway Commission Bridge Division Marsh Bridge over Marshall Brook Between the Towns of Southwest Harbor and Tremont*, February 5, 1931.
[2] *MaineDOT, Southwest Harbor & Tremont Marsh Bridge over Marshall Brook Construction field book, R.P. Thaxter, C.L. English, September 22, 1931 to June 8, 1932.*
[3] *Marsh Bridge Bass Harbor Over Marshall Brook, Tremont, Hancock County, Bridge Plans, STP-2170(200)*, VHB, August 17, 2017.

2.0 GEOLOGIC SETTING

2.1 Regional Surficial Geology

The proposed bridge replacement site is located in southern-central Maine within the Seaboard Lowlands Physiographic Province[4]. Regional surficial geologic mapping indicates the regional surficial soils are Pleistocene-aged emerged marine sediments deposited in an environment when relative sea level was higher than present[5]. A layer of till generally overlies bedrock and consists of a heterogeneous, non-sorted to poorly sorted mixture of clay, silt, sand, pebbles, cobbles and boulders. The till generally overlies bedrock, and is in turn overlain by Pleistocene-aged ice-contact gravel; marine shoreline deposits; glaciomarine silts, clays and fine sands of the Presumpscot Formation; younger marine shoreline deposits; salt marsh deposits; freshwater wetland muck, peat, silt and sand; and alluvial deposits. The Presumpscot Formation glaciomarine deposits consist of fine-grained, marine mud (silt and clay with sandy lenses), commonly containing gravel dropstones and, more rarely, marine shell fossils. These sediments were deposited in deeper, quieter water during the marine submergence of the coast. The salt marsh deposits consist of grass, reed, and sedge wetland, inundated at high tide, underlain by fine grained sediment about 1 to 6 ft thick.[6]

2.2 Regional Bedrock Geology

Regional bedrock geologic mapping indicates the bedrock consists of light-colored volcanic tuffs and metamorphosed siltstones and slates of the Silurian aged Cranberry Island Volcanic Series[7,8]. The bedrock beneath the site is mapped as the upper member of the series, consisting of blue-gray to light brown felsites and gray to brown dacite flows. The felsites have also been mapped as a uniform, pink, very fine- to fine grained granite.[9] Bedding in the volcanic and sedimentary rocks strikes east-northeast, and dips moderately to the south-southeast.

[4] Fenneman, N.M., and Johnson, D.W., 1946. Physiographic Divisions of the Conterminous U.S., U.S. Geological Survey, 1 sheet, scale 1:7,000,000.
[5] Lowel, T.V. and Borns, H.W. Jr., 1988. Surficial Geology of Mount Desert Island – A Visitor's Guide to the Geology of Acadia National Park, Maine Geological Survey, Bulletin 38, map scale 1:50,000.
[6] Braun, D.D., Lowell, T.V., and Weddle, T.K., 2016. Surficial Geology of the Southwest Harbor Quadrangle, Maine. Maine Geological Survey, Open-File Map No. 16-21, scale 1:24,000.
[7] Gilman, R.A., Chapman, C.A., Lowel, T.V. and Borns, H.W.Jr., 1988. The Geology of Mount Desert Island – A Visitor's Guide to the Geology of Acadia National Park. Maine Geological Survey, Bulletin 38, 50 p.
[8] Graham, J., 2010. Acadia National Park: Geologic Resources Inventory Report. Natural Resource Report NPS/NRPC/GRD/NRR-2010/232, National Park Service, Ft. Collins, Colorado, 63 p.
[9] Lubick, N., 1994. The Silurian (?) Cranberry Island Volcanic Series on Mt. Desert Island, Maine. In Woodward, H.H., ed., The Keck Geology Consortium, 7th Keck Symposium Volume, Beloit College, p. 25-28.

3.0 SUBSURFACE EXPLORATION PROGRAM

3.1 Phase I Geotechnical Exploration

On May 8 to 11, 2017 New England Boring Contractors (NEB) of Hermon, Maine drilled four borings (BB-MAR-101, BB-MAR-102, BB-MAR-103A/B and BB-MAR-104). Boring locations were surveyed by MaineDOT following completion of the drilling program. Locations shown on Figure 2 and elevations included on the boring logs in Appendix A represent as-drilled locations. Test borings were drilled along the road located on either side of the existing bridge to depths of about 8 to 60 ft below ground surface (bgs). Boring BB-MAR-103A was advanced to 8.3 ft bgs but refusal conditions were encountered on suspected timber cribbing from the original bridge foundation. Boring BB-MAR-103B was offset roughly 3.5 ft from BB-MAR-103A and was washed ahead to roughly 8.5 ft bgs before sampling was resumed.

Golder classified soils in the field and collected samples for visual identification and laboratory testing. Standard Penetration Test (SPT) sampling was conducted through the overburden. Standard 1-3/8-inch ID split spoons were driven 24 inches by a 140 pound hammer dropped 30 inches using a rope and cathead. Golder recorded the number of hammer blows required to advance the sampler at 6-inch increments, lithologic descriptions, and sample recovery lengths. Drill behavior during casing and drill rod advancement and cuttings observed during drilling were also recorded. Golder collected soil samples from each SPT split spoon for visual identification and laboratory testing. A hammer efficiency factor of 0.60 for the rope and cathead drilling method was used in determining the N_{60} corrections[10] shown on the logs. Soils were field classified in general accordance with ASTM D 2488 and the MaineDOT Key to Soil and Rock Descriptions and Terms. In addition, Golder performed in-situ field vane shear tests within the clay layers in accordance with MaineDOT's required equipment and testing procedures. Shelby Tube sampling was attempted throughout the drilling program, and one sample was successfully retrieved during drilling. After reaching bedrock refusal, NEB obtained approximately 9 to 10 feet of NQ sized bedrock core from each of the borings.

Details of the sampling methods used, field data obtained, and soil, bedrock and groundwater conditions encountered are presented on the attached boring logs included in Appendix A. Photographs of the rock core and a summary of rock core descriptions and rock quality assessments are included in Appendix B. Table 2 summarizes rock core descriptions and quality assessments.

3.2 Phase II Geotechnical Exploration

A Phase II exploration program was performed to obtain additional stratification definition and in-situ shear strength and compressibility information for the soft soil deposits at the site. From September 25 to 29,

[10] Standard Penetration Test N-values (blows per foot) corrected for hammer energy to determine an equivalent N value for a hammer with 60% efficiency are designated as N_{60} values. N values discussed in the report generally refer to corrected N_{60} values.

2017 New England Boring Contractors (NEB) of Hermon, Maine drilled seven borings (BB-MAR-201A, BB-MAR-201B, BB-MAR-202, BB-MAR-203A, BB-MAR-203B, BB-MAR-204 and BB-MAR-205), at the locations shown on attached Figure 2. Test borings BB-MAR-201A/B and BB-MAR-202 were drilled along the road located on either side of the existing bridge. Borings BB-MAR-201A, BB-MAR-201B and BB-MAR-202 were drilled to depths of 18, 15 and 43 ft bgs respectively using a Mobile Truck Mounted drill rig. Borings BB-MAR-203A, BB-MAR-203B, BB-MAR-204 and BB-MAR-205 were drilled at the toe of the embankment slopes along the eastern approach using an M-1 track-mounted drill rig to depths ranging from 16 to 38 ft bgs. All toe borings, excluding BB-MAR-205, were accessed by mobilizing around the guardrail and using mats placed on top of the marsh grass. Boring BB-MAR-205 was accessed by removing a section of the guardrail and mobilizing over the embankment. The guardrail was replaced upon completion of the boring.

Golder classified soils in the field and collected samples for visual identification and laboratory testing. Standard Penetration Test (SPT) sampling was conducted through the overburden. Standard 1-3/8-inch ID split spoons were driven 24 inches by a 140 pound hammer dropped 30 inches using a rope and cathead. Golder recorded the number of hammer blows required to advance the sampler at 6-inch increments, lithologic descriptions, and sample recovery lengths. Drill behavior during casing and drill rod advancement and cuttings observed during drilling were also recorded. Golder collected soil samples from each SPT split spoon for visual identification and laboratory testing. A hammer efficiency factor of 0.60 for the rope and cathead drilling method (M-1 drill rig) and a hammer efficiency factor of 0.885[11] was used for the automatic hammer (MobileTruck mounted drill rig) to determine the N_{60} corrections[12] shown on the logs. Soils were field classified in general accordance with ASTM D 2488 and the MaineDOT Key to Soil and Rock Descriptions and Terms. In addition, Golder performed in-situ field vane shear tests within the organic silt and silty clay layers. In total, six Shelby tubes were taken in the organic silt and silty clay layers.

Details of the sampling methods used, field data obtained, and soil and groundwater conditions encountered are presented on the attached boring logs included in Appendix A.

3.3 Geophysical Investigation

On May 4th, 2017 Hager-Richer Geoscience, Inc. (HR) of Salem, New Hampshire performed a geophysical survey at the site. The geophysical survey consisted of a ground penetrating radar (GPR) survey across the area of interest (i.e. location of historic timber cribbing) and two seismic refraction profiles, one on each

11 GZA Geo Environmental (2017). SPT Energy Testing report titled “Truck 83 – New Hampshire License Plate No.: 4368,” prepared for New England Boring Contractors.

[12] Standard Penetration Test N-values (blows per foot) corrected for hammer energy to determine an equivalent N value for a hammer with 60% efficiency are designated as N_{60} values. N values discussed in the report generally refer to corrected N_{60} values.

of the approaches. The GPR area and seismic refraction line locations are shown on Figure 2. The GPR survey was conducted in an area measuring approximately 30-feet by 550-feet with traverses spaced 2 feet apart oriented parallel to the travel lanes, and along traverses spaced 50 feet apart oriented perpendicular to the travel lanes. The seismic refraction profile consisted of two 235-foot long seismic refraction survey lines. The interpretation of the GPR data and the results of the seismic refraction are shown on the attached Figures 2 and 3 and the complete HR report is included in Appendix D.

4.0 LABORATORY TESTING PROGRAM

Geotechnical laboratory tests were performed on representative soil and rock samples collected during the subsurface investigation to assist in soil classification and to assess soil and rock engineering properties. Samples were collected at all borings locations during the 2017 Golder Investigations.

Testing on the selected soil samples was conducted by GeoTesting Express in Acton, Massachusetts. Laboratory work was performed in accordance with applicable AASHTO and American Society for Testing Materials (ASTM) testing procedures. Testing performed for the investigation is summarized below.

Soil Laboratory Test	Testing Procedure	Number of Tests Completed	
		Phase I	Phase II
Grain Size Analysis, sieve only	AASHTO T88, ASTM D422	3	'- -
Grain Size Analysis including Hydrometer	AASHTO T88, ASTM D422	6	'- -
Natural Moisture Content	AASHTO T265, ASTM D2216	17	10
Atterberg Limits	AASHTO T89 & T90, ASTM D4318	10	4
Moisture, Ash and Organic Matter	ASTM D2974	2	4
Incremental Consolidation	ASTM D2435 – Method A and B	1	2
Direct Simple Shear	ASTM D6528	'--	1
Compressive Strength and Elastic Moduli of Rock	ASTM D7012 – Method D	2	'- -

Selected soil and rock testing results are included on the boring logs in Appendix A and summarized on Table 3. Complete laboratory testing results are provided in Appendix C.

5.0 GENERALIZED SUBSURFACE CONDITIONS

5.1 Soil Conditions

Soils encountered at the borings were found to generally include asphalt and fill materials placed during construction of the roadway and bridge abutments overlying naturally occurring peat, organic silty clay, glaciomarine silty clay (Presumpscot Formation) and glacial till sediments. The underlying bedrock surface is interpreted to range from about 9 to 52 ft below the road grade sloping down to the east as shown on the attached interpreted subsurface profile on Figure 3. Detailed descriptions of the soil and bedrock conditions encountered at the borings are provided on the attached boring logs. The following sections summarize the major stratigraphic units.

Asphalt Pavement

Asphalt pavement thickness ranged between approximately 6 and 12 inches thick was encountered in each of the borings.

Fill

Sand and gravel subgrade fill materials ranging in thickness from approximately 3.5 to 8 ft were encountered directly beneath the asphalt pavement at the boring locations drilled through the roadway embankment. The fill materials generally consist of brown, damp, loose to medium dense, fine sand with some gravel and silt. Corrected SPT N-values[13] in the fill materials ranged from 6 to 36 with an average of 20, indicating a medium dense consistency. The fill was generally densest directly below the asphalt layer and decreased in density with depth. There was also 2 feet of fill at the top of BB-MAR-205, which is located at the toe of the embankment slope, however, this is interpreted to be from surficial sloughing of the embankment fill.

Organic Silt and Clay

A layer of organic silt and clay with peat and fine sand was encountered directly underlying the fill layer in the roadway borings and below the root mat in the toe borings. The organic layer is between 3.5 ft and 6.5 ft thick and is thicker on the east side of the bridge than on the west side. The layer consists of dark brown to gray-brown, wet, very soft to medium stiff, organic silty clay or organic silt with trace to little fine sand, trace to some fibrous organic material, and a strong organic odor. N-values ranged from WOH (weight of hammer) to 10 with an average of 5 indicating a medium stiff consistency.

The samples collected in this layer beneath the embankment, especially under the western approach, indicated presence of wood and gravel. Based on the samples and insitu testing, it is Golder's opinion that this layer may have been amended with wood and gravel during the initial approach embankment

[13] All subsequent references to SPT N-values in this report pertain to the energy corrected N_{60} values shown on the boring logs in Appendix A.

construction to improve subgrade conditions, The samples of this layer collected from the toe borings appeared to be more native (i.e., probably not amended with wood and gravel during the initial construction). Laboratory testing indicates that the material is classified as MH and OL based on the Unified Classification System (UCS) and has a natural water between 62% and 286%.

The field vane shear tests indicated an undrained shear strength between 67 psf and 1071 psf. A laboratory direct simple shear test of a sample collected beyond the northern toe of the eastern approach indicated an undrained strength of 168 psf. In general the lowest strengths (i.e., less than 350 psf) were only observed beyond the northern toe of the eastern approach fill. Insitu strengths observed beneath the embankment were approximately 1,000 psf, and insitu strengths were between approximately 350 and 700 psf beyond the southern toe of the eastern approach. As such, this layer was modeled accordingly in our analyses to reflect the variation in shear strengths observed. Refer to Figure 4 for a plot of field vane shear strength vs elevation values and Appendix E.1, Clay Design Parameters.

Glaciomarine Silty Clay (Presumpscot Formation)

A layer of glaciomarine silty clay (Presumpscot Formation) was encountered beneath the fill and organic silty clay layers. This layer varied between approximately 4 ft thick at the western end of the causeway and 24 ft thick at the eastern end of the causeway. The Presumpscot clay layer consisted of gray, moist to wet, very soft to stiff silty clay with trace fine sand and low to medium plasticity. N-values ranged from WOR (weight of rod) to 33 and field vane shear tests indicated undrained shear strengths between approximately 250 psf and 1050 psf. A stiffer crust was identified at the top of the layer. The thickness of this crust was variable but was generally observed from the bottom of the Organic Layer to roughly Elevation -15.0 (NAVD 88 datum). Refer to Figure 4 for a compilation of vaneshear strength vs elevation values. Laboratory testing indicates that the material is classified as CL based on the Unified Classification System (UCS). Laboratory tests indicated the natural water content, PI and LL generally increased with depth (water content between 20% and 44%; PI between 10 and 24; LL between 23% and 43%). Consolidation test results indicated a compression ratio between 0.14 and 0.18 and a recompression ratio of 0.01 to 0.02. Refer to Table 3 for a summary of all the laboratory results.

Glacial Till

A layer of glacial till was encountered beneath the Presumpscot clay east of the existing bridge in borings BB-MAR-104, BB-MAR-202 and BB-MAR-205 but was not sampled in the 200-series borings. This layer is interpreted to be roughly 5 to 11 feet thick and lies directly above bedrock. Sample recovery in BB-MAR-104 was poor within the till layer, so classification is based on drill rig behavior and observations in the wash water as well as descriptions from sample recovery. The glacial till layer consists of gray, moist to wet, medium dense gravel with little silt and trace clay. N-values ranged from 21 to 29, with an average of 26, indicating a medium dense consistency.

5.2 Bedrock Conditions

Bedrock was encountered in all four Phase I borings as determined from rock cores in BB-MAR-101, -102, -103 and -104; and was encountered in BB-MAR-201A, -201B, -202, -203A, -205 based on drill rig refusal conditions. Bedrock surface depths are interpreted to vary, from west to east, between about 9.1 and 51.5 ft. bgs. Based on the data obtained in the borings and the geophysical seismic refraction survey, we interpret bedrock to dip at an approximately 20% slope from shallower under the west approach to deeper under the east approach. In addition, review of the 1931 construction field book pile installation data suggests that the bedrock is deeper on the north side of the road than it is on the south side of the road in the abutment areas. Logs suggest that bedrock slopes approximately 30% down to the north under the west abutment and approximately 10% down to the north under the east abutment.

Rock quality designation (RQD) of the core samples indicated that the bedrock varies from poor (42%) in the first run of BB-MAR-101 to poor (35%) in the first run of BB-MAR-102 to good (87.5%) in the first run of BB-MAR-103B to very poor (0%) in the first run of BB-MAR-104. Core samples showed bedrock consisting of pale red to light bluish grey to dark grey, fine to medium grained biotite-granite. This rock is strong (R4), fresh (W1) to slightly weathered (W2), with extremely close to closely spaced (<20 mm to 400 mm) discontinuities. Rock Mass Rating (RMR) of the rock core runs ranged from 43 to 58 with an average of 48. Laboratory testing indicated an unconfined compressive strength (UCS) of 33,030 (BB-MAR-102) and 28,876 (BB-MAR-103B) psi. A detailed summary of the rock core is presented in Table 2.

5.3 Geophysical Survey Results/Interpretation

GPR data indicated three shallow possible buried structures are present at the locations shown on the attached figures. Two of the possible structures are located near the abutments of the existing bridge, approximately where existing timber cribbing is shown on historical plans. The third possible structure is located approximately 110 feet southwest of the Marsh Bridge. The seismic refraction survey, as shown on the attached figures, indicated the depth of bedrock below the surveyed portion of the roadway varies between about 9 and 52 feet and is deepest east of the Marsh Bridge.

5.4 Groundwater

Groundwater level measurements were measured during drilling, upon completion of the boreholes, and following removal of the casing. Groundwater level measurements taken within each roadway boring varied between about 1.8 and 8.8 feet below the roadway (~El. 2 to 8). Groundwater level measurements taken within the toe borings were observed between the ground surface and up to 1.3 feet below the ground surface. The observed variation is likely influenced by several factors including the addition of water used during the drilling process, the casing of boreholes through cohesive soils layers and tidal fluctuation and may not be representative of the actual water level. Groundwater levels also fluctuate due to other natural variations, such as seasonal/climatic influences, precipitation and temperature, and to other variations such

as construction and groundwater pumping. The interpreted groundwater level shown on the subsurface profile (Figure 3) is based on the Mean Low Water Level (El 2.6) reported by North Star Hydro[14] for Marshall Brook flowing under Marsh Bridge. The North Star Hydro report indicates that the tidal variation is only 2 ft between MLW (El. 2.6) and MHHW (El. 4.6).

[14] North Star Hydro, (2017). *Preliminary Design Hydrology and Hydraulics Report: Southwest Harbor and Tremont – Marsh Bridge #2511*.

6.0 EVALUATIONS AND RECOMMENDATIONS

The proposed design approach by VHB includes the reuse of the existing timber pile supported abutments and replacement of the bridge superstructure. The 1931 design plans**Error! Bookmark not defined.** indicated that both abutments are constructed of 13 ft tall granite block abutments resting atop concrete pile cap foundations. These foundations consist of 48 piles designed to carry 12 tons each, although the actual diameters and species of the timber piles are unknown. VHB provided excerpts from a historical (i.e., original construction era) MaineDOT Standard Specification Section 64 – Wood Piling, that suggests that the existing piles likely had a 10-inch minimum butt diameter, but not less than 8-inches at the tip. The piles are assumed to be end bearing and fully submerged below the water table. The bottom of pile cap (top of pile) for the existing abutments is at approximately El. -2 and the North Star Hydro report indicates MLLW of El.2.6. This suggests that the piles have been under over 4 ft of water throughout their life. VHB's design drawings indicate that the existing abutment will be excavated down to top of pile cap elevation so that a concrete block and dowel system can be installed over the back edge of the existing granite blocks to secure them in place.

In addition to the proposed bridge work, VHB also proposes widening the approach embankments to accommodate 4-foot wide shoulders on each side of the road. To achieve the widening and maintain a toe of slope within the right of way, the proposed grading required steepening both sides of the east and west approach embankment slopes to a 1.5H:1V riprap reinforced slope. In addition, short retaining walls (i.e., approximately 10 to 50 feet long) that extend from the four corners of the existing abutment wingwalls were added to retain the approach embankment fill within the right of way. Refer to Appendix F for VHB's proposed roadway sections and short retaining wall locations. Proposed new short retaining wall locations are also shown on Figure 2.

Due to the soft subsurface conditions, close proximity of the existing right of way, and complexity of reusing the existing foundation and abutments, many design alternatives and analyses were evaluated during the preliminary and final design phases. These included: varying earth pressures behind the abutment walls, varying pile fixity at the pile cap, installing multiple sizes, shapes and locations of retaining walls and moment slabs to retain slope fills, varying thicknesses and slopes of the proposed riprap layer, burying riprap beyond the toe of slope, and over-excavating the organic layer and replacing with improved structural fill materials. The evaluations and recommendations discussed in this report focus on the final design options selected by VHB.

6.1 Earth Pressures on Existing Abutments

To support VHBs analysis of the existing abutments, Golder estimated the earth pressure acting behind the abutments. Golder understands that VHB has not finalized the details of the proposed connection between the existing abutment and the proposed bridge structure. Therefore, Golder evaluated the earth pressures

on the existing abutments at both the active and at-rest states. Active pressure coefficients (k_a) are calculated using Coulomb Earth Pressure Theory per AASHTO 3.11.5.3 and at-rest coefficients (k_0) are calculated using AASHTO 3.11.5.2-2.

Based on review of the historic 1931 design drawings, the abutments are assumed to be approximately 13 feet tall and constructed with granite blocks bearing on a concrete pile cap. The abutments were assumed to be backfilled with controlled fill material throughout the entire depth of the wall, but have since been "contaminated" with migration of fines. Golder accounted for this by lowering the assumed backfill friction angle to 30 degrees with a unit weight of 125pcf (i.e. lower strength Granular Underwater Backfill) for soil layers adjacent to the abutment containing appreciable fines. VHBs final design drawings indicate that the entire back face of the abutment wall will be exposed, capped with concrete and backfilled with clean, compacted backfill. As such, this approach is conservative and earth pressures acting against the abutment wall in the final design condition will be less. An equivalent live load surcharge equal to 2.7 ft of soil was also assumed to be retained by the abutments.

Based on the MaineDOT Bridge Design Guide, a 3 ft unbalanced water load should be accounted for at walls located along a river. Therefore, the water level behind the wall was estimated to be 4.2 ft below the road surface grade based on low tide level data provided in the 1931 design drawings. This assumption was verified with the 2017 hydraulic data for the site from North Star Hydro. The results of this earth pressure analysis is presented in Appendix E.

6.2 Evaluation of Existing Pile Foundations

To evaluate the feasibility of reusing the existing timber pile abutment foundations, the design team considered the probable existing pile condition, diameter and axial capacity and the design loads imposed by the proposed bridge replacement. Based on the interpreted subsurface conditions, the 1931 design plans and typical MaineDOT construction practice in that era[15], it was assumed the piles are tapered with a 10-inch butt diameter and a minimum tip diameter of 8 inches, and were driven to bedrock refusal. Accordingly, the axial capacity of the piles will be governed by their structural capacity. Considering the pile head elevations are located about 4 ft below MLLW elevation, it was assumed the piles are likely to be in relatively good condition without significant decomposition. To confirm this assumption, we recommend the head section of at least one existing timber pile be exposed below the pile cap during construction and inspected. The inspection should include obtaining a core sample of the pile to verify the species of wood, a pile diameter measurement, and an assessment of pile condition.

[15] Maine Department of Transportation (1945), Book of Specifications, Section 64 – Wood Piling, pp. 241-243.

Axial loads imposed on the piles by the new bridge structure and applicable surcharge loads in accordance with AASHTO LRFD criteria will be determined by VHB. To account for estimated settlements of proposed short retaining wall structures located adjacent to the wingwalls (as discussed above and in Sections 6.3 and 6.4) we estimate that an unfactored downdrag load of 16 kips should be applied to each pile.

To assess the lateral resistance of the existing timber piles Golder developed a simplified model of the pile cap in FB-Multipier (version 5.1.1) software from the Bridge Software Institute. The FB-MultiPier analysis program uses nonlinear finite element analysis methods to evaluate the response of pile foundations subjected to various loading combinations. This analysis program couples nonlinear structural finite element analysis with nonlinear static soil models for axial, lateral and torsional soil behavior to provide a robust system of analysis for coupled bridge pier structures and foundation systems. The abutment footings were modeled as stand-alone elements with no stiffness contribution from the superstructure.

The input parameters used in the program were developed based on layer models defined by the program, laboratory test results, correlations to soil and rock characteristics determined from the field investigations, correlations to soil and rock properties identified in the FB-MultiPier user manual, and correlations to soil and rock properties in LRFD and FHWA. The soil model assumes that the pile cap is embedded 2.5 ft below the bottom of the current stream channel and the piles are embedded within a soft clay layer with their tips bearing on rock. A summary of soil and rock parameters used in the FB-MultiPier analysis for the driven piles are provided in Table 4.

As discussed in Section 6.1, Golder developed both active and at rest earth pressure diagrams in support of the analysis of the existing abutments. During preliminary design the team considered pile response at both the east and west abutments using both active and at-rest earth pressures looking at both partially pinned and fixed head pile conditions. Based on the results of the preliminary analysis and subsequent discussion with the design team, Golder analyzed the east abutment pile foundation using at-rest earth pressures which assumes the existing granite abutments will not yield sufficiently to engage active loading conditions. As requested by VHB, the final analysis also assumed a fixed head connection between the piles and the pile cap. To assess the lateral stiffness under service loading conditions and the maximum axial forces and moments within the timber piles under strength loading conditions, VHB provided Golder with the following vertical, lateral, and moment forces acting on the pile caps at each abutment:

Load Case	Vertical Force (kip/ft)	Lateral Force (kip/ft)	Moment Force (kip-ft/ft)
Load Case 1: At-Rest Strength	18.8	4.8	15.0
Load Case 2: At-Rest Service	12.8	3.3	9.9

Note: VHB revised the differential water effects from the earth pressure calculation provided by Golder as described in section 6.1 and presented in Appendix E, to reflect a 1 foot water differential between front and back abutment faces, when calculating the pile cap loads presented in this table.

Golder subsequently used FB-Multipier to analyze pile group response under these loading conditions. The timber piles were modeled as linear elastic elements in which the user provides gross section properties of the piles.

The existing "horseshoe" shaped pile cap and pile spacing configuration, as shown in Maine Highway Commission Bridge Plans (1931) in Appendix G, was simplified for modelling purposes to exclude the wingwalls resulting in a 27.7 ft by 6.0 ft rectangular shape. The 1931 design plans indicated that each abutment consists of 48 piles designed to carry 12 tons each, although the actual diameters and species of the timber piles are unknown. Typical MaineDOT construction practices of the era indicate that tapered piles with a butt diameter of 10-inches and a tip diameter of 8-inches were common[15]. The construction specification also list several accepted species of wood that could be used as timber piling. For the purposes of the existing foundation analysis, Golder assumed a red-oak timber species, which was a MaineDOT accepted species**Error! Bookmark not defined.**, because it provides slightly more conservative wood strength parameters than the red pine. For lateral resistance modeling purposes Golder assumed a uniform diameter of 8-inches to account for possible section loss at the top of the pile. This section loss could be caused by a variety of factors including wood decay, insect damage and driving damage. A pile length of 19 feet was used in the east abutment model. No P-Delta effects were modeled. The pile parameters used in the model were correlated from the literature and are presented below:

Strength Parameters	Assumed 8-inch Pile Properties
Young's Modulus[16] (E), ksi	1250
Unit Weight[17], pcf	64
Shear Modulus (G)[18], ksi	100

The results of these analyses are shown in the calculations presented in Appendix E and are summarized below:

East Abutment - Fixed Head and At-Rest Earth Pressures	
Maximum Moment at Strength Limit State Loading (kip-ft)	16.9
Maximum Axial Force at Strength Limit State Loading (kips)	-40.2
Deflection at Service Limit State Loading (in)	0.4

[16] ASD Manual for Engineered Wood Construction (2003), "Timber Poles and Piles," page PP-13.
[17] Chellis, Robert D., (1961), "Pile Foundations," 2ed, McGraw-Hill Book Company, New York.
[18] MnDOT (1992), "USFS Timber Bridge Manual," p 3-18.

6.3 Global Stability

The final design includes widening the approach embankments to accommodate 4-foot wide shoulders on each side of the road. To achieve the widening and maintain a toe of slope within the right of way, the proposed grading requires steepening both sides of the approach embankment slope to 1.5H:1V and installing short (i.e., approximately 10 to 50 feet long) embankment retaining walls that extend from the four corners of the existing abutment wingwalls. As discussed above, an approximately 5 ft thick organic layer over a varying thickness of clay was encountered under the approach fills and beyond the existing toe of slope. The presence of this soft organic layer over the clay strata presents design restrictions for embankment widening (i.e., steepened slopes and retaining walls) considering global stability mechanisms and wall foundation bearing capacity and settlement.

Based on field observations and the Preliminary Design Hydrology and Hydraulics Report, the stability models assumed a water level at EL. 4 in the embankment fill materials and EL. 2.6 at the toe. This corresponds to the MLW at the toe and an average water level observed within the embankment. It is assumed that a lag effect occurs during tidal fluctuations and the phreatic surface will remain higher within the embankment during the MLW event.

Golder evaluated the existing subsurface conditions and VHB's proposed final design conditions and selected seven representative design sections to evaluate global stability. The proposed design alternatives are required to prevent encroachment on the state owned right-of-way and achieve a minimum factor of safety of 1.3. The typical design sections analyzed and slip surfaces with the lowest calculated factors of safety are presented in Appendix E.4 and include the following:

- Station 15+50 Left (Figure E-2): This section models a retaining wall system with a two foot thick 1.5H:1V riprap slope in front. To achieve a global stability FS>1.3, this section requires all organic materials below the proposed wall structure to be removed and replaced with a select granular fill material and that a minimum depth to height ratio of 0.7H be installed. Station 15+50 Left is intended to be representative of northern and southern embankment retaining wall sections on the western approach fill.
- Station 15+50 Right (Figure E-3): This section models a two feet thick 1.5H:1V riprap slope roughly at the end of the 10 feet long retaining wall extending back from the abutment. This section has a FS>1.3 as proposed by VHB. Station 15+50 Right is intended to be representative of steepened slope sections over the western approach that do not include a retaining wall.
- Station 16+00 Left (Figure E-4): This section models a two foot thick 1.5H:1V riprap slope roughly at the end of the 10 feet long retaining wall extending back from the abutment. The calculated minimum FS is 1.32. This section is intended to be representative of embankment slopes from ~ Station 16+00 (past the end of the retaining wall) to ~ Station 17+50 Left.
- Station 16+15 Right (Figure E-5): This section models a two foot thick 1.5H:1V riprap slope roughly at the end of the 10 feet long retaining wall extending back from the abutment. To achieve a global stability FS>1.3, this section requires a surcharge load, equivalent to 350

psf, to be placed at the toe of the riprap slope. The required surcharge is approximately equivalent to a 2.5 ft tall by 5 ft wide gabion basket. This section is intended to be representative of embankment slopes from ~ Station 16+00 (past the end of the retaining wall) to ~ Station 17+50 Right.

- Abutment Corner Wall (Figure E-6): This section is intended to represent the northern and southern eastern approach embankment retaining walls near the end of the existing abutment wingwalls. The section is cut at the SE abutment corner (see Figure E-9) and includes a retaining wall with a 2 ft thick 1.5H:1V riprap slope in front. To achieve a global stability with a FS > 1.3, this section requires all organic materials below the proposed wall structure to be removed and replaced with a select fill material and that a minimum depth to height ratio of 0.7H be installed
- Station 17+50 Left (Figure E-7): This section models a two foot thick 1.5H:1V riprap slope. This section has a FS>1.3. Station 17+50 Left is intended to be representative of embankment slopes from ~ Station 17+50 to ~ Station 18+50 Left.
- Station 17+50 Right (Figure E-8): This section models a soil fill placed at a 3H:1V slope. This section has a FS>1.3. Station 17+50 Right is intended to be representative of embankment slopes from ~ Station 17+50 to ~ Station 18+50 Right.

6.4 Embankment Retaining Wall Bearing Capacity and Allowable Settlement

The final design provided by VHB details four retaining walls extending from the abutment corner wingwalls as shown on Figure 2 and the drawings in Appendix F. Three of the walls (northeast, southeast, southwest) are proposed to be roughly 10 feet long and the fourth, which extends from the northwest abutment corner, is roughly 50 feet long. Based on the results of global stability analyses discussed in Section 6.3 evaluated for the typical wall sections, Golder determined that a retaining wall system was globally stable within the proposed embankment grading as long as it extended at least 0.7H back into the embankment and all of the organic layer was removed beneath the wall system (i.e., the wall needs to bear directly on the upper stiffer crust of the clay layer or on compacted fill extending down to the clay layer).

Since the proposed walls are planned to bear on stiff clay overlying soft clay, settlement and bearing resistance for the wall system is also a concern. Golder and the design team discussed possible wall system options and determined that a flexible wall would have a higher tolerance for foundation settlement than a typical rigid gravity retaining wall system. Golder contacted flexible wall system suppliers (i.e., Modular Block Retaining Walls - Redi-Rock and The Neel Company (T-Wall)) to discuss the feasibility of their wall systems for the Marsh Bridge application, discuss tolerable total and differential settlements for their system, and evaluate preliminary analyses for strength and service limit state bearing resistances. The results of the study indicated that either wall system (Redi-Rock or T-Wall) appears to be feasible for the proposed embankment retaining wall applications. Redi-Rock personnel indicated that their concrete block wall facings could tolerate ½ to 1 inch of differential settlement over 10 ft of wall. T-Wall personnel indicated that their system could tolerate 3 inches of differential settlement. Based on the results of our global stability analyses and the recommended tolerable settlements and allowable bearing pressures from the retaining wall suppliers, Golder evaluated allowable bearing pressures for up to 3 inches of settlement

for walls bearing on a 6 and 8 ft wide pad. These analyses are described below and included in Appendix E Calculations.

Golder's retaining wall sections were evaluated for bearing resistance and settlement as a single uniform soil block which assumes any applied load is distributed equally over the entire wall system and the system will bear and settle uniformly. As described in Section 5.1, the clay deposit thickness varies significantly from the west to east abutment, so separate soil profiles were used for design at each of the abutments.

The bearing capacity and settlement calculations were performed for the west abutment for a 50 ft long wall and a 10 ft long wall. At the east abutment the calculations were performed for a 10 ft long wall. For each of the three wall scenarios, retaining wall systems with depths into the embankment of 6 ft and 8 ft were considered. Each of the wall combinations were analyzed to determine the factored strength limit state bearing resistance as well as the allowable service bearing stress corresponding to settlements of 1, 2 and 3-inches. The results of these analyses are summarized in the table below. Due to the limitations of the available subsurface data and the short length of the retaining walls, differential settlements were not evaluated, but are estimated to be up to half of the total settlement.

Footing Size	**East or West Approach**	**Strength Limit State Bearing Resistance (ksf)**	**Service Limit State Bearing Resistance (ksf)***		
			1" Settlement	2" Settlement	3" Settlement
50ft x 6ft	West Approach	3.64	4.47	6.02	7.98
50ft x 8ft	West Approach	3.46	3.79	5.11	6.77
10ft x 6ft	East Approach	4.43	1.56	2.40	3.17
10ft x 8ft	East Approach	4.32	1.26	1.99	2.61
10ft x 6ft	West Approach	4.43	4.47	6.02	7.98
10ft x 8ft	West Approach	4.32	3.79	5.11	6.77

*Note: The service limit state bearing resistance values are based on primary consolidation settlement.

To obtain the bearing reistances presented above, the walls must be founded on a minimum 2 ft thick compacted structural fill pad bearing on the Presumpscot Clay stiff crust and all organic material must be removed from below the bottom of the wall structure. The wall should be founded a minimum of four feet below the top of the exterior rip rap slope in front of the wall. The compacted fill pad should extend out at a 1H:1V slope from the toe and heel of the wall bearing elevation until the native Presumpscot Clay is encountered. The structural designer can select the design bearing capacity based on the amount of settlement that the wall can tolerate. It is assumed that the final design and detailing of the retaining wall system will be performed by the selected wall supplier.

It can be anticipated that 90% of the primary settlement will occur within about 3 months after wall and backfill placement for the west walls, and within about 31 months (2.6 years) after wall and backfill placement for the east walls.

6.5 Widened Embankment Fill Settlement

The widened embankment fill at both approaches, beyond the retaining walls, is proposed to be armored with a 2 ft thick 1.5H:1V riprap blanket placed over the existing side slopes. As shown in the global stability analysis, the riprap blanket can be placed above the organic silt and clay layer. However, we estimate that up to roughly 14-inches of slope settlement should be anticipated when riprap is placed on the organic silt layer. The majority of this settlement occurs within the organic layer and Golder estimates that 50% of the settlement will occur within 5 months of riprap placement and 90% consolidation will require approximately 20 months after placement to complete.

7.0 CONSTRUCTION CONSIDERATIONS

Construction activities will require soil excavation and will likely include earth support systems to support approach fills during construction. The contractor should control groundwater and surface water infiltration so that subgrade preparation and backfill placement can be completed in the dry. The native soils at the site are susceptible to disturbance as a result of exposure to water and construction traffic. All subgrade surfaces should be protected from unnecessary construction traffic. If rutting occurs or loose or soft materials are encountered in the excavation, the loose, soft, or disturbed areas should be removed and replaced with compacted Granular Borrow for Underwater Backfill (MaineDOT 703.19) or ¾ inch crushed stone (MaineDOT 703.13). In addition, remnants of an original timber crib structure was located by HR's geophysical survey. If the timber crib structure is encountered during subgrade preparation for the embankment retaining walls, the timbers should be over-excavated and replaced with compacted granular borrow or crushed stone.

7.1 Timber Pile Condition Assessment

As discussed in Section 6.2 we recommend that the head of at least one existing timber pile supporting one of the existing bridge abutments be exposed during construction to confirm the timber species, the pile diameter and the general condition of the pile including an assessment of pile deterioration, if any. A core sample of the pile should be obtained as part of the assessment. The observed conditions should be compared to the design assumptions to verify the appropriateness of the foundation design.

7.2 Abutment Backfill

Backfill adjacent to the abutment wall should consist of Granular Borrow Underwater Backfill (Backfill, MaineDOT Section 703.19) and be separated from the existing embankment fill soils with a geotextile separation fabric (Item 722.04 Separation Geotextile). As recommended in the MaineDOT Bridge Design Guide, granular backfill should extend a horizontal distance of 10 ft from the back face of the abutment wall. The backfill should be placed in 6 to 8 inch loose lifts and compacted to a minimum of 95 percent of the maximum dry density as determined by AASHTO T-180. Mechanical tampers or approved alternative, should be used to compact the backfill adjacent abutments walls. In addition, since the proposed final design includes capping (i.e., sealing) the backside of the granite block abutment walls, any existing gaps between the existing block walls that may have promoted positive drainage will be sealed. Therefore, it is recommended that a french drain with outlets be installed at the base of the abutment wall.

7.3 Embankment Retaining Walls

Golder's design calculations were performed for a generic flexible wall system and final design recommendations will be the responsibility of the selected wall supplier design team. The following recommendations are based on the criteria used in our analyses and are recommended for design.

However, once a wall supplier is selected and submits a final wall system design, Golder should review the submitted design to verify that our recommendations still apply.

Golder recommends the embankment retaining walls be founded on a minimum 2 ft thick structural fill pad bearing on the stiff Presumpscot Clay crust and all organic material must be removed from below bottom of the wall structure. The structural fill pad shall be constructed from Item 703.19 Granular Borrow for Underwater Backfill and founded a minimum of 4 ft below the top of the exterior rip rap slope in front of the wall to protect the wall from frost action. The compacted fill pad should extend out at a 1H:1V slope from the toe and heel of the wall bearing elevation until the native Presumpscot Clay is encountered. Retaining wall backfill shall meet the requirement described above for Abutment Backfill or as recommended by the wall supplier and approved by the geotechnical engineer. In all cases, the design shall follow the recommendations outlined in Section 5.6.5 of the MaineDOT Bridge Design Guide for Prefabricated Proprietary Walls and MaineDOT Standard Specification Section 674 for Prefabricated Concrete Modular Gravity Wall. The walls should consist of Class “LP” concrete and epoxy coated rebar. The precast units should contain calcium nitrate solution or equivalent to inhibit corrosion.

7.4 Rip Rap Slopes

The existing ground surface should be prepared to the grades shown on the design drawings and covered with a non-woven erosion control geotextile meeting the requirements of Standard Specification 722.03. The geotextile should be covered with a 12-inch thick layer of cushion material (Item 703.19 Granular Borrow for Underwater Backfill) prior to placement of the riprap blanket (Item 703.26 Plain and Hand Laid Riprap). As discussed above, the organic material beneath the riprap blanket is expected to experience significant settlement (i.e. in excess of 12-inches). Golder recommends that the contractor perform routine inspection of the slopes throughout construction and place additional riprap, as necessary, to maintain a 1.5H:1V slope.

8.0 CLOSING

This Geotechnical Design Report was prepared for the exclusive use of VHB for specific application to the proposed Marsh Bridge Replacement, in the towns of Southwest Harbor and Tremont, Maine, in accordance with generally accepted soil and foundation engineering practices practiced in this geographical area and under similar time and financial constraints. Golder makes no other warranty, either express or implied. In the event that any changes in the nature, design or location of the proposed project are planned, Golder should be notified to review the appropriateness of our conclusions and recommendations, and to modify the recommendations as appropriate to reflect the changes in design. In addition, Golder should review the final plans and specifications to evaluate compliance with these recommendations.

Our analyses, and recommendations are based, in part, on information obtained from the referenced subsurface explorations completed at the discrete locations described in the report. Readers of this report should make their own interpretation of the results of the subsurface investigations and laboratory testing. Variations in the nature and extent of subsurface conditions between explorations should be expected. Golder should be notified if conditions encountered during construction vary from those described in this report so that we may re-evaluate, and if necessary, revise the recommendations made in this report.

The professional services provided by Golder for this project include only the geotechnical aspects of the subsurface conditions at this site. The presence or implications of possible surface and/or subsurface contamination resulting from previous activities or uses of the site and/or resulting from the introduction onto the site of materials from off-site sources are outside the terms of reference for this report and have not been investigated or addressed.

TABLES

Table 1: Subsurface Exploration Locations
Geotechnical Design Report
Marsh Bridge Replacement
Tremont/Southwest Harbor, Maine
MaineDOT WIN: 21702.00

Test Boring Designation[1,2,3]	As-Drilled Locations[4,5]		Existing Ground Surface Elevation[4,5] (feet)	Boring Depth[6] (feet)
	Northing	Easting		
BB-MAR-101	153436.3276	2174473.066	10.63	18.10
BB-MAR-102	153447.9209	2174535.295	10.94	29.60
BB-MAR-103A	153477.6211	2174568.948	10.81	8.33
BB-MAR-103B	153476.5764	2174566.402	10.88	42.40
BB-MAR-104	153500.676	2174622.772	9.29	60.20
BB-MAR-201A	--	--	10	18.0
BB-MAR-201B	--	--	10	15.0
BB-MAR-202	--	--	10	43.3
BB-MAR-203A	--	--	4	29.0
BB-MAR-203B	--	--	4	18.0
BB-MAR-204	--	--	4	16.0
BB-MAR-205	--	--	4	38.3

Notes:

1. Test boring locations are shown in Figure 1 - "Boring Location Plan".
2. Borings BB-MAR-101, BB-MAR-102, BB-MAR-103A, BB-MAR-103B, and BB-MAR-104 were performed by the New England Boring Contractors in May, 2017.
3. Borings BB-MAR-201A, BB-MAR-201B, BB-MAR-202, BB-MAR-203A, BB-MAR-203B, BB-MAR-204, and BB-MAR-205 were performed by the New England Boring Contractors in September, 2017.
4. As-drilled locations and elevations of borings BB-MAR-101, BB-MAR-102, BB-MAR-103A, BB-MAR-103B, and BB-MAR-104 are based on MaineDOT survey information: "Borings_19May2017.dgn" received by Golder on May 19, 2017.
5. Elevations of borings BB-MAR-201A, BB-MAR-201B, BB-MAR-202, BB-MAR-203A, BB-MAR-203B, BB-MAR-204, and BB-MAR-205 estimated from basemap provided by VHB on April 27, 2017.
6. Depth below ground surface.
7. Boring logs presented in Appendix A.

Prepared By: LLM
Checked By: TRM
Reviewed By: MCM

Table 2: Summary of Rock Core Quality
Geotechnical Design Report
Marsh Bridge Replacement
Tremont/Southwest Harbor, Maine
MaineDOT WIN: 21702.00

Test Boring Designation	Core Size	Existing Ground Surface Elevation[1]	Run						TCR[2]		RQD[3]			Physical Rock Parameters			Lithologic, Rock Mass and Discontinuity Description
			No.	Midpoint Depth Below Bedrock Surface	Depth Below Ground Surface [ft.]			Length	Length	%	Length	%	Designation	Weathering[4]	Estimated Field Strength[4]	Rock Mass Rating [RMR][5]	
	[in]	[ft.]		[ft.]	Start	End	Midpoint	[ft.]	[ft.]		[ft.]						
BB-MAR-101	NQ [1.875"]	10.63	R1	2.10	9.10	13.30	11.20	4.2	3.1	73%	1.8	42%	Poor	Fresh (W1)	Strong (R4)	47	9.10 ft. - 13.30 ft.: Pale Brown (5YR 5/2) to Light Bluish Grey (5B 7/1), fine to medium grained, GRANITE, strong (R4), fresh (W1), discontinuities dipping 45 to 90 degrees relative to horizontal axis, spacing is extremely close to closely spaced (<20 mm to 190 mm), discontinuity surfaces are planar to curved to irregular, texture is smooth to very rough, some white quartz infilling discontinuities [Castine Formation (Devonian-Silurian)].
			R2	6.60	13.30	18.10	15.70	4.8	4.8	100%	3.3	69%	Fair	Fresh (W1) to Slightly Weathered (W2)	Strong (R4)	51	13.30 ft. - 18.10 ft.: Light Bluish Grey (5B 7/1) to Medium Bluish Grey (5B 5/1), fine to medium grained, GRANITE, strong (R4), fresh to slightly weathered (W1 - W2), discontinuities dipping 20 to 90 degrees relative to horizontal axis, spacing is very close to moderately spaced (30 mm to 280 mm), discontinuity surfaces are planar to stepped, texture is smooth to very rough, discontinuities have some calcite and silt infilling, light yellow in color, soft, and possibly gypsum [Castine Formation (Devonian-Silurian)].
BB-MAR-102	NQ [1.875"]	10.94	R1	2.50	19.60	24.60	22.10	5.0	4.3	87%	1.8	35%	Poor	Fresh (W1) to Slightly Weathered (W2)	Strong (R4)	43	19.60 ft. - 18.10 ft.: Light Bluish Grey (5B 7/1), fine to medium grained, GRANITE, strong (R4), fresh to slightly weathered (W1 - W2), discontinuities dipping at 20 to 90 degrees relative to horizontal axis, spacing is extremely close to closely spaced (<20 mm to 160 mm), discontinuity surfaces are planar to irregular to stepped, texture is smooth to very rough, some soft red material, possibly silt, infilling joints [Castine Formation (Devonian-Silurian)].
			R2	7.50	24.60	29.60	27.10	5.0	4.8	97%	2.6	52%	Fair	Fresh (W1) to Slightly Weathered (W2)	Strong (R4)	50	24.60 ft. - 29.60 ft.: Medium Dark Grey (N4) to Moderate Red (5R 5/4), fine to medium grained, GRANITE, strong (R4), fresh to slightly weathered (W1 - W2), discontinuities dipping at 45 to 90 degrees relative to horizontal axis, spacing is very close to moderately spaced (30 mm to 300 mm), discontinuity surfaces are planar to curved to stepped, texture is smooth to rough, no infilling material was observed [Castine Formation (Devonian-Silurian)].
BB-MAR-103B	NQ [1.875"]	10.88	R1	2.50	32.40	37.40	34.90	5.0	4.7	93%	4.4	88%	Good	Fresh (W1) to Slightly Weathered (W2)	Strong (R4)	58	32.40 ft. - 37.40 ft.: Pale Red (5R 6/2), medium to fine grained, GRANITE, strong (R4), fresh to slightly weathered (W1 - W2), discontinuities dipping at 10 to 90 degrees relative to core axis, spacing is extremely close to moderately spaced (20 mm to 400 mm), discontinuity surfaces are planar to curved to irregular, texture is smooth to very rough, insipient joints have some mineral growth, green in color, possibly chlorite [Castine Formation (Devonian-Silurian)].
			R2	7.50	37.40	42.40	39.90	5.0	5.0	100%	2.8	55%	Fair	Fresh (W1) to Slightly Weathered (W2)	Strong (R4)	52	37.40 ft. - 42.40 ft.: Pale Red (5R 6/2), medium to fine grained, GRANITE, strong (R4), slightly weathered to fresh (W1 - W2), discontinuities dipping at 10 to 90 degrees relative to core axis, spacing is very closely spaced to moderately spaced (20 mm to 400 mm), discontinuity surfaces are planar to curved to irregular, texture is smooth to very rough, insipient joints have some mineral growth, green in color, possibly chlorite [Castine Formation (Devonian-Silurian)].
BB-MAR-104	NQ [1.875"]	9.29	R1	0.50	51.50	52.50	52.00	1.0	0.7	67%	0.0	0%	Very Poor	Fresh (W1) to Slightly Weathered (W2)	Strong (R4)	44	51.50 ft. - 52.50 ft.: Dark Grey (N3) to Light Bluish Grey (5B 7/1), fine to medium grained, GRANITE, strong (R4), fresh to slightly weathered (W1 - W2), discontinuities are nearly horizontal to vertical, spacing is extremely close to closely spaced (<20 mm to 90 mm), discontinuity surfaces are planar to curved to stepped, texture is smooth to very rough [Castine Formation (Devonian-Silurian)].
			R2	0.50	52.50	55.20	53.85	2.7	2.7	100%	0.8	28%	Poor	Fresh (W1) to Slightly Weathered (W2)	Strong (R4)	47	52.50 ft. - 55.20 ft.: Dark Grey (N3) to Light Bluish Grey (5B 7/1), fine to medium grained, GRANITE, strong (R4), fresh to slightly weathered (W1 - W2), discontinuities are nearly horizontal to vertical, spacing is extremely close to closely spaced (<20 mm to 130 mm), surfaces are planar to curved to stepped, texture is smooth to very rough [Castine Formation (Devonian-Silurian)].
			R3	38.10	55.20	60.20	57.70	5.0	5.0	100%	2.3	45%	Poor	Fresh (W1) to Slightly Weathered (W2)	Strong (R4)	44	55.20 ft. - 60.20 ft.: Dark Grey (N3) to Light Bluish Grey (5B 7/1), fine to medium grained, GRANITE, strong (R4), fresh to slightly weathered (W1 - W2), discontinuities are nearly horizontal to vertical, spacing is extremely close to closely spaced (<20 mm to 200 mm), surfaces are planar to curved to stepped, texture is smooth to very rough [Castine Formation (Devonian-Silurian)].

Notes:

1. As-drilled surface elevations were surveyed by MaineDOT and provided to Golder on May 19, 2017.
2. TCR = total core recovery. Total core recovery is the length of core recovered divided by the length of the run.
3. 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 4 inches) 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.
4. Weathering and Estimated Field Strength based on Tables II.4 and II.3 (respectively) in Willey, 2004 (based on ISRM, 1981).
5. Rock Mass Rating System (RMR; Bieniawski, 1989) assigns numerical ratings to six parameters, including the strength of the intact rock, the RQD, the discontinuity spacing, discontinuity conditions, groundwater conditions, and orientation of discontinuities. These ratings are summed to give the RMR value. As proposed foundations or tie-back designs are not complete, the rating adjustment for joint orientation was assigned a value of 0.
6. ft. = feet, in = inches

Prepared by: LLM
Checked by: JRS
Reviewed by: MCM

Golder Associates

Table 3: Summary of Laboratory Testing Results
Geotechnical Design Report
Marsh Bridge Replacement
Tremont/Southwest Harbor, Maine

Test Boring Designation[1]	Existing Ground Surface Elevation [2,3] (ft.)	Sample Number	Sample Depth Below Ground Surface (ft.)	Approximate Sample Elevation (ft.)	Sieve Minus No. 200 (%):	Natural Moisture Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	Liquidity Index	Ash Content (%)	Organic Content (%)	Consolidation		Corrected Peak Shear Stress (psf)	AASHTO Soil Classification[5]	USCS Soil Classification[5]	Rock Uniaxial Compressive Strength (psi)
													Compression Ratio[4] $C_{\varepsilon c}$	Recompression Ratio[4] $C_{\varepsilon r}$				
BB-MAR-101	10.63	1D	1.0 - 3.0	8.63	27.9	9.8	- -	- -	- -	- -	- -	- -	- -	- -	- -	A-2-4	SM	- -
		2D	5.0 - 7.0	4.63	72.3	23.5	31	20	11	0.3	- -	- -	- -	- -	- -	A-6 (6)	CL	- -
BB-MAR-102	10.94	1D	1.0 - 3.0	8.94	13.6	8.0	- -	- -	- -	- -	- -	- -	- -	- -	- -	A-1-b (0)	SM	- -
		2D	5.0 - 7.0	4.94	- -	27.7	40	22	18	0.3	- -	- -	- -	- -	- -	A-6	CL	- -
		3D	12.0 - 14.0	-2.06	85.0	19.9	25	14	11	0.5	- -	- -	- -	- -	- -	A-6 (7)	CL	- -
		4D	15.0 - 17.0	-5.06	- -	26.1	38	18	20	0.4	- -	- -	- -	- -	- -	A-6	CL	- -
		R2	24.8 - 25.2	-14.01	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	33,030
BB-MAR-103A	10.81	2D	2.5 - 4.5	7.31	29.5	12.2	- -	- -	- -	- -	- -	- -	- -	- -	- -	A-2-4 (0)	SM	- -
BB-MAR-103B	10.88	5D	8.0 - 10.0	1.88	79.9	62.0	32	19	13	3.3	92.3	7.7	- -	- -	- -	A-6 (9)	CL	- -
		10D	18.0 - 20.0	-8.12	- -	25.4	23	13	10	1.2	- -	- -	- -	- -	- -	A-4	CL	- -
		11D	20.0 - 22.0	-10.12	- -	37.2	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
		12D	22.0 - 24.0	-12.12	- -	41.7	41	20	21	1	- -	- -	- -	- -	- -	A-7-6	CL	- -
		R1	34.8 - 35.1	-24.07	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	28,876
BB-MAR-104	9.29	5D	10.5 - 12.5	-2.21	93.9	271.0	- -	- -	- -	- -	64.3	35.7	- -	- -	- -	A-4 (0)	OL	- -
		6D/B	12.5 - 14.5	-4.21	96.9	28.0	35	17	18	0.6	- -	- -	- -	- -	- -	A-6 (17)	CL	- -
		7D	15.0 - 17.0	-6.71	- -	35.5	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
		8D	17.0 - 19.0	-8.71	- -	35.9	35	18	17	1.1	- -	- -	- -	- -	- -	A-6	CL	- -
		U1	20.0 - 21.4	-11.41	- -	- -	- -	- -	- -	- -	- -	- -	0.14	0.02	- -	- -	- -	- -
		9D	25.0 - 27.0	-16.71	99.0	43.6	43	19	24	1	- -	- -	- -	- -	- -	A-7-6 (26)	CL	- -
		11D	35.0 - 37.0	-26.71	- -	40.9	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
BB-MAR-201B	10	3D	10.0 - 12.0	-1.00	- -	22.6	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
BB-MAR-202	10	3DB	9.0 - 11.0	0.00	- -	193.0	- -	- -	- -	- -	64.8	35.2	- -	- -	- -	- -	- -	- -
		6D	15.0 - 17.0	-6.00	- -	25.1	34	17	17	0.5	- -	- -	- -	- -	- -	- -	CL	- -
		11D	28.0 - 30.0	-19.00	- -	42.7	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
		U1	35.0 - 37.0	-26.00	- -	35.2	- -	- -	- -	- -	- -	- -	0.18	0.01	- -	- -	- -	- -
BB-MAR-203A	4	2DA	2.0 - 4.0	1.00	- -	159.0	- -	- -	- -	- -	84.9	15.1	- -	- -	- -	- -	- -	- -
		U1	6.5 - 8.5	-3.50	72.0	286.0	255	100	155	1.2	71.9	28.1	0.37	0.12	168.0	A-7-5 (130)	MH	- -
		3D	8.5 - 10.5	-5.50	- -	27.3	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
		4D	11.5 - 13.5	-8.50	- -	32.0	35	18	17	0.8	- -	- -	- -	- -	- -	- -	CL	- -
		6D	21.0 - 23.0	-18.00	- -	43.6	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
BB-MAR-203B	4	6D	12.0 - 14.0	-9.00	- -	36.9	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
BB-MAR-204	4	3D	8.0 - 10.0	-5.00	- -	29.0	- -	- -	- -	- -	99.4	0.6	- -	- -	- -	- -	- -	- -
		4D	10.0 - 12.0	-7.00	- -	36.1	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
		6D	14.0 - 16.0	-11.00	- -	37.8	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
BB-MAR-205	4	4DB	8.0 - 10.0	-5.00	- -	25.0	32	17	15	0.5	- -	- -	- -	- -	- -	- -	CL	- -
		9D	18.0 - 20.0	-15.00	- -	39.5	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -
		12D	26.0 - 28.4	-23.20	- -	42.4	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -	- -

Notes:

1. Test boring locations are shown in Figure 2 - "Boring Location Plan"
2. As-drilled locations for BB-MAR-101, BB-MAR-102, BB-MAR-103A, BB-MAR-103B, and BB-MAR-104 were surveyed by MEDOT and received by Golder on May 19, 2017.
3. Elevations of borings BB-MAR-201A, BB-MAR-201B, BB-MAR-202, BB-MAR-203A, BB-MAR-203B, BB-MAR-204, and BB-MAR-205 estimated from basemap provided by VHB on April 27, 2017.
4. Compression and recompression ratios determined using Casagrande (1936) method. Refer to Appendix E for Casagrande Constructions.
5. AASHTO and USCS symbols assigned based on interpretation of laboratory test results.
6. Laboratory testing was performed by GeoTesting Express, Inc. Complete laboratory test results are provided in Appendix C.

Prepared By: LLM
Checked By: TRM
Reviewed By: MCM

Table 4: Summary of Soil Parameters for FB MultiPier Analysis - Existing Timber Piles
Geotechnical Design Report
Marsh Bridge Replacement WIN 21702.00
Tremont & Southwest Harbor, Maine
MaineDOT WIN 21702.00

Layer Description	Top of Layer (Depth BGS, ft)	Recommended Lateral Layer Model	Recommended Axial Layer Model	Recommended Torsional Layer Model	Total Unit Weight (lb/ft^3)	Friction Angle, ϕ	Shear Modulus, G (kip/in^2)	Torsional Shear Stress, τ (kip/ft^2)	Uniaxial Compressive Strength (lb/ft^2)	Undrained Shear Strength (lb/ft^2)	ε_{50}	Poisson's Ratio, v
Soft, Silty CLAY	13	Clay (Soft<Water) Clay(API)	Driven Pile	Hyperbolic	0.115	n/a	Top: 1.65	0.1	n/a	Top: 900	0.01	Top: 0.45
							Bottom: 0.6			Bottom: 400	0.02	Bottom: 0.4
Soft, Silty CLAY	25.5	Clay (Soft<Water) Clay(API)	Driven Pile	Hyperbolic	0.115	n/a	0.60	0.1	n/a	400	0.02	0.4
GRANITE	34.5	Limestone (McVay)	Driven Pile	Hyperbolic	0.165	34	4750	0.7	4032000	n/a	n/a	0.21

Notes:

1. Unless otherwise noted, all properties used in the model are assumed based on the FB-MultiPier Soil Parameter Table (US Customary Unit) (2013), Bridge Software Institute, University of Florida.

2. The shear modulus, uniaxial compressive strength and Poisson's ratio for bedrock was taken from Geotesting Express laboratory data provided on 6/8/2017 and shown in Appendix C.

3. The undrained shear strength used in the clay model is based on the average field vane measurements taken during the field exploration and shown in the Interpreted Subsurface Profile (Figure 3).

Prepared By: CJS
Checked By: CCB
Reviewed By: MCM

FIGURES



REFERENCES

BASEMAP TAKEN FROM USGS 7.5 MINUTE SERIES (TOPOGRAPHIC) QUADRANGLE MAP TITLED, "SOUTHWEST HARBOR, MAINE" AND "BASS HARBOR, MAINE" DATED 1983.

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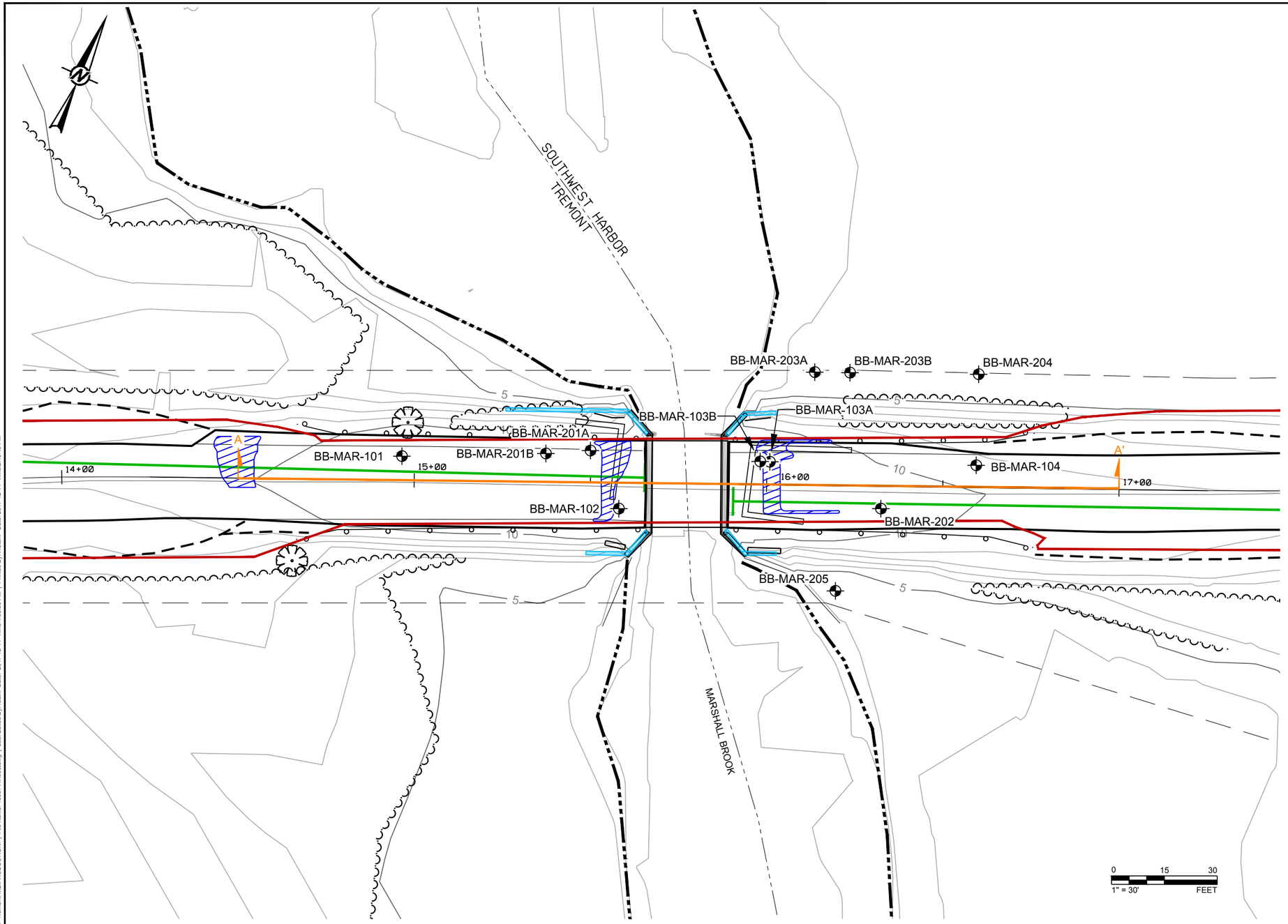
PROJECT
MARSH BRIDGE REPLACEMENT
TREMONT AND SOUTHWEST HARBOR, MAINE
MAINEDOT WIN 21702.00

TITLE
SITE LOCATION MAP

PROJECT NO.	SUBTITLE	REV.	FIGURE
166-8741	A	0	1

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IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI A



LEGEND

- 5 FT INDEX CONTOUR (FT MSL)
- 1 FT INDEX CONTOUR (FT MSL)
- EDGE OF DRAINAGE WATERWAY
- GUARDRAIL BEAM
- EDGE OF PAVEMENT
- EDGE OF ROADWAY SHOULDER
- SEISMIC REFRACTION SURVEY LINE (SEE HAGER-RICHTER REPORT)
- APPROXIMATE LIMITS OF GPR SURVEY AREA (SEE HAGER-RICHTER REPORT)
- RIGHT OF WAY
- TREELINE
- DECIDUOUS TREES
- BB-HAMP-101 — APPROXIMATE BORING LOCATION FROM 2017 GOLDER EXPLORATIONS
- A — SUBSURFACE PROFILE LOCATION AND DIRECTION (FIGURE 3)
- APPROXIMATE LOCATION OF EXISTING TIMBER CRIBBING (SEE REF. 4)
- ESTIMATED LOCATION OF TIMBER CRIB FROM HAGER-RICHTER SURVEY MAY 2017 & 1931 HISTORICAL DRAWINGS
- PROPOSED RETAINING WALL LOCATIONS

NOTES

1. THE 100-SERIES BORING LOCATIONS SHOWN IN THIS FIGURE WERE SURVEYED BY MAINEDOT AND RECEIVED BY GOLDER ON MAY 19, 2017. GOLDER HAS NOT RECEIVED SURVEY DATA FOR THE 200-SERIES BORINGS AT THE WRITING OF THIS REPORT. THE 200-SERIES BORINGS ARE LOCATED BASED ON APPROXIMATE FIELD MEASUREMENTS.
2. SEE BORING LOGS FOR DETAILED LITHOLOGIC DESCRIPTIONS.
3. SEISMIC SURVEY WAS COMPLETED BY HAGER-RICHTER ON MAY 4, 2017 AND WAS SUMMARIZED IN A REPORT TO GOLDER ASSOCIATES ON JUNE 7, 2017.

REFERENCES

1. BASEMAP ELEMENTS FROM VHB DRAWING TITLED "3DTopo_13Dec2016.dgn" DATED ON DECEMBER 13, 2016. RECEIVED BY GOLDER ON APRIL 27, 2017.
2. BORINGS WERE LOCATED AND OBSERVED BY GOLDER AND DRILLED BY NEW ENGLAND BORING CONTRACTORS OF HERMON, MAINE FROM MAY 8-11, 2017 (100-SERIES BORINGS) AND SEPTEMBER 25-29, 2017 (200-SERIES BORINGS).
3. SEISMIC SURVEY DATA PROVIDED IN DRAWING TITLED "Figure 2-3.dwg" BY HAGER-RICHTER DATED JUNE 7, 2017.
4. EXISTING STRUCTURES ARE APPROXIMATE AND TAKEN FROM MAINE HIGHWAY COMMISSION, MARSH BRIDGE OVER MARSHALL BROOK, SOUTHWEST HARBOR & TREATMENT, FEBRUARY 1931.
5. LOCATION OF PROPOSED RETAINING WALLS TAKEN FROM VHB DRAWING TITLED "MARSH BRIDGE BRIDGE PLAN AND ELEVATION" DATED AUGUST 17, 2017.
6. ELEVATIONS ARE BASED ON NORTH AMERICAN VERTICAL DATUM (NAVD) OF 1988.

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APPROVED	MCM

PROJECT
MARSH BRIDGE REPLACEMENT
TREMONT AND SOUTHWEST HARBOR, MAINE
MAINEDOT WIN 21702.00

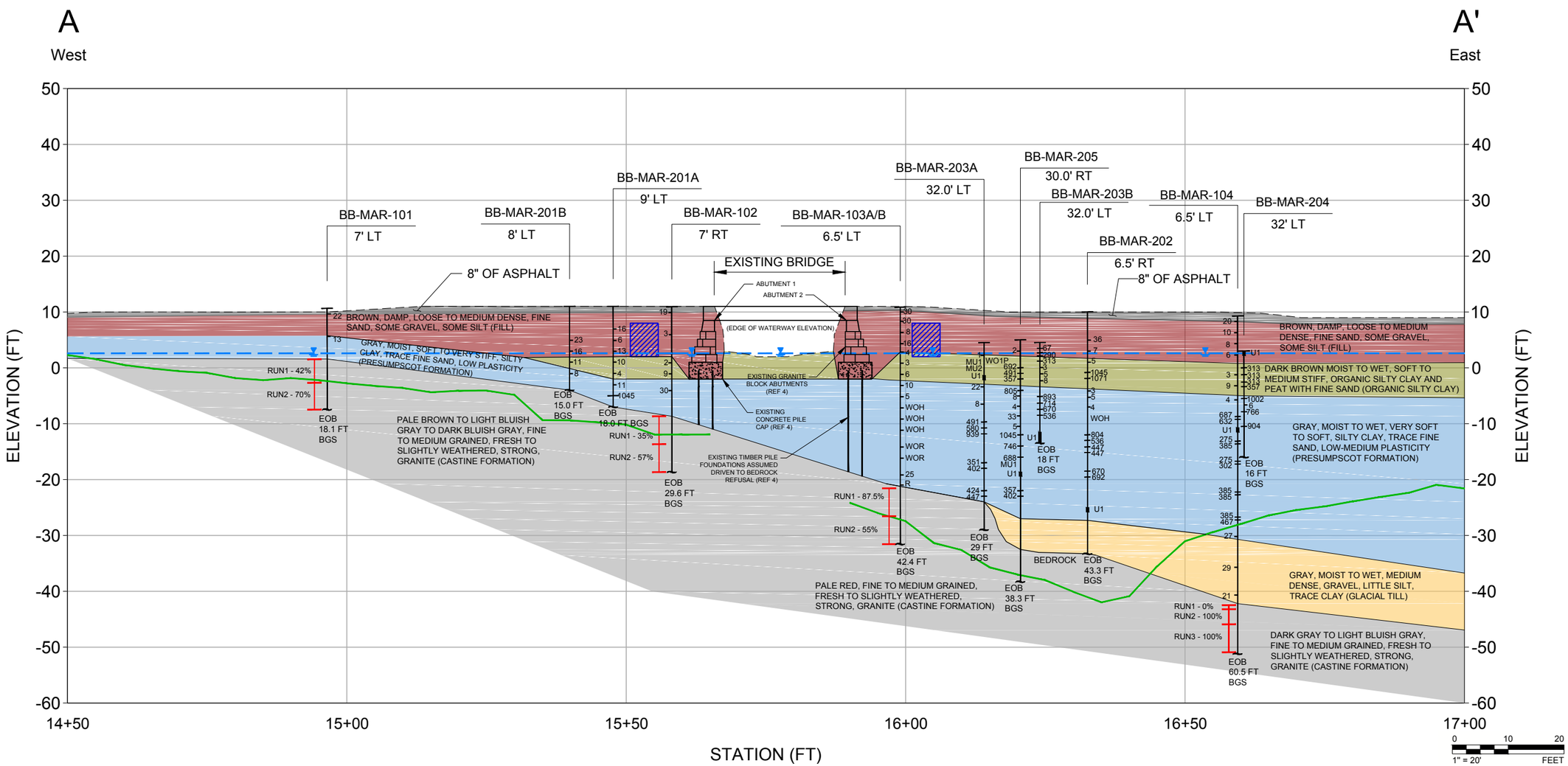
TITLE
BORING LOCATION PLAN

PROJECT NO.	SUBTITLE	REV.	FIGURE
166-8741	A	0	2

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI D

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LEGEND

- BB-MAR-101 / 7FT RT — BORING LOCATION I.D. / OFFSET FROM CENTERLINE
- WOH / WOR / WO1P / 10 / 687 / 632 — SPT: WEIGHT OF HAMMER / SPT: WEIGHT OF RODS / SPT: WEIGHT OF ONE PERSON / SPT: N - VALUE / FIELD VANE SHEAR TEST LOCATION AND PEAK UNDRAINED SHEAR STRENGTH (PSF)
- GROUNDWATER ELEVATION (FT)
- RUN1 - 42% / EOB 18.1 FT BGS — ROCK CORE RUN NUMBER AND RQD / END OF BORING
- ASPHALT
- FILL
- ORGANIC SILTY CLAYS AND PEAT
- PRESUMPSCOT FORMATION
- GLACIAL TILL
- GRANITE, CASTINE FORMATION
- ESTIMATED BEDROCK SURFACE FROM SEISMIC SURVEY HAGER-RICHTER MAY 2017
- ESTIMATED LOCATION OF TIMBER CRIB FROM HAGER-RICHTER SURVEY MAY 2017 & 1931 HISTORICAL DRAWINGS

NOTES

1. THE WATER TABLE SHOWN IN THIS FIGURE IS THE MEAN LOW WATER LEVEL REPORTED BY NORTH STAR HYDRO (REF.5). THE ACTUAL WATER TABLE IS TIDALLY INFLUENCED AND ACTUAL FIELD CONDITIONS WILL VARY.
2. THE 100-SERIES BORING LOCATIONS SHOWN IN THIS FIGURE WERE SURVEYED BY MAINEDOT AND RECEIVED BY GOLDER ON MAY 19, 2017. GOLDER HAS NOT RECEIVED SURVEY DATA FOR THE 200-SERIES BORINGS AT THE WRITING OF THIS REPORT. THE 200-SERIES BORINGS ARE LOCATED BASED ON APPROXIMATE FIELD MEASUREMENTS.
3. SEE BORING LOGS FOR DETAILED LITHOLOGIC DESCRIPTIONS.
4. THIS GENERALIZED SUBSURFACE PROFILE IS INTENDED TO CONVEY TRENDS IN SUBSURFACE CONDITIONS. THE BOUNDARIES BETWEEN STRATA ARE APPROXIMATE AND IDEALIZED, AND HAVE BEEN DEVELOPED BY INTERPRETATIONS OF WIDELY SPACED EXPLORATIONS OF SAMPLES. ACTUAL SOIL AND ROCK TRANSITIONS MAY VARY AND ARE PROBABLY MORE ERRATIC. FOR MORE SPECIFIC INFORMATION REFER TO EXPLORATION LOGS.
5. SEISMIC SURVEY WAS COMPLETED BY HAGER-RICHTER ON MAY 4, 2017 AND WAS SUMMARIZED IN A REPORT TO GOLDER ASSOCIATES ON JUNE 7, 2017.

REFERENCES

1. BASEMAP ELEMENTS FROM VHB DRAWING TITLED "3DTopo_13Dec2016.dgn" DATED ON DECEMBER 13, 2016. RECEIVED BY GOLDER ON APRIL 27, 2017.
2. BORINGS WERE LOCATED AND OBSERVED BY GOLDER AND DRILLED BY NEW ENGLAND BORING CONTRACTORS OF HERMON, MAINE FROM MAY 8-11, 2017 (100-SERIES BORINGS) AND SEPTEMBER 25-29, 2017 (200-SERIES BORINGS).
3. SEISMIC SURVEY DATA PROVIDED IN DRAWING TITLED "Figure 4.dwg" BY HAGER-RICHTER DATED JUNE 7, 2017.
4. EXISTING STRUCTURES ARE APPROXIMATE AND TAKEN FROM MAINE HIGHWAY COMMISSION, MARSH BRIDGE OVER MARSHALL BROOK, SOUTHWEST HARBOR & TREMONT, FEBRUARY 1931.

REFERENCES (CONT.)

5. NORTH STAR HYDRO, "Preliminary Design Hydrology and Hydraulics Report: Southwest Harbor and Tremont - Marsh Bridge #2511," DATED AUGUST 2017.
6. ELEVATIONS ARE BASED ON NORTH AMERICAN VERTICAL DATUM (NAVD) OF 1988.

CLIENT
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PROJECT
MARSH BRIDGE REPLACEMENT
TREMONT AND SOUTHWEST HARBOR, MAINE
MAINEDOT WIN 21702.00

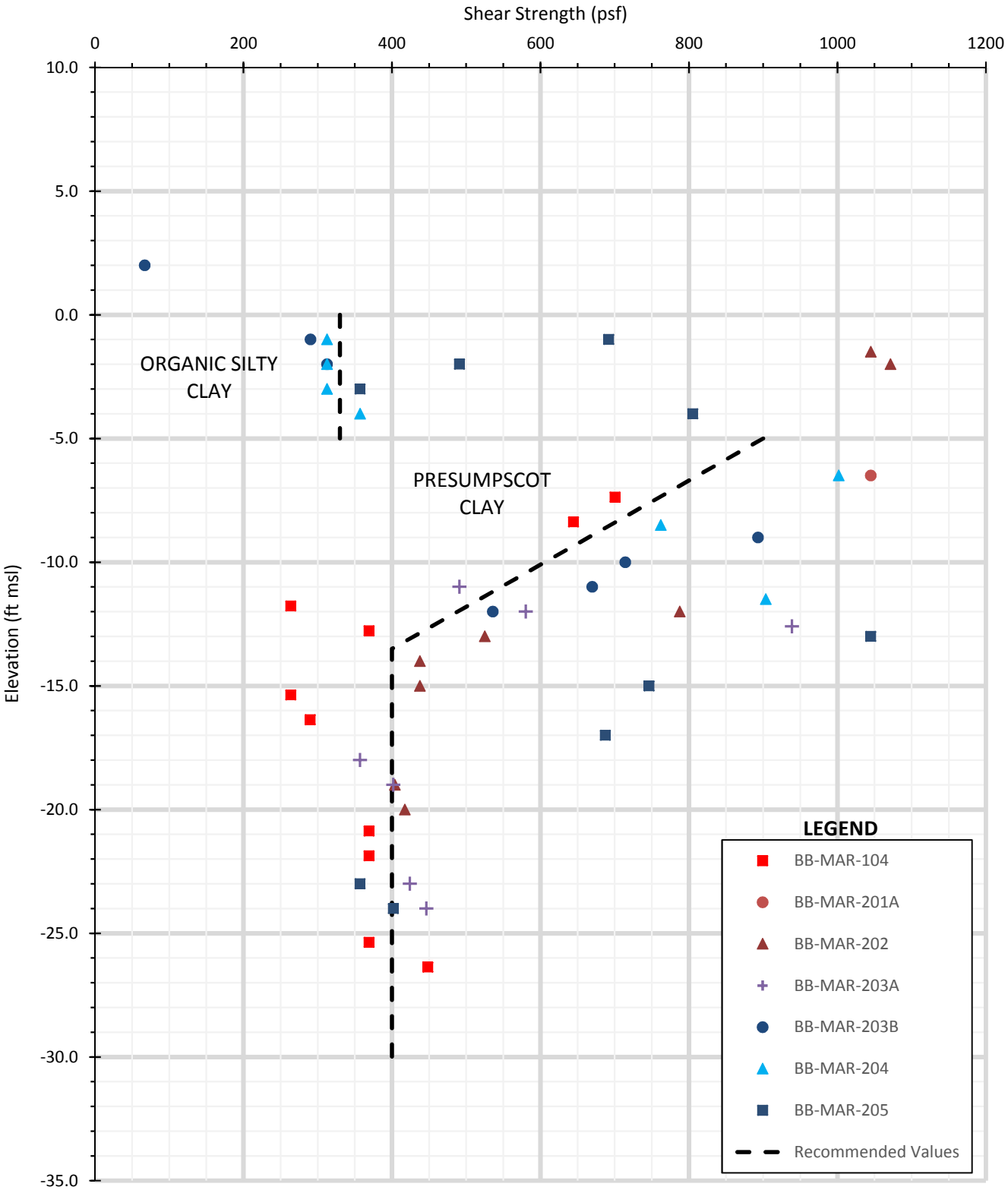
TITLE
INTERPRETED SUBSURFACE PROFILE A-A'

PROJECT NO.	SUBTITLE	REV.	FIGURE
166-8741	A	0	3

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IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI B
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FIGURE 4: In Situ Vane Shear Strength vs. Elevation

Geotechnical Design Report
Marsh Bridge Replacement
Tremont/Southwest Harbor, Maine
MaineDOT WIN: 21702.00



Note: Shear strength data presented in this plot is obtained from vane shear tests completed in the field and has been corrected for plasticity index as shown in Appendix E.

Made By: LLM
Checked By: CJS
Reviewed By: MCM

Figure 4_Strength Plots.xlsx

APPENDIX A
BORING LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE-GRAINED SOILS (more than half of material is larger than No. 4 sieve size)	GRAVELS (more than half of coarse fraction is larger than No. 4 sieve size)	CLEAN GRAVELS (little or no fines)	GW	Well-graded gravels, gravel-sand mixtures, little or no fines.
			GP	Poorly-graded gravels, gravel sand mixtures, little or no fines.
		GRAVEL WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures.
			GC	Clayey gravels, gravel-sand-clay mixtures.
	SANDS (more than half of coarse fraction is smaller than No. 4 sieve size)	CLEAN SANDS (little or no fines)	SW	Well-graded sands, gravelly sands, little or no fines
			SP	Poorly-graded sands, gravelly sand, little or no fines.
		SANDS WITH FINES (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures.
FINE-GRAINED SOILS (more than half of material is smaller than No. 200 sieve size)	SILTS AND CLAYS (liquid limit less than 50)		ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands, or clayey silts with slight plasticity.
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
			OL	Organic silts and organic silty clays of low plasticity.
	SILTS AND CLAYS (liquid limit greater than 50)		MH	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.
			CH	Inorganic clays of high plasticity, fat clays.
			OH	Organic clays of medium to high plasticity, organic silts.
	HIGHLY ORGANIC SOILS		Pt	Peat and other highly organic soils.

Desired Soil Observations (in this order, if applicable):

Color (Munsell color chart)
Moisture (dry, damp, moist, wet)
Density/Consistency (from above right hand side)
Texture (fine, medium, coarse, etc.)
Name (sand, silty sand, clay, etc., including portions - trace, little, etc.)
Gradation (well-graded, poorly-graded, uniform, etc.)
Plasticity (non-plastic, slightly plastic, moderately plastic, highly plastic)
Structure (layering, fractures, cracks, etc.)
Bonding (well, moderately, loosely, etc.,)
Cementation (weak, moderate, or strong)
Geologic Origin (till, marine clay, alluvium, etc.)
Groundwater level

MODIFIED BURMISTER SYSTEM

Descriptive Term	Portion of Total (%)
trace	0 - 10
little	11 - 20
some	21 - 35
adjective (e.g. sandy, clayey)	36 - 50

TERMS DESCRIBING DENSITY/CONSISTENCY

Coarse-grained soils (more than half of material is larger than No. 200 sieve): Includes (1) clean gravels; (2) silty or clayey gravels; and (3) silty, clayey or gravelly sands. Density is rated according to standard penetration resistance (N-value).

Density of Cohesionless Soils	Standard Penetration Resistance N-Value (blows per foot)
Very loose	0 - 4
Loose	5 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	> 50

Fine-grained soils (more than half of material is smaller than No. 200 sieve): Includes (1) inorganic and organic silts and clays; (2) gravelly, sandy or silty clays; and (3) clayey silts. Consistency is rated according to undrained shear strength as indicated.

Consistency of Cohesive soils	SPT N-Value (blows per foot)	Approximate Undrained Shear Strength (psf)	Field Guidelines
Very Soft	WOH, WOR, WOP, <2	0 - 250	Fist easily penetrates
Soft	2 - 4	250 - 500	Thumb easily penetrates
Medium Stiff	5 - 8	500 - 1000	Thumb penetrates with moderate effort
Stiff	9 - 15	1000 - 2000	Indented by thumb with great effort
Very Stiff	16 - 30	2000 - 4000	Indented by thumbnail
Hard	>30	over 4000	Indented by thumbnail with difficulty

Rock Quality Designation (RQD):

RQD (%) = sum of the lengths of intact pieces of core* > 4 inches / length of core advance

*Minimum NQ rock core (1.88 in. OD of core)

Correlation of RQD to Rock Mass Quality

Rock Mass Quality	RQD (%)
Very Poor	≤25
Poor	26 - 50
Fair	51 - 75
Good	76 - 90
Excellent	91 - 100

Desired Rock Observations (in this order, if applicable):

Color (Munsell color chart)
Texture (aphanitic, fine-grained, etc.)
Rock Type (granite, schist, sandstone, etc.)
Hardness (very hard, hard, mod. hard, etc.)
Weathering (fresh, very slight, slight, moderate, mod. severe, severe, etc.)
Geologic discontinuities/jointing:
- -dip (horiz - 0-5 deg., low angle - 5-35 deg., mod. dipping - 35-55 deg., steep - 55-85 deg., vertical - 85-90 deg.)
- -spacing (very close - <2 inch, close - 2-12 inch, mod. close - 1-3 feet, wide - 3-10 feet, very wide >10 feet)
- -tightness (tight, open, or healed)
- -infilling (grain size, color, etc.)

Formation (Waterville, Ellsworth, Cape Elizabeth, etc.)
RQD and correlation to rock mass quality (very poor, poor, etc.)
ref: ASTM D6032 and AASHTO Standard Specification for Highway Bridges, 17th Ed. Table 4.4.8.1.2A
Recovery (inch/inch and percentage)
Rock Core Rate (X.X ft - Y.Y ft (min:sec))

Maine Department of Transportation
Geotechnical Section
Key to Soil and Rock Descriptions and Terms
Field Identification Information

Sample Container Labeling Requirements:

WIN
Bridge Name / Town
Boring Number
Sample Number
Sample Depth
Blow Counts
Sample Recovery
Date
Personnel Initials

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Marsh Bridge
Location: Termont / Southwest Harbour, ME

Boring No.: BB-MAR-101
WIN: 21702.00

Driller:	NEBC	**Elevation (ft.)**	10.63	**Auger ID/OD:**	4"
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	TRM	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140 lbs. / 30"
Date Start/Finish:	05/11/17 8:50 AM/11:00 AM	**Drilling Method:**	SSA / D+W	**Core Barrel:**	NQ
Boring Location:	See Remarks	**Casing ID/OD:**	4"	**Water Level*:**	4.6' bgs at 11:30 5/11/17
Hammer Efficiency Factor:	0.6	**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
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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: 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							SSA			0.9' Asphalt	
								9.73		0.90	
	1D	24/16.8	1.00 - 3.00	9/8/14/12	22	22				1D (1.00'-3.00'): Brown, damp, medium dense, fine to coarse SAND, some silt, some silt, little gravel, little clay 'petrol odor' (FILL)	#412791 WC: 9.8 % A-2-4/SM
2.5											
							▽				
5								5.63		5.00	
	2D	24/12	5.00 - 7.00	7/4/9/6	13	13	30			2D (5.00'-7.00'): Brownish-gray, dry, stiff, silty CLAY, little fine sand, trace gravel, trace clay, trace organics, low plasticity.	#412794 WC: 23.5% A-6(6)/CL
							27				
7.5							56				
							103				
	R1	50.4/37	9.10 - 13.30	RQD = 42%			NQ	1.53		Brownish-gray wash to 9' with fragments of rock, looks like pieces of granite. Grinding/chattering at 9'.	
10										9.10	
										Top of Rock at 9.1'. Run 1: Pale Brown (5YR 5/2) to Light Bluish Gray (5B 7/1), fine to medium grained, GRANITE, strong (R4), fresh (W1), discontinuities dipping 45 to 90 degrees relative to horizontal axis, spacing is extremely close to closely spaced (<20 mm to 190 mm), discontinuity surfaces are planar to curved to irregular, texture is smooth to very rough, some white quartz infilling discontinuities, highly fractured rock from 11.1ft to 12.1 ft [Castin Formation (Devonian-Silurian)] . R1 Core Times (min:sec): 9.1' - 10.1': (2:00) 10.1' - 11.1': (1:45) 11.1 - 12.1: (1:45) 12.1' - 13.1: (2:15) 13.1 - 13.3: (0:30) Recovery: 73%	
12.5											
	R2	57.6/57.6	13.30 - 18.10	RQD = 69%						Run 2: Light Bluish Gray (5B 7/1) to Medium Bluish Gray (5B 5/1), fine to medium grained, GRANITE, strong (R4), fresh to slightly weathered (W1 - W2), discontinuities dipping 20 to 90 degrees relative to horizontal axis, spacing is very close to moderately spaced	
15											

Remarks:

1. Hole open to 16.3' b.g.s. 2. Borehole backfilled with gravelly cuttings, roadside gravel, 25 lbs of gravel, and 20 lbs of coldpatch.
2. Boring is located in the western bridge approach. Refer to Figure 2 for approximate location.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Marsh Bridge
Location: Termont / Southwest Harbour, ME

Boring No.: BB-MAR-101
WIN: 21702.00

Driller:	NEBC	**Elevation (ft.)**	10.63	**Auger ID/OD:**	4"
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	TRM	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140 lbs. / 30"
Date Start/Finish:	05/11/17 8:50 AM/11:00 AM	**Drilling Method:**	SSA / D+W	**Core Barrel:**	NQ
Boring Location:	See Remarks	**Casing ID/OD:**	4"	**Water Level*:**	4.6' bgs at 11:30 5/11/17
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample Information: Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
15										(30 mm to 280 mm), discontinuity surfaces are planar to stepped, texture is smooth to very rough, discontinuities have some calcite and silt infilling, light yellow in color, soft, and possibly gypsum [Castine Formation (Devonian-Silurian)]. R2 Core Times (min:sec): 13.3' - 14.3': (1:45) 14.3' - 15.3': (1:45) 15.3' - 16.3: (2:12) 16.3' - 17.3: (2:08) 17.3' - 18.1: (1:28) Recovery: 100%	
17.5								-7.47		18.10 **Bottom of Exploration at 18.10 feet below ground surface.**	
20											
22.5											
25											
27.5											
30											

Remarks:

1. Hole open to 16.3' b.g.s. 2. Borehole backfilled with gravelly cuttings, roadside gravel, 25 lbs of gravel, and 20 lbs of coldpatch.
2. Boring is located in the western bridge approach. Refer to Figure 2 for approximate location.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Marsh Bridge
Location: Termont / Southwest Harbour, ME

Boring No.: BB-MAR-102
WIN: 21702.00

Driller:	NEBC	**Elevation (ft.)**	10.94	**Auger ID/OD:**	4"
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	TRM	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140 lbs. / 30"
Date Start/Finish:	15/11/17 11:50 AM/4:00 PM	**Drilling Method:**	SSA / D+W	**Core Barrel:**	NQ
Boring Location:	See Remarks	**Casing ID/OD:**	4"	**Water Level*:**	6.8' bgs at 16:20 5/11/17
Hammer Efficiency Factor: 0.6		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA			0.95' Asphalt.	
								9.99		0.95	
	1D	24/14.4	1.00 - 3.00	14/10/9/8	19	19				1D (1.00'-3.00'): Brown, damp, medium dense, fine to coarse SAND, some gravel, little silt (FILL).	#412792 WC: 8.0% A-1-b(0)/SM
2.5											
5								5.94		5.00	
	2D	24/12	5.00 - 7.00	3/1/2/12	3	3	9			2D (5.00'-7.00'): Brown-gray with brown-red staining, moist, soft, silty CLAY, some fine sand, trace gravel, low to medium plasticity. PP: 2.25 TSF	#412782 WC: 27.7% A-6/CL
							50				
7.5							130				
							142			Hard drilling, chattering/grinding from 8' to 9'.	
							22				
10											
	MD1	24/0	10.00 - 12.00	3/1/1/2	2	2	30			Pushed a rock/gravel.	
	MV1			Would Not Push			21			Failed 65x130mm vane. Gravel obstruction.	
								-1.06		12.00	
12.5	3D	24/21.6	12.00 - 14.00	3/4/5/7	9	9	30			3D (12.00'-14.00'): 3" Spoon: Gray, damp, stiff, silty CLAY, little fine sand, trace organics, increasing sand content with depth, low plasticity. PP: 1.5,1.5,1.5 TSF; TV: 0.368 TSF (TOP), 0.24 TSF (BOTTOM).	#412795 WC: 19.9 A-6/CL
							35				
	MV2			Would Not Push			40			Failed 65x130mm vane. Gravel obstruction.	
15											

Remarks:

1. Borehole backfilled with cuttings, roadside gravel, 0.5 bag of gravel, 15 lbs. cold patch.
2. Boring is located near the western bridge abutment. Refer to Figure 2 for approximate location.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Marsh Bridge
Location: Termont / Southwest Harbour, ME

Boring No.: BB-MAR-102
WIN: 21702.00

Driller:	NEBC	**Elevation (ft.)**	10.94	**Auger ID/OD:**	4"
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	TRM	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140 lbs. / 30"
Date Start/Finish:	15/11/17 11:50 AM/4:00 PM	**Drilling Method:**	SSA / D+W	**Core Barrel:**	NQ
Boring Location:	See Remarks	**Casing ID/OD:**	4"	**Water Level*:**	6.8' bgs at 16:20 5/11/17
Hammer Efficiency Factor: 0.6		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
15	4D	24/8.4	15.00 - 17.00	14/16/14/17	30	30	32			4D (15.00'-17.00'): 3" Spoon: Gray with dark staining, damp, hard, CLAY, pushed a piece of gravel, low to medium plasticity; sample sheared in half vertically. PP: 2.25, 1.0 TSF.	#412784 WC: 26.1% A-6/CL
	MV3			Would Not Push			28			Failed 65x130mm vane. Vane pushed 6" before refusal.	
17.5							20				
							17/6"				
	R1	60/52	19.60 - 24.60	RQD = 35%			NQ	-8.66		19.60 Run 1: Light Bluish Gray (5B 7/1), fine to medium grained, GRANITE, strong (R4), fresh to slightly weathered (W1 - W2), discontinuities dipping at 20 to 90 degrees relative to horizontal axis, spacing is extremely close to closely spaced (<20 mm to 160 mm), discontinuity surfaces are planar to irregular to stepped, texture is smooth to very rough, some soft red material, possibly silt, infilling joints [Castine Formation (Devonian-Silurian)]. Core Times: (min:sec): 19.6' - 20.6': (2:42) 20.6' - 21.6': (1:40) 21.6' - 22.6': (1:25) 22.6' - 23.6': (1:43) 23.6' - 24.6': (2:10) Recovery: 87%	
20											
22.5											
	R2	60/58	24.60 - 29.60	RQD = 52%						Run 2: Medium Dark Gray (N4) to Moderate Red (5R 5/4), fine to medium grained GRANITE, strong (R4), fresh to slightly weathered (W1 - W2), discontinuities dipping at 45 to 90 degrees relative to horizontal axis, spacing is very close to moderately spaced (30 mm to 300 mm) discontinuity surfaces are planar to curved to stepped, texture is smooth to rough, no infilling material was observed [Castine Formation (Devonian-Silurian)]. Core Times: (min:sec): 24.6' - 25.6': (2:12) 25.6' - 26.6': (1:40) 26.6' - 27.6': (1:40) 27.6' - 28.6': (2:25) 28.6' - 29.6': (2:03) Recovery: 97%	
25											
27.5											
30								-18.66		29.60 **Bottom of Exploration at 29.60 feet below ground surface.**	

Remarks:

1. Borehole backfilled with cuttings, roadside gravel, 0.5 bag of gravel, 15 lbs. cold patch.
2. Boring is located near the western bridge abutment. Refer to Figure 2 for approximate location.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Marsh Bridge
Location: Termont / Southwest Harbour, ME

Boring No.: BB-MAR-103 A
WIN: 21702.00

Driller:	NEBC	**Elevation (ft.)**	10.81	**Auger ID/OD:**	4"
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	TRM	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140 lbs. / 30"
Date Start/Finish:	9:10 AM 05/08/17	**Drilling Method:**	SSA	**Core Barrel:**	N/A
Boring Location:	See Remarks	**Casing ID/OD:**	4"	**Water Level*:**	Dry
Hammer Efficiency Factor: 0.6		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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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: 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	1D	24/14.4	0.50 - 2.50	35/14/16/20	30	30	SSA	10.31		0.5' Asphalt (0.50)	
										1D (0.50'-2.50'): Top 0.8': Black, damp, medium dense, fine to coarse SAND, little gravel, petrol odor (FILL) (1B). Bottom 0.4': Brown, damp, medium dense to dense, fine to coarse SAND, little gravel, petrol odor (Fill) (1A).	
2.5	2D	24/6	2.50 - 4.50	20/14/16/13	30	30				2D (2.50'-4.50'): Brown, damp, medium dense to dense, fine to coarse SAND, some silt, little gravel (FILL).	#412793 WC: 12.2% A-2-4/SM
	3D	24/8.4	4.50 - 6.50	9/4/4/3	8	8				3D (4.50'-6.50'): Brown, wet, loose, fine to coarse silty SAND, some gravel (FILL) .	
5							13				
	4D	22/6	6.50 - 8.33	2/4/12/(50/4")	16	16	14			4D (6.50'-8.50'): Brown, wet, medium dense, silty fine to coarse SAND, some gravel, trace organics (wood pieces) (FILL).	
7.5							16				
								2.48		Refusal on suspected timber cribbing. (8.33)	
										Bottom of Exploration at 8.33 feet below ground surface.	
10											
12.5											
15											

Remarks:

1. Started with safety winch drive hammer. At 103B started with rope and cathead, drive and wash.
2. Auger (SSA) used for first 5'.
3. Boring is located near the eastern bridge abutment. Refer to Figure 2 for approximate location.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Marsh Bridge
Location: Termont / Southwest Harbour, ME

Boring No.: BB-MAR-103 B
WIN: 21702.00

Driller:	NEBC	**Elevation (ft.)**	10.88	**Auger ID/OD:**	4"
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	TRM	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140 lbs. / 30"
Date Start/Finish:	5/8/17 (9:10AM) / 5/9/17 (11:30PM)	**Drilling Method:**	SSA / D+W	**Core Barrel:**	NQ
Boring Location:		**Casing ID/OD:**	4"	**Water Level*:**	5.3' bgs at 9:20 5/9/17
Hammer Efficiency Factor: 0.6		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	10.38		Asphalt.	
										0.50	
										Wash ahead to bottom of boring BB-MAR-103A.	
2.5											
5							15				
							14				
7.5							16			Drill grinding, brown wash.	
								2.88		8.00	
	5D	24/18	8.00 - 10.00	5/3/1/3	4	4	9			5D (8.00'-10.00'): Alternating layers of gray, moist, soft, organic silty CLAY; brown, moist, very loose, silty SAND; moist, soft, ORGANIC SILT.	#412796 WC: 62.0% A-6(9)/OL
							7				
10	6D	24/14.4	10.00 - 12.00	1/1/2/1	3	3	8			6D (10.00'-12.00'): Upper 0.8': Gray, wet, soft, organic silty CLAY, little fine sand, trace gravel, strong odor. Lower 0.4': Dark brown, damp, soft, organic silty CLAY, strong odor, low plasticity .	
							7				
12.5	7D	24/20.4	12.00 - 14.00	WOH/3/3/2	6	6	12			7D (12.00'-14.00'): Upper 0.9': Dark brown, moist, medium stiff, PEAT. Lower 0.8': Gray, moist, medium stiff, clayey SAND, trace silt, trace organics, higher plasticity than above.	
								-2.12		13.00	
							12				
15	8D	24/4.8	14.00 - 16.00	4/4/6/6	10	10	21			8D (14.00'-16.00'): Gray, moist, stiff, sandy CLAY, some gravel, trace silt, trace organics, low plasticity. TV: 0.14 TSF	

Remarks:

1. Started with rope and cathead, drive and wash.
2. Water levels: 7.9' b.g.s. at end of day 5/8/17; 5.3' b.g.s. at 9:20 AM 5/9/17; 8.8' b.g.s. at 12:00 5/9/17.
3. Hole collapsed back to 33.3' when casing removed.
4. Borehole backfilled with cuttings, 4 bags of gravel, 1.5 bags of sand, and 0.5 bags of coldpatch.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Marsh Bridge
Location: Termont / Southwest Harbour, ME

Boring No.: BB-MAR-103 B
WIN: 21702.00

Driller:	NEBC	**Elevation (ft.)**	10.88	**Auger ID/OD:**	4"
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	TRM	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140 lbs. / 30"
Date Start/Finish:	5/8/17 (9:10AM) / 5/9/17 (11:30PM)	**Drilling Method:**	SSA / D+W	**Core Barrel:**	NQ
Boring Location:		**Casing ID/OD:**	4"	**Water Level*:**	5.3' bgs at 9:20 5/9/17
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
15							19				
	9D	24/18	16.00 - 18.00	4/2/3/2	5	5	23			9D (16.00'-18.00'): Gray, moist, medium stiff, silty CLAY, trace fine sand, medium plasticity, large piece of gravel. PP: 1.75 TSF; TV: 0.34 TSF	
17.5	MV1			Would Not Push			16				
	10D	24/24	18.00 - 20.00	WOH			9			Sand seam at 17.9' Obstruction at 18', failed 65 x 130 vane test. 10D (18.00'-19.00'): Upper 1.5': Gray, moist, very soft, silty CLAY, some sand; black seam in lower 0.5', low plasticity. PP: 0 Drill rig grinding, wash turned black.	#412786 WC: 25.4 % A-4/CL
							8				
20	11D	24/12	20.00 - 22.00	1/WOH			17			11D (20.00'-22.00'): Gray, moist, very soft, silty CLAY, low plasticity. PP: 0; TV: 0.1 TSF	WC: 37.2%
							13				
22.5	12D	24/24	22.00 - 24.00	WOH			12			12D (22.00'-24.00'): Gray with dark staining, wet, very soft, silty CLAY, too wet to roll, low to medium plasticity. PP: 0; TV: 0.05 TSF	#412787 WC: 41.7% A-7-6/CL
							10				
							9				
25	13D MV2	24/24	25.00 - 27.00	WOR Would Not Push			15			13D (25.00'-27.00'): Gray with dark staining, wet, very soft, silty CLAY, too wet to roll, low to medium plasticity. PP: 0; TV: 0.05 TFS Failed 65x130 vane. Felt like gravel obstructing; soft clay stuck on vane.	
							12				
27.5	14D	24/24	27.00 - 29.00	WOR			9			14D (27.00'-29.00'): Gray with dark staining, wet, very soft, silty CLAY, too wet to roll, low to medium plasticity. PP: 0; TV: 0.05 TSF Gritty wash.	
							7				
30											

Remarks:

1. Started with rope and cathead, drive and wash.
2. Water levels: 7.9' b.g.s. at end of day 5/8/17; 5.3' b.g.s. at 9:20 AM 5/9/17; 8.8' b.g.s. at 12:00 5/9/17.
3. Hole collapsed back to 33.3' when casing removed.
4. Borehole backfilled with cuttings, 4 bags of gravel, 1.5 bags of sand, and 0.5 bags of coldpatch.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Marsh Bridge
Location: Termont / Southwest Harbour, ME

Boring No.: BB-MAR-103 B
WIN: 21702.00

Driller:	NEBC	**Elevation (ft.)**	10.88	**Auger ID/OD:**	4"
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	TRM	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140 lbs. / 30"
Date Start/Finish:	5/8/17 (9:10AM) / 5/9/17 (11:30PM)	**Drilling Method:**	SSA / D+W	**Core Barrel:**	NQ
Boring Location:		**Casing ID/OD:**	4"	**Water Level*:**	5.3' bgs at 9:20 5/9/17
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
30	15D	24/2.4	30.00 - 32.00	WOR/14/11/8	25	25	22			15D (30.00'-32.00'): Gray with dark staining, wet, very stiff, silty CLAY. Rocks and gravel in wash from 30' to 32'.	
							21				
								-21.12		32.00	
32.5	16D R1	60/55.8	32.00 - 32.33 32.40 - 37.40	RQD = 87.5%			NQ	-21.52		16D (32.00'-33.00'): 3" Spoon: Gray, wet, very dense, silty SAND, some clay, some black sub-angular gravel with rock fragments. 32.40	
										Run 1: Pale Red (5R 6/2), medium to fine grained, GRANITE, strong (R4), fresh to slightly weathered (W1 - W2), discontinuities dipping at 10 to 90 degrees relative to core axis, spacing is extremely close to moderately spaced (20 mm to 400 mm), discontinuity surfaces are planar to curved to irregular, texture is smooth to very rough, insipient joints have some mineral growth, green in color, possibly chlorite [Castine Formation (Devonian-Silurian)]. R1 Core Times (min:sec): 32.4' - 33.4': (4:12) 33.4' - 34.4': (2:00) 34.4' - 35.4': (1:45) 35.4' - 36.4': (1:42) 36.4' - 37.4': (1:50) Recovery: 93%	
35											
37.5	R2	60/60	37.40 - 42.40	RQD = 55%						Run 2: Pale Red (5R 6/2), medium to fine grained, GRANITE, strong (R4), slightly weathered to fresh (W1 - W2), discontinuities dipping at 10 to 90 degrees relative to core axis, spacing is very closely spaced to moderately spaced (20 mm to 400 mm), discontinuity surfaces are planar to curved to irregular, texture is smooth to very rough, insipient joints have some mineral growth, green in color, possibly chlorite [Castine Formation (Devonian-Silurian)]. R2 Core Times (min:sec): 37.4' - 38.4': (1:24) 38.4' - 39.4': (1:45) 39.4' - 40.4': (1:40) 40.4' - 41.4': (1:40) 41.4' - 42.4': (1:15) Recovery: 100%	
40											
42.5								-31.52		42.40 **Bottom of Exploration at 42.40 feet below ground surface.**	
45											

Remarks:

1. Started with rope and cathead, drive and wash.
2. Water levels: 7.9' b.g.s. at end of day 5/8/17; 5.3' b.g.s. at 9:20 AM 5/9/17; 8.8' b.g.s. at 12:00 5/9/17.
3. Hole collapsed back to 33.3' when casing removed.
4. Borehole backfilled with cuttings, 4 bags of gravel, 1.5 bags of sand, and 0.5 bags of coldpatch.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Marsh Bridge
Location: Termont / Southwest Harbour, ME

Boring No.: BB-MAR-104
WIN: 21702.00

Driller:	NEBC	**Elevation (ft.)**	9.29	**Auger ID/OD:**	4"
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	TRM	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140 lbs. / 30"
Date Start/Finish:	5/9/17 (12:40) / 5/10/17 (4:15 PM)	**Drilling Method:**	SSA / D+W	**Core Barrel:**	NQ
Boring Location:	East Approach	**Casing ID/OD:**	4"	**Water Level*:**	4.6' at 10:00 5/10/17
Hammer Efficiency Factor: 0.6		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA			0.75' Asphalt	
								8.54		0.75	
	1D	24/13.2	1.00 - 3.00	8/11/9/9	20	20				1D (1.00'-3.00'): Brown, damp, medium dense, fine to coarse SAND, some gravel, some silt (FILL).	
2.5											
	2D	24/13.2	3.00 - 5.00	8/7/3/5	10	10				2D (3.00'-5.00'): Brown, moist, loose, fine to coarse SAND, some gravel, some clay, little silt (FILL).	
							▽				
5	3D	24/8.4	5.00 - 7.00	4/6/2/1	8	8	20			3D (5.00'-7.00'): Top 0.2': Gray, moist, medium stiff, clayey fine SAND, little gravel, trace silt. (3A) Bottom 0.5': Brown/gray, wet, loose, fine to coarse SAND, some gravel. (3B) (FILL)	
							8				
7.5	4D	24/24	7.00 - 9.00	1/2/4/11	6	6	13			4D (7.00'-9.00'): Top 1.5': Gray, moist, loose, silty fine to coarse SAND, trace clay. (4A) Bottom 0.5': Gray, damp, medium stiff, organic silty CLAY, some fine sand, some organics (fibrous material, wood chunks) (4B) (FILL). Casing dropped 0.5' during wash from 5' to 10'. wash turned gray at about 7', large gravel or rock fragments in wash.	
							32	0.79		8.50	
							22				
10	5D	24/18	10.50 - 12.50	1/1/2/1	3	3	9			5D (10.50'-12.50'): Dark brown, saturated, soft, PEAT, strong odor.	#412797 WC: 271.0 % A-4/OL
							8				
12.5	6D	24/24	12.50 - 14.50	5/4/5/4	9	9	9			6D (12.50'-14.50'): Top 1.5': Dark brown-gray, damp, stiff, silty fine SAND, little clay, trace organics. (6A) Bottom 0.5': Gray, damp, stiff, silty CLAY, trace fine sand. (6B) PP: 1.0, 1.25, 1.0 TSF; TV: 0.3, 0.34 TSF	#412798 WC: 28.0% A-6/CL
							14	-3.71		13.00	
15							13				

Remarks:

1. Blow counts for spoons driven through a disturbed field vane test location cannot be used to determine N-value.
2. Water Levels: 4.6' b.g.s. at 5:15 PM 5/9/17; 4.6' b.g.s. at 10:00 AM 5/10/17; 2.4' b.g.s. at 4:30 PM 5/10/17.
3. Boring is located in the eastern bridge approach. Refer to Figure 2 for approximate location.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Marsh Bridge
Location: Termont / Southwest Harbour, ME

Boring No.: BB-MAR-104
WIN: 21702.00

Driller:	NEBC	**Elevation (ft.)**	9.29	**Auger ID/OD:**	4"
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	TRM	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140 lbs. / 30"
Date Start/Finish:	5/9/17 (12:40) / 5/10/17 (4:15 PM)	**Drilling Method:**	SSA / D+W	**Core Barrel:**	NQ
Boring Location:	East Approach	**Casing ID/OD:**	4"	**Water Level*:**	4.6' at 10:00 5/10/17
Hammer Efficiency Factor:	0.6	**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
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R = Rock Core Sample
SSA = Solid Stem Auger
HSA = Hollow Stem Auger
RC = Roller Cone
WOH = Weight of 140lb. Hammer
WOR/C = Weight of Rods or Casing
WO1P = Weight of One Person
S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
15	7D MV1	24/24	15.00 - 17.00	4/2/2/2 Would Not Push	4	4	26			7D (15.00'-17.00'): 3" Spoon: Gray, moist, soft, silty CLAY, trace organics, 0.5" woody/fibrous layer in 3" spoon at 16.4' depth, 2" & 3" gravel pieces, suspected to be from layers above. PP: 0; TV: 0.2, 0.1 TSF Failed vane due to gravel obstruction.	WC: 35.5%
							9				
17.5	8D V2	24/19.2	17.00 - 19.00 17.60 - 18.00	WOR Su= 687/82 psf	Note 1		9			8D (17.00'-19.00'): Gray, wet, very soft, silty CLAY, little fine sand; too wet to roll, low to medium plasticity. PP:0; TV:0.04 TSF V2: 25 / 3 ft.-lbs.	#412789 WC: 35.9% A-6/CL
	V3		18.60 - 19.00	Su= 632/55 psf			8			V3: 23 / 2 ft.-lbs.	
							8				
20	1U	24/16.8	20.00 - 22.00	Shelby Tube							
22.5	V4		22.00 - 22.40	Su= 275/55 psf			11			V4: 10 / 2 ft.-lbs.	
	V5		23.00 - 23.40	Su= 385/55 psf			10			V5: 14 / 2 ft.-lbs.	
							8				
25	9D V6	24/24	25.00 - 27.00 25.60 - 26.00	WOR Su= 275/55 psf	Note 1		10			9D (25.00'-27.00'): Gray, damp, very soft, CLAY, trace silt, too wet to roll, low to medium plasticity. PP: 0 V6: 10 / 2 ft.-lbs.	#412799 WC: 43.6% A-7-6/CL
	V7		26.60 - 27.00	Su= 302/55 psf			8			V7: 11 / 2 ft.-lbs.	
27.5							8				
							8				
30							9				

Remarks:

1. Blow counts for spoons driven through a disturbed field vane test location cannot be used to determine N-value.
2. Water Levels: 4.6' b.g.s. at 5:15 PM 5/9/17; 4.6' b.g.s. at 10:00 AM 5/10/17; 2.4' b.g.s. at 4:30 PM 5/10/17.
3. Boring is located in the eastern bridge approach. Refer to Figure 2 for approximate location.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Marsh Bridge
Location: Termont / Southwest Harbour, ME

Boring No.: BB-MAR-104
WIN: 21702.00

Driller:	NEBC	**Elevation (ft.)**	9.29	**Auger ID/OD:**	4"
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	TRM	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140 lbs. / 30"
Date Start/Finish:	5/9/17 (12:40) / 5/10/17 (4:15 PM)	**Drilling Method:**	SSA / D+W	**Core Barrel:**	NQ
Boring Location:	East Approach	**Casing ID/OD:**	4"	**Water Level*:**	4.6' at 10:00 5/10/17
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
30	10D	24/24	30.50 - 32.50	WOR	Note 1		13			Gray wash. Rod dropped to 30.5'. 10D (30.50'-32.50'): Gray with some brown staining, wet, very soft, CLAY, trace silt, too wet to roll, low to medium plasticity. PP: 0 V8: 14 / 3 ft.-lbs.	
	V8			Su= 385/82 psf			10				
32.5	V9			Su= 385/82 psf			10			V9: 14 / 3 ft.-lbs.	
							11				
							11				
35	11D V10	24/24	35.00 - 37.00	WOR Su= 385/82 psf	Note 2		15			11D (35.00'-37.00'): Gray with some black staining, wet, very soft, CLAY, trace silt, too wet to roll, low to medium plasticity. PP: 0 Gray wash V9: 14 / 3 ft.-lbs.	WC: 40.9%
	V11			Su= 467/82 psf			10			V9: 17 / 3 ft.-lbs.	
37.5							9				
							7				
							5				
40								-30.71		40.00	
	MD1 MV12	0/0	40.00 - 40.00	7/12/15/11	27	27	8			Obstruction preventing sample. Failed 65 x 130 mm vane.	
							8				
42.5							6				
							64			Chattering/ grinding at 43.8.	
45							89				

Remarks:

1. Blow counts for spoons driven through a disturbed field vane test location cannot be used to determine N-value.
2. Water Levels: 4.6' b.g.s. at 5:15 PM 5/9/17; 4.6' b.g.s. at 10:00 AM 5/10/17; 2.4' b.g.s. at 4:30 PM 5/10/17.
3. Boring is located in the eastern bridge approach. Refer to Figure 2 for approximate location.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Marsh Bridge
Location: Termont / Southwest Harbour, ME

Boring No.: BB-MAR-104
WIN: 21702.00

Driller:	NEBC	**Elevation (ft.)**	9.29	**Auger ID/OD:**	4"
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	TRM	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140 lbs. / 30"
Date Start/Finish:	5/9/17 (12:40) / 5/10/17 (4:15 PM)	**Drilling Method:**	SSA / D+W	**Core Barrel:**	NQ
Boring Location:	East Approach	**Casing ID/OD:**	4"	**Water Level*:**	4.6' at 10:00 5/10/17
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
45	12D	24/6	45.00 - 47.00	24/18/11/10	29	29	42			12D (45.00'-47.00'): Gravel/rock fragments with little silt, trace clay, trace fine sand (TILL). Grinding/chattering, rock fragments in wash to 45'.	
							62				
47.5							98				
							93				
							87				
50	13D	21/3.6	50.00 - 51.75	26/12/9/(34/3")	21	21	73			13D (50.00'-51.75'): Gravel/rock fragments, little gray clay, trace fine sand, trace silt, fragments of pink gravel, 2.0" in size (TILL). Wash to 50' gray with gravel & rock fragments. Grinding/chattering past 50', wash turned clear.	
	R1	9/6	51.50 - 52.25	RQD = 0%			97/6" NQ	-42.21		51.50 Run 1: Dark Gray (N3) to Light Bluish Gray (5B 7/1), fine to medium grained, GRANITE, strong (R4), fresh to slightly weathered (W1 - W2), discontinuities are nearly horizontal to vertical, spacing is extremely close to closely spaced (<20 mm to 90 mm), discontinuity surfaces are planar to curved to stepped, texture is smooth to very rough, very fractured, top 0.2' is rock fragments, [Castine Formation (Devonian-Silurian)]. R1 Core Times (min:sec): 51.75' - 52.50': (1:50) Recovery: 67% Not smooth coring, drill rod rattled 4" casing too much. Brought up bit and added 3" casing up to surface.	
52.5	R2	32.4/32.4	52.50 - 55.20	RQD = 28%						Run 2: Dark Gray (N3) to Light Bluish Gray (5B 7/1) fine to medium grained, GRANITE, strong (R4), fresh to slightly weathered (W1 - W2), discontinuities are nearly horizontal to vertical, spacing is extremely close to closely spaced (<20 mm to 130 mm), surfaces are planar to curved to stepped, texture is smooth to very rough [Castine Formation (Devonian-Silurian)]. R2 Core Times (min:sec): 52.5' - 53.5': (2:20) 53.5' - 54.5': (1:20) 54.5' - 55.2': (1:43) Recovery: 100%	
55	R3	60/60	55.20 - 60.20	RQD = 45%						Run 3: Dark Gray (N3) to Light Bluish Gray (5B 7/1), fine to medium grained, GRANITE, strong (R4), fresh to slightly weathered (W1 - W2), discontinuities are nearly horizontal to vertical, spacing is extremely close to closely spaced (<20 mm to 200 mm), surfaces are planar to curved to stepped, texture is smooth to very rough [Castine Formation (Devonian-Silurian)]. R3 Core Times (min:sec): 55.2' - 56.2': (1:38) 56.2' - 57.2': (2:00)	
57.5											
60											

Remarks:

1. Blow counts for spoons driven through a disturbed field vane test location cannot be used to determine N-value.
2. Water Levels: 4.6' b.g.s. at 5:15 PM 5/9/17; 4.6' b.g.s. at 10:00 AM 5/10/17; 2.4' b.g.s. at 4:30 PM 5/10/17.
3. Boring is located in the eastern bridge approach. Refer to Figure 2 for approximate location.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Marsh Bridge
Location: Termont / Southwest Harbour, ME

Boring No.: BB-MAR-104
WIN: 21702.00

Driller:	NEBC	**Elevation (ft.)**	9.29	**Auger ID/OD:**	4"
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	TRM	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140 lbs. / 30"
Date Start/Finish:	5/9/17 (12:40) / 5/10/17 (4:15 PM)	**Drilling Method:**	SSA / D+W	**Core Barrel:**	NQ
Boring Location:	East Approach	**Casing ID/OD:**	4"	**Water Level*:**	4.6' at 10:00 5/10/17
Hammer Efficiency Factor: 0.6		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

Definitions:
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U = Thin Wall Tube Sample
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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: 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.
60								-50.91		57.2' - 58.2': (2:43) 58.2' - 59.2': (1:48) 59.2' - 60.2': (2:00) 100% Recovery 60.20 **Bottom of Exploration at 60.20 feet below ground surface.**	
62.5											
65											
67.5											
70											
72.5											
75											

Remarks:

1. Blow counts for spoons driven through a disturbed field vane test location cannot be used to determine N-value.
2. Water Levels: 4.6' b.g.s. at 5:15 PM 5/9/17; 4.6' b.g.s. at 10:00 AM 5/10/17; 2.4' b.g.s. at 4:30 PM 5/10/17.
3. Boring is located in the eastern bridge approach. Refer to Figure 2 for approximate location.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Marsh Bridge Supplemental Borings
Location: Tremomt / Southwest Harbor, ME

Boring No.: BB-MAR-201A
WIN: 21702.00

Driller:	NEBC	**Elevation (ft.)**	10 (See remarks)	**Auger ID/OD:**	4"
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon / 3"
Logged By:	LLM	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140 lbs./ 30"
Date Start/Finish:	7:55 / 10:55 (9/28/17)	**Drilling Method:**	SSA / D+W	**Core Barrel:**	N/A
Boring Location:	See Remarks	**Casing ID/OD:**	4"	**Water Level*:**	1.8' at 10:45
Hammer Efficiency Factor:	0.885	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	9.55		0.45' Asphalt	
								9.00		0.45 0.55' Reclaimed asphalt 1.00	
2.5											
	1D	24/15.6	4.00 - 6.00	16/11/5/8	16	24	46			1D (4.00' - 6.00'): Top 0.2' is light gray powder, augured through a rock. The rest was brown, damp, medium dense, fine to coarse SAND, little gravel, little silt (FILL).	
5							5				
	2D	24/7.2	6.00 - 8.00	4/2/4/3	6	9	19			2D (6.00' - 8.00'): Top 0.45' is brown, wet, medium stiff, sandy SILT, little gravel, little clay. Bottom 0.15' is wood chunks (FILL).	
								3.00		7.00	
7.5							28				
	3D	24/3.6	8.00 - 10.00	9/9/4/3	13	19	55			3D (8.00' - 10.00'): Chunks of wood.	
							26				
10								0.00		10.00	
	4D	24/16.8	10.00 - 12.00	9/5/5/9	10	15	19			3" Spoon (10.00' - 12.00'): 4DA (Top 0.9'): Dark brown, damp, stiff, ORGANIC SILT and pieces of wood, trace gravel.	
							48	-1.00		11.00 4DB (Bottom 0.5'): Gray, damp, stiff, fine sandy SILT, some clay.	
12.5	5D MV1	24/19.2	12.00 - 14.00	3/2/2/3 Would Not Push	4	6	34			5D (12.00' - 14.00'): Top 0.5' is bits of wood. Bottom 1.1' is gray, damp, soft, clayey SILT, some fine sand, trace gravel. More sandy at the top of the sample and more clayey towards the bottom. MV1 (12'): Failed 55 x 110 mm vane.	
							38				
15	6D MV2	24/4.8	14.00 - 16.00	3/5/6/8 Would Not Push	11	16	OPEN			6D (14.00' - 16.00'): Gray, wet, stiff, silty CLAY, trace gravel, some wood bits, probably being pushed down from above. MV2 (14'): Failed 55 x 110 mm vane.	

Remarks:

1. Blow counts for spoons driven through a disturbed field vane test location cannot be used to determine N-value.
2. Boring collapsed back to 6.5' after casing removed.
3. No water in boring after it collapsed to 6.5'.
4. Boring is located in the western bridge approach. Refer to Figure 2 for approximate location.
5. Boring elevation estimated from basemap provided by VHB on April 27, 2017.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Marsh Bridge Supplemental Borings
Location: Tremomt / Southwest Harbor, ME

Boring No.: BB-MAR-201A
WIN: 21702.00

Driller:	NEBC	**Elevation (ft.)**	10 (See remarks)	**Auger ID/OD:**	4"
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon / 3"
Logged By:	LLM	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140 lbs./ 30"
Date Start/Finish:	7:55 / 10:55 (9/28/17)	**Drilling Method:**	SSA / D+W	**Core Barrel:**	N/A
Boring Location:	See Remarks	**Casing ID/OD:**	4"	**Water Level*:**	1.8' at 10:45
Hammer Efficiency Factor:	0.885	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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SSA = Solid Stem Auger
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WOH = Weight of 140lb. Hammer
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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: 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.
15											
	7D V1	15.6/15.6	16.00 - 17.30	4/68/(50/0.3") > 1045 psf	Note 1			-6.90		3" Spoon (16.00' - 17.30'): 7DA (Top 0.9'): Gray, wet, silty CLAY, trace gravel, some wood bits. PP: 0.75, 0.75, 1.0, 1.25 TSF; T_V: 0.25 TSF. V1 (16.3'): 25.4 x 50.8 mm vane maxed out at 26.6 in-lbs.	
								-7.30		16.90 — 7DB (Bottom 0.4'): Gray, wet, silty, fine to coarse SAND, little gravel.	
17.5								-8.00		17.30 — Top of Rock at 17.3'. 18.00 — **Bottom of Exploration at 18.00 feet below ground surface.**	
20											
22.5											
25											
27.5											
30											

Remarks:

1. Blow counts for spoons driven through a disturbed field vane test location cannot be used to determine N-value.
2. Boring collapsed back to 6.5' after casing removed.
3. No water in boring after it collapsed to 6.5'.
4. Boring is located in the western bridge approach. Refer to Figure 2 for approximate location.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Marsh Bridge Supplemental Borings
Location: Tremomt / Southwest Harbor, ME

Boring No.: BB-MAR-201B
WIN: 21702.00

Driller:	NEBC	**Elevation (ft.)**	10 (See Remarks)	**Auger ID/OD:**	4"
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon / 3"
Logged By:	LLM	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140 lbs./ 30"
Date Start/Finish:	11:10 / 14:20 (9/28/17)	**Drilling Method:**	SSA / D+W	**Core Barrel:**	N/A
Boring Location:	See Remarks	**Casing ID/OD:**	4"	**Water Level*:**	2.7' at 13:50
Hammer Efficiency Factor:	.885	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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MV = Unsuccessful Field Vane Shear Test Attempt
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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
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N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_V = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA			Asphalt	
								9.00		1.00	
2.5											
5											
	1D	24/6	6.00 - 8.00	3/11/12/11	23	34				1D (6.00' - 8.00'): Tan-brown, damp, very stiff, SILT, some gravel, some sand, little clay.	
7.5							63				
								2.00		8.00	
	2D	24/7.2	8.00 - 10.00	3/8/8/7	16	24	52			2D (8.00' - 10.00'): Top 0.3' is pink granite and wood chips. Bottom 0.3' is dark brown to gray SILT, some sand, trace gravel.	
							94			8' - 10' wood and gravel in wash.	
10								0.00		10.00	
	3D	24/14.4	10.00 - 12.00	2/5/6/9	11	16	20			3D 3" Spoon (10.00' - 12.00'): Gray, wet, silty CLAY, little sand in top 0.6', trace organics throughout sample. PP: 1.75, 0.75, 1.5 TSF; T_V: 0.26 TSF	#428403 WC: 22.6%
							27				
12.5	4D	24/12	12.00 - 14.00	WOR/1/7/11	8	12	27			4D (12.00' - 14.00'): Top 0.4' is gray, wet, clayey SAND, little gravel (4DA). Bottom 0.6' is gray, wet, silty CLAY, little sand, trace gravel (4DB).	
							127				
								-4.00		14.00	
15							OPEN			Drill grinding and rocks coming up in wash at about 14' b.g.s., rollerconed to 15'. Split spoon refusal, top of rock at 15' b.g.s.	

Remarks:

1. Boring is located in the western bridge approach. Refer to Figure 2 for approximate location.
2. Boring elevation estimated from basemap provided by VHB on April 27, 2017.
3. Hammer efficiency factor provided in GZA SPT Energy Report for NEB Truck 83 Drillrig, NH plate number 4368, dated July 25, 2017.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Marsh Bridge Supplemental Borings
Location: Tremomt / Southwest Harbor, ME

Boring No.: BB-MAR-201B
WIN: 21702.00

Driller:	NEBC	**Elevation (ft.)**	10 (See Remarks)	**Auger ID/OD:**	4"
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon / 3"
Logged By:	LLM	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140 lbs./ 30"
Date Start/Finish:	11:10 / 14:20 (9/28/17)	**Drilling Method:**	SSA / D+W	**Core Barrel:**	N/A
Boring Location:	See Remarks	**Casing ID/OD:**	4"	**Water Level*:**	2.7' at 13:50
Hammer Efficiency Factor:	.885	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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V = Field Vane Shear Test, PP = Pocket Penetrometer
MV = Unsuccessful Field Vane Shear Test Attempt
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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: 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.
15								-5.00		15.00 **Bottom of Exploration at 15.00 feet below ground surface.**	
										Bottom of Exploration at 15.00 feet below ground surface.	
17.5											
20											
22.5											
25											
27.5											
30											

Remarks:

1. Boring is located in the western bridge approach. Refer to Figure 2 for approximate location.
2. Boring elevation estimated from basemap provided by VHB on April 27, 2017.
3. Hammer efficiency factor provided in GZA SPT Energy Report for NEB Truck 83 Drillrig, NH plate number 4368, dated July 25, 2017.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Marsh Bridge Supplemental Borings
Location: Tremomt / Southwest Harbor, ME

Boring No.: BB-MAR-202
WIN: 21702.00

Driller:	NEBC	**Elevation (ft.)**	10 (See Remarks)	**Auger ID/OD:**	4"
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon/ 3" Split
Logged By:	LLM	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140 lbs./ 30"
Date Start/Finish:	8:15 / 14:20 (9/29/17)	**Drilling Method:**	SSA / D+W	**Core Barrel:**	N/A
Boring Location:	See Remarks	**Casing ID/OD:**	4"	**Water Level*:**	5.4' at 14:10
Hammer Efficiency Factor:	.885	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	9.40		Asphalt — 0.60	
2.5											
5	1D	24/12	5.00 - 7.00	6/14/22/13	36	53	15			1D (5.00' - 7.00'): Top 0.55' is brown, bottom 0.45' is gray, damp, dense, fine to coarse SAND, little gravel, little silt, piece of asphalt from 0.55' - 0.75' (FILL).	
							67				
7.5	2D	24/10.8	7.00 - 9.00	7/4/3/4	7	10	37			2D (7.00' - 9.00'): Top 0.7' is brown-gray, damp, medium stiff, sandy SILT, little gravel. Sand and gravel content decreases with depth (2DA) (FILL).	
								2.00		8.00	
							23			Bottom 0.2' Dark brown, damp, medium stiff, ORGANIC SILT, little sand, some bits of wood (2DB).	
	3D MV1	24/19.2	9.00 - 11.00	2/2/3/2 Would Not Push	5	7	NR			3D 3" Spoon (9.00' - 11.00'): Top 0.3' is brown/gray, wet, loose, silty GRAVEL (3DA). Bottom 1.3' dark brown, damp, medium stiff, ORGANIC SILT (3DB). MV1 (9'): Failed 25.4 x 50.8 mm vane.	#428418 WC: 193% Ash: 64.8% Organic: 35.2%
10							24				
	4D V1	24/7.2	11.00 - 13.00	(WOH/1.5')/3 > 1045 psf	Note 1		20			4D (11.00' - 13.00'): Dark brown, wet, very soft, ORGANIC SILT, some gravel, organic odor. Bottom 0.1' is gray, fine sandy SILT. V1 (11.5'): 25.4 x 50.8 mm vane maxed out at 26.6 in-lbs.	
12.5	V2 MV2			1071 / 170 psf Would Not Push			27			V2 (12'): 24 ft-lbs/4 ft-lbs on 55 x 110 mm vane. MV2 (12.1'): Failed 55 x 110 mm vane would not push past to 13".	
								-3.00		13.00	
	5D MV3	24/13.2	13.00 - 15.00	WOH/2/1/3 Would Not Push	3	4	31			5D (13.00' - 15.00'): Top 0.2' is dark brown, wet, soft, ORGANIC SILT, little gravel. Bottom 0.9' is gray to dark gray, wet, soft, silty CLAY. little fine sand. MV3 (13.2'): Failed 55 x 110 mm vane, only pushed 0.2' due to gravel obstruction.	
15							37				

Remarks:

1. Blow counts for spoons driven through a disturbed field vane test location cannot be used to determine N-value.
2. NR = Not Recorded
3. Boring is located in the eastern bridge approach. Refer to Figure 2 for approximate location.
4. Boring elevation estimated from basemap provided by VHB on April 27, 2017.
5. Hammer efficiency factor provided in GZA SPT Energy Report for NEB Truck 83 Drillrig, NH plate number 4368, dated July 25, 2017.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Marsh Bridge Supplemental Borings
Location: Tremomt / Southwest Harbor, ME

Boring No.: BB-MAR-202
WIN: 21702.00

Driller:	NEBC	**Elevation (ft.)**	10 (See Remarks)	**Auger ID/OD:**	4"
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon/ 3" Split
Logged By:	LLM	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140 lbs./ 30"
Date Start/Finish:	8:15 / 14:20 (9/29/17)	**Drilling Method:**	SSA / D+W	**Core Barrel:**	N/A
Boring Location:	See Remarks	**Casing ID/OD:**	4"	**Water Level*:**	5.4' at 14:10
Hammer Efficiency Factor:	.885	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

Definitions:
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MV = Unsuccessful Field Vane Shear Test Attempt
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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: 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.
15	6D MV4	24/20.4	15.00 - 17.00	WOH/3/2/3 Would Not Push	5	7	OPEN			6D (15.00' - 17.00'): Gray, wet, medium stiff, silty CLAY, little gravel in top 0.3'. PP: 1.5, 1.75 TSF; T_V: 0.4 TSF MV4 (15'): Failed 55 x 110 mm vane due to gravel obstruction. Drill grinding.	#428403 WC: 25.1%
17.5	7D MV5	24/24	17.00 - 19.00	1/2/2/3 Would Not Push	4	6				7D 3" Spoon (17.00' - 19.00'): Gray, wet, soft, silty CLAY, trace gravel, trace fine sand, trace organics. 2" by 1" piece of gravel at bottom of spoon. MV5 (17'): Failed 55 x 110 mm vane due to gravel obstruction.	
20	8D MV6	24/24	19.00 - 21.00	WOH Would Not Push						8D (19.00' - 21.00'): Gray, wet, very soft, silty CLAY, some fine to medium sand at bottom of sample, trace gravel. PP: 0 TSF; T_V: 0.13 TSF MV6 (19'): Failed 55 x 110 mm vane due to gravel obstruction.	
	9D	24/8.4	21.00 - 23.00	WOR	Note 1					9D (21.00' - 23.00'): Gray, wet, very soft, silty CLAY, little sand, trace gravel. Sand content decreases with depth.	
22.5	V3 V4			804 / 134 psf 536 / 89 psf						V3 (22'): 18 ft-lbs/3 ft-lbs on 55 x 110 mm vane. V4 (23'): 12 ft-lbs/2 ft-lbs on 55 x 110 mm vane.	
	10D	24/24	23.00 - 25.00	WOR/WOR/WO1P/ 12"	Note 1					10D (23.00' - 25.00'): Gray, wet, very soft, silty CLAY, trace gravel, black streaks in bottom 0.2'.	
	V5			447 / 89 psf						V5 (24'): 10 ft-lbs/2 ft-lbs on 55 x 110 mm vane.	
25	V6			447 / 89 psf						V6 (25'): 10 ft-lbs/2 ft-lbs on 55 x 110 mm vane.	
27.5											
	11D	24/24	28.00 - 30.00	WOR	Note 1					11D (28.00' - 30.00'): Gray, wet, very soft, silty CLAY, trace shell fragments.	#428403 WC: 42.7%
	V7			670 / 112 psf						V7 (29'): 15 ft-lbs/2.5 ft-lbs on 55 x 110 mm vane.	
30											

Remarks:

1. Blow counts for spoons driven through a disturbed field vane test location cannot be used to determine N-value.
2. NR = Not Recorded
3. Boring is located in the eastern bridge approach. Refer to Figure 2 for approximate location.
4. Boring elevation estimated from basemap provided by VHB on April 27, 2017.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Marsh Bridge Supplemental Borings
Location: Tremomt / Southwest Harbor, ME

Boring No.: BB-MAR-202
WIN: 21702.00

Driller:	NEBC	**Elevation (ft.)**	10 (See Remarks)	**Auger ID/OD:**	4"
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon/ 3" Split
Logged By:	LLM	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140 lbs./ 30"
Date Start/Finish:	8:15 / 14:20 (9/29/17)	**Drilling Method:**	SSA / D+W	**Core Barrel:**	N/A
Boring Location:	See Remarks	**Casing ID/OD:**	4"	**Water Level*:**	5.4' at 14:10
Hammer Efficiency Factor:	.885	**Hammer Type:**	Automatic ☒ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
30	V8 MU1	2/0	30.00 - 30.17	692 / 134 psf Failed Shelby						V8 (30'): 15.5 ft-lbs/3 ft-lbs on 55 x 110 mm vane. MU1: Failed Shelby Tube due to equipment malfunction.	
32.5											
35	U1	24/6	35.00 - 37.00	Shelby Tube						U1 (35.00' - 37.00'): Gray, wet, silty CLAY.	
37.5								-27.30		37.30 Drill grinding.	
40											
42.5								-32.20		42.20 Top of Rock at 42.2' b.g.s.	
45								-33.30		43.30 **Bottom of Exploration at 43.30 feet below ground surface.**	

Remarks:

1. Blow counts for spoons driven through a disturbed field vane test location cannot be used to determine N-value.
2. NR = Not Recorded
3. Boring is located in the eastern bridge approach. Refer to Figure 2 for approximate location.
4. Boring elevation estimated from basemap provided by VHB on April 27, 2017.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Marsh Bridge Supplemental Borings
Location: Tremomt / Southwest Harbor, ME

Boring No.: BB-MAR-203A
WIN: 21702.00

Driller:	NEBC	**Elevation (ft.)**	4 (See Remarks)	**Auger ID/OD:**	4"
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	3" Split Spoon
Logged By:	LLM	**Rig Type:**	M-1	**Hammer Wt./Fall:**	140 lbs./ 30"
Date Start/Finish:	13:35 (9/25/17)/ 10:50 (9/26/17)	**Drilling Method:**	D+W	**Core Barrel:**	N/A
Boring Location:	See Remarks	**Casing ID/OD:**	4"	**Water Level*:**	Surface
Hammer Efficiency Factor: 0.6		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/15.6	0.00 - 2.00	(1/1')/(1/1')	1	1	HP	3.50		1D (0.00' - 2.00'): Top 0.5' is dark brown, wet, very soft, PEAT (1D).	
										0.50	
										Bottom 0.8' is gray-brown, very soft, wet, silty CLAY, little sand, roots throughout (1DB). PP: 0.5, 0.5 TSF; T_V: 0.16 TSF.	
2.5	2D	24/14.4	2.00 - 4.00	WO1P						2D (2.00' - 4.00'): Top 1' is gray-brown, wet, very soft, silty CLAY, little sand, trace organics (roots) (2DA). Bottom 0.2' is dark brown-black, wet, very soft, ORGANIC SILT, trace sand (2DB).	#428418 WC: 159% Ash: 84.9% Organics: 15.1%
	MU1	24/0	4.00 - 6.00	Failed Shelby						MU1 (4'): Failed Shelby Tube due to high root content.	
5	MU2	24/2.4	5.00 - 7.00	Failed Shelby						MU2 (5'): Failed Shelby Tube, gray-brown, damp, silty CLAY with roots.	
	U1	24/13.2	6.50 - 8.50	Shelby Tube			67			U1 (6.50' - 8.50'): Shelby Tube with gray-brown silty clay. Brown, organic smelling wash to 7'.	
								-3.00		7.00	
7.5							40			Milky gray wash with some bits of sand to 7' - 8.5'.	
	3D MV1	24/8.4	8.50 - 10.50	10/13/9/8 Would Not Push	22	22	OPEN			3D (8.50' - 10.50'): Dark gray, wet, very stiff, silty CLAY, trace gravel, trace sand. MV1 (8.5'): Failed 65 x 130 mm vane. Only pushed 0.2'due to an obstruction.	#428403 MC: 27.3%
10	MV2			Would Not Push						MV2 (10.5'): Failed 65 x 130 mm vane. Obstruction at 10.5'.	
	4D	24/13.2	11.50 - 13.50	3/4/4/3	8	8				4D (11.50' - 13.50'): Gray, wet, medium stiff, silty CLAY, trace sand. PP: 0, 0; T_V: 0.1 TSF.	
12.5	MV3			Would Not Push						MV3 (12.5'): Failed 65 x 130 mm vane. Vane only pushed 0.5', due to and obstruction.	
15											

Remarks:

1. Blow counts for spoons driven through a disturbed field vane test location cannot be used to determine N-value.
2. Water levels: Water at surface at 5:15 PM (9/25/17), water at surface at 7:15 AM (9/26/17), water at 3.4' in casing at 10:25 AM (9/26/17).
3. Boring is located at the toe of the embankment slope on the north eastern side of the bridge, near the abutment. Refer to Figure 2 for approximate location.
4. Boring elevation estimated from basemap provided by VHB on April 27, 2017.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Marsh Bridge Supplemental Borings
Location: Tremomt / Southwest Harbor, ME

Boring No.: BB-MAR-203A
WIN: 21702.00

Driller:	NEBC	**Elevation (ft.)**	4 (See Remarks)	**Auger ID/OD:**	4"
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	3" Split Spoon
Logged By:	LLM	**Rig Type:**	M-1	**Hammer Wt./Fall:**	140 lbs./ 30"
Date Start/Finish:	13:35 (9/25/17)/ 10:50 (9/26/17)	**Drilling Method:**	D+W	**Core Barrel:**	N/A
Boring Location:	See Remarks	**Casing ID/OD:**	4"	**Water Level*:**	Surface
Hammer Efficiency Factor: 0.6		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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SSA = Solid Stem Auger
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WOH = Weight of 140lb. Hammer
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S_u = Peak/Remolded Field Vane Undrained Shear Strength (psf)
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q_p = Unconfined Compressive Strength (ksf)
N-uncorrected = Raw Field SPT N-value
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N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
15	V1 V2			491 / 45 psf 580 / 89 psf						V1 (15'): 11 ft-lbs / 1 ft-lbs with 65 x 130 mm vane. V2 (16'): 13 ft-lbs / 2 ft-lbs with 65 x 130 mm vane.	
	5D V3	24/21.6	16.00 - 18.00	2/2/2/2 939/117 psf	Note 1					5D (16.00' - 18.00'): Gray, wet, soft, silty CLAY, trace gravel. PP: 0, 0; T_V: 0.5 (on dial) V3 (16.6'): 23.9 in-lbs / 4.5 in-lbs with 25.4 x 50.8 mm vane.	
17.5											
20											
	6D	24/24	21.00 - 23.00	1/1/1/2	Note 1					6D (21.00' - 23.00'): Gray, wet, very soft, silty CLAY, 2" by 4" rock at 21.5' b.g.s.	#428403 WC: 43.6%
22.5	V4			351 / 89 psf						V4 (22'): 8 ft-lbs/2 ft-lbs with 55 x 110 mm vane.	
	V5			402 / 67 psf						V5 (23'): 9 ft-lbs/1.5 ft-lbs with 55 x 110 mm vane.	
25											
	7D	24/24	26.00 - 28.00	(WOR/1')/1/3	Note 1					7D (26.00' - 28.00'): Gray with black streaks, wet, silty clay PP: 0; T_V: 0.9 on dial	
27.5	V6			424 / 89 psf						V6 (27'): 9.5 ft-lbs/2 ft-lbs with 55 x 110 mm vane.	
	V7			447 / 67 psf						V7 (28.00' - 30.00'): 10 ft-lbs/1.5 ft-lbs with 55 x 110 mm vane.	
								-25.00		Drill girnding. Top of Rock	
30										-29.00- **Bottom of Exploration at 29.00 feet below ground surface.**	

Remarks:

1. Blow counts for spoons driven through a disturbed field vane test location cannot be used to determine N-value.
2. Water levels: Water at surface at 5:15 PM (9/25/17), water at surface at 7:15 AM (9/26/17), water at 3.4' in casing at 10:25 AM (9/26/17).
3. Boring is located at the toe of the embankment slope on the north eastern side of the bridge, near the abutment. Refer to Figure 2 for approximate location.
4. Boring elevation estimated from basemap provided by VHB on April 27, 2017.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Marsh Bridge Supplemental Borings
Location: Tremomt / Southwest Harbor, ME

Boring No.: BB-MAR-203B
WIN: 21702.00

Driller:	NEBC	**Elevation (ft.)**	4 (See Remarks)	**Auger ID/OD:**	4"
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	LLM	**Rig Type:**	M-1	**Hammer Wt./Fall:**	140 lbs./ 30"
Date Start/Finish:	11:05/14:30 (9/26/17)	**Drilling Method:**	SSA / D+W	**Core Barrel:**	N/A
Boring Location:	See Remarks	**Casing ID/OD:**	4"	**Water Level*:**	0.8' at 14:25 (9/26/17)
Hammer Efficiency Factor: .6		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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MV = Unsuccessful Field Vane Shear Test Attempt
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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
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G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA				
2.5	1D	24/2.4	2.00 - 4.00	(WOR/1')/WO1P/1'	Note 1					1D (2.00' - 4.00'): Dark brown, wet, very soft, PEAT	
	V1 AS1 MV1			67 / 45 psf Would Not Push				0.80		V1 (1.5'): 1.5 ft-lbs/ 1 ft-lbs with 55 x 110 mm vane. 3.20 AS1 (3.20' - 4.00'): Dark brown, wet, very soft, ORGANIC SILT, trace gravel. MV1 (1.5'): 55 x 110 mm vane only pushed 0.3'.	
5											
	V2 2D	24/18	5.00 - 7.00	290 / 45 psf (WOR/1')/1/4	Note 1		OPEN			V2 (5'): 6.5 ft-lbs/ 1 ft-lbs with 55 x 110 mm vane. 2D (5.00' - 7.00'): Top 0.9' is dark brown, wet, PEAT. 0.9' to 1.2' is brown, wet, ORGANIC SILT (2DA). Bottom 0.3' is gray, wet, silty fine SAND (2DB).	
	V3			313 / 22 psf						V3 (6'): 7 ft-lbs/ 0.5 ft-lbs with 55 x 110 mm vane.	
								-3.00		7.00	
7.5	3D	24/21.6	7.00 - 9.00	1/1/2/3	3	3				3D (7.00' - 9.00'): Top 0.6' is gray, wet, silty fine SAND, trace organics (3DA). Bottom 1.2' is dark gray, wet, soft, fine sandy SILT, trace organics (2DB). PP: 0.75, 0.5, 0.75 TSF; T_V: 0.3 TSF	
	MV2 4D	12/12	9.00 - 10.00	Would Not Push 2/3						MV2 (9'): 55 x 110 mm vane would not push. 4D (9.00' - 10.00'): Gray, damp, fine sandy SILT, trace gravel, trace organics.	
10	5D MV3	24/20.4	10.00 - 12.00	2/4/4/3 Would Not Push	8	8				5D (10.00' - 12.00'): Gray, moist, clayey SILT, some fine sand, trace organics (wood chips). PP: 1.0, 1.5, 1.25 TSF; T_V: 0.25 TSF MV2 (10'): 55 x 110 mm vane would not push.	
12.5											
	6D V4	24/24	13.00 - 15.00	WO1P 893 / 89 psf	Note 1					6D (13.00' - 15.00'): Gray, wet, very soft, silty CLAY, little fine sand in layers. PP: 0; T_V: 0.18 TSF V4 (13'): 20 ft-lbs/ 2 ft-lbs with 55 x 110 mm vane.	#428403 WC: 36.9%
	V5			714 / 67 psf						V5 (14'): 16 ft-lbs/ 1.5 ft-lbs with 55 x 110 mm vane.	
15											

Remarks:

1. Blow counts for spoons driven through a disturbed field vane test location cannot be used to determine N-value.
2. Water Levels: 1' b.g.s. at 14:20 9/26/2017; 0.8' b.g.s. at 14:25 9/26/2017
3. Boring is located at the toe of the embankment slope on the north eastern side of the bridge. Refer to Figure 2 for approximate location.
4. Boring elevation estimated from basemap provided by VHB on April 27, 2017.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Marsh Bridge Supplemental Borings
Location: Tremomt / Southwest Harbor, ME

Boring No.: BB-MAR-203B
WIN: 21702.00

Driller:	NEBC	**Elevation (ft.)**	4 (See Remarks)	**Auger ID/OD:**	4"
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	LLM	**Rig Type:**	M-1	**Hammer Wt./Fall:**	140 lbs./ 30"
Date Start/Finish:	11:05/14:30 (9/26/17)	**Drilling Method:**	SSA / D+W	**Core Barrel:**	N/A
Boring Location:	See Remarks	**Casing ID/OD:**	4"	**Water Level*:**	0.8' at 14:25 (9/26/17)
Hammer Efficiency Factor:	.6	**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: 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.
15	V6			670 / 89 psf						V6 (15'): 15 ft-lbs/ 2 ft-lbs with 55 x 110 mm vane.	
	U1 V7	24/20.4	16.00 - 18.00	Shelby Tube 536 / 45 psf						U1 (16.00' - 18.00'): Gray, wet, silty CLAY, little fine sand. V7 (16'): 12 ft-lbs/ 1 ft-lbs with 55 x 110 mm vane.	
17.5								-14.00		18.00 **Bottom of Exploration at 18.00 feet below ground surface.**	
20											
22.5											
25											
27.5											
30											

Remarks:

1. Blow counts for spoons driven through a disturbed field vane test location cannot be used to determine N-value.
2. Water Levels: 1' b.g.s. at 14:20 9/26/2017; 0.8' b.g.s. at 14:25 9/26/2017
3. Boring is located at the toe of the embankment slope on the north eastern side of the bridge. Refer to Figure 2 for approximate location.
4. Boring elevation estimated from basemap provided by VHB on April 27, 2017.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Marsh Bridge Supplemental Borings
Location: Tremomt / Southwest Harbor, ME

Boring No.: BB-MAR-204
WIN: 21702.00

Driller:	NEBC	**Elevation (ft.)**	4 (See Remarks)	**Auger ID/OD:**	4"
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	LLM	**Rig Type:**	M-1	**Hammer Wt./Fall:**	140 lbs./ 30"
Date Start/Finish:	14:45/ 17:05 (9/26/17)	**Drilling Method:**	SSA / D+W	**Core Barrel:**	N/A
Boring Location:	See Remarks	**Casing ID/OD:**	4"	**Water Level*:**	1.2' at 17:12 (9/26/17)
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA				
2.5	U1	24/16.8	2.00 - 4.00	Shelby Tube			OPEN			U1 (2.00' - 4.00'): Dark brown, organic SILT.	
								0.00		4.00	
	1D	24/4.8	4.00 - 6.00	WOH	Note 1					1D (4.00' - 6.00'): Dark brown, wet, very soft, ORGANIC SILT, organic smell.	
5	V1			313 / 45 psf						V1 (5'): 7 ft-lbs / 1 ft-lbs with 55 x 110 mm vane.	
	V2 2D	24/12	6.00 - 8.00	313 / 89 psf WO1P	Note 1					V2 (6'): 7 ft-lbs / 1 ft-lbs with 55 x 110 mm vane. 2D (6.00' - 8.00'): Top 0.4' is dark brown, wet, PEAT, some silt (2DA).	
7.5	V3			313 / 0 psf						Bottom 0.6' is gray, wet, very soft, SILT, little sand, trace organics (leaves/roots) (2DB). V3 (7'): 7 ft-lbs / 0 ft-lbs with 55 x 110 mm vane.	
	3D V4	24/12	8.00 - 10.00	2/3/3/4 357 / 45 psf	Note 1					3D (8.00' - 10.00'): Gray, wet, medium stiff, silty CLAY, little fine sand, trace gravel, trace organics. PP: 0.25, 0 TSF; T_V: 0.25 TSF V4 (8'): 8 ft-lbs / 1 ft-lbs with 55 x 110 mm vane.	#428418 WC: 29% Ash: 99.4% Organics: 0.6%
								-5.00		9.00	
	MV1			Would Not Push						MV1 (9'): Failed 55 x 110 mm vane.	
10	4D V5	24/24	10.00 - 12.00	1/(1/1')/1 1002 / 255 psf	Note 1					4D (10.00' - 12.00'): Gray, wet, very soft, silty CLAY, little fine sand. V5 (10.3'): 25.5 in-lbs / 6.5 in-lbs with 25.4 x 50.8 mm vane.	#428403 WC: 36.1%
12.5	5D V6	24/24	12.00 - 14.00	WO1P 766 / 157 psf	Note 1					5D (12.00' - 14.00'): Gray, wet, very soft, silty CLAY, little fine sand, trace organics. Sandier at top than bottom. V6 (12.3'): 19.5 in-lbs / 4 in-lbs with 25.4 x 50.8 mm vane.	
15	6D	24/24	14.00 - 16.00	WO1P	Note 1					6D (14.00' - 16.00'): Gray, wet, silty CLAY, little fine sand, trace gravel. PP: 0; T_V: 0.07 TSF	#428403 WC: 37.8%

Remarks:

1. Blow counts for spoons driven through a disturbed field vane test location cannot be used to determine N-value.
2. Water Levels: Water at 3.4' b.g.s. in casing at 17:05; water at 1.2' b.g.s. at 17:12 after casing pulled. 3. Boring is located at the toe of the embankment slope on the north eastern side of the bridge. Refer to Figure 2 for approximate location.
4. Boring elevation estimated from basemap provided by VHB on April 27, 2017.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Marsh Bridge Supplemental Borings
Location: Tremomt / Southwest Harbor, ME

Boring No.: BB-MAR-204
WIN: 21702.00

Driller:	NEBC	**Elevation (ft.)**	4 (See Remarks)	**Auger ID/OD:**	4"
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon
Logged By:	LLM	**Rig Type:**	M-1	**Hammer Wt./Fall:**	140 lbs./ 30"
Date Start/Finish:	14:45/ 17:05 (9/26/17)	**Drilling Method:**	SSA / D+W	**Core Barrel:**	N/A
Boring Location:	See Remarks	**Casing ID/OD:**	4"	**Water Level*:**	1.2' at 17:12 (9/26/17)
Hammer Efficiency Factor:	0.6	**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: 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.
15	V7			904 / 236 psf						V7 (15.5'): 23 in-lbs / 6 in-lbs with 25.4 x 50.8 mm vane.	
								-12.00		16.00	
										Bottom of Exploration at 16.00 feet below ground surface.	
17.5											
20											
22.5											
25											
27.5											
30											

Remarks:

1. Blow counts for spoons driven through a disturbed field vane test location cannot be used to determine N-value.
2. Water Levels: Water at 3.4' b.g.s. in casing at 17:05; water at 1.2' b.g.s. at 17:12 after casing pulled. 3. Boring is located at the toe of the embankment slope on the north eastern side of the bridge. Refer to Figure 2 for approximate location.
4. Boring elevation estimated from basemap provided by VHB on April 27, 2017.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Marsh Bridge Supplemental Borings
Location: Tremomt / Southwest Harbor, ME

Boring No.: BB-MAR-205
WIN: 21702.00

Driller:	NEBC	**Elevation (ft.)**	4	**Auger ID/OD:**	4"
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon/ 3" Split
Logged By:	LLM	**Rig Type:**	M-1	**Hammer Wt./Fall:**	140 lbs./ 30"
Date Start/Finish:	9:34 / 16:30 (9/27/17)	**Drilling Method:**	SSA / D+W	**Core Barrel:**	N/A
Boring Location:		**Casing ID/OD:**	4"	**Water Level*:**	1.3' b.g.s. at 16:20 (9/27/17)
Hammer Efficiency Factor:	0.6	**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

Sample Information

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							SSA				
2.5	1D	24/12	2.00 - 4.00	WOH/1/1/1	2	2				1D (2.00' - 4.00'): Top 0.3' gray-brown, wet, fine to coarse SAND, little gravel, little silt (1DA) (Fill).	
								1.00		3.00	
										Bottom 0.7' is gray-brown, damp, ORGANIC SILT, with roots and organic odor (1DB).	
	2D	24/15.6	4.00 - 6.00	WO1P/(1/1.5')	Note 1		HP			2D (4.00' - 6.00'): Top 0.3' is gray-brown, wet, sandy SILT, little gravel (2DA). Bottom 1' is gray-brown to dark brown-black to red-brown, damp, very soft, ORGANIC SILT (2DB). V1 (5'): 15.5 ft-lbs / 2.5 ft-lbs with 55 x 110 vane.	
5	V1 V2			692 / 112 psf 491 / 89 psf			4			V2 (6'): 11 ft-lbs / 2 ft-lbs with 55 x 110 vane.	
	3D	24/12	6.00 - 8.00	WO1P/1/1/2	Note 1		4			3D (6.00' - 8.00'): Top 0.6' is brown-black to red-brown to light brown, wet, very soft, ORGANIC SILT, trace gravel, roots throughout (3DA). Bottom 0.4' is gray, wet, damp, loose, silty, fine to medium SAND, trace gravel, trace organics (2DB).	
7.5	V3 MV1			357 / 67 psf Would Not Push			5			V3 (7'): 8 ft-lbs / 1.5 ft-lbs with 55 x 110 vane. MV1 (7.5'): 55 x 110 vane only pushed 0. 5'. V4 (8'): 20.5 in-lbs / 7.5 in-lbs with 24.5 x 50.8 vane.	
	V4 4D	24/24	8.00 - 10.00	805 / 295 psf 4/4/5/12	9	9	20	-4.50		4D (8.00' - 10.00'): Top 0.55' is tan-brown, damp, stiff, ORGANIC SILT, trace gravel, roots throughout, organic odor (4DA).	
										8.50	
							20			Bottom 1.45' is gray with darker streaks throughout, damp, stiff, silty CLAY, little fine sand. Sandier at top and becoming clayier towards bottom of sample (4DB).	
10	5D MV2	24/12	10.00 - 12.00	1/4/4/3 Would Not Push	8	8	51			5D (10.00' - 12.00'): Gray, wet, silty CLAY, little fine sand. Bottom 0.1' is sand. PP: 0, 0.75 TSF; T_v: 0.05 TSF MV2 (10'): Failed 25.4 x 50.8 mm vane.	
							24				
12.5	6D MV3	24/16.8	12.00 - 14.00	9/2/2/1 Would Not Push	4	4	17			6D (12.00' - 14.00'): Gray, wet, silty CLAY, little fine sand. PP: 0, 0.25 TSF; T_v: 0.05 TSF MV3 (12'): Failed 25.4 x 50.8 mm vane.	
							11				
15	7D	24/7.2	14.00 - 16.00	10/13/20/15 Would Not Push	33	33	11			7D 3" Spoon (14.00' - 16.00'): Gray, wet, silty CLAY, little fine sand, trace gravel. MV3 (14'): Failed 25.4 x 50.8 mm vane.	

Remarks:

1. Blow counts for spoons driven through a disturbed field vane test location cannot be used to determine N-value.
2. Water Levels: Water at 0.9' b.g.s. in casing at 15:54, water at 1.3' b.g.s. after casing pulled at 16:20.
3. Boring is located at the toe of the embankment slope on the south eastern side of the bridge, near the abutment. Refer to Figure 2 for approximate location.
4. Boring elevation estimated from basemap provided by VHB on April 27, 2017.

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Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Marsh Bridge Supplemental Borings
Location: Tremomt / Southwest Harbor, ME

Boring No.: BB-MAR-205
WIN: 21702.00

Driller:	NEBC	**Elevation (ft.)**	4	**Auger ID/OD:**	4"
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon/ 3" Split
Logged By:	LLM	**Rig Type:**	M-1	**Hammer Wt./Fall:**	140 lbs./ 30"
Date Start/Finish:	9:34 / 16:30 (9/27/17)	**Drilling Method:**	SSA / D+W	**Core Barrel:**	N/A
Boring Location:		**Casing ID/OD:**	4"	**Water Level*:**	1.3' b.g.s. at 16:20 (9/27/17)
Hammer Efficiency Factor:	0.6	**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☒		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
15							38				
	8D	24/21.6	16.00 - 18.00	2/2/3/3	Note 1		25			8D (16.00' - 18.00'): Gray, wet, silty CLAY, trace gravel, trace sand. PP: 0.25, 0.75, 0.5 TSF; T_V: 0.3 TSF	
17.5	V5			> 1045 psf			21			V5 (17'): Maxed out 25.4 x 50.8 mm vane at 26.6 in-lbs.	
	9D	24/24	18.00 - 20.00	WOR/WOR/WO1P/1'	Note 1		22			9D (18.00' - 20.00'): Gray, wet, very soft, silty CLAY. PP: 0; T_V: 0.5	#428403 WC: 39.5%
	V6			746 / 118 psf			20			V6 (19'): 19 in-lbs / 3 in-lbs with 25.4 x 50.8 mm vane.	
20	10D	24/24	20.00 - 22.00	WOR/(1/1')/1	Note 1		57			10D (20.00' - 22.00'): Gray with darker gray streaks, wet, silty CLAY, trace sand, trace shell fragments. PP: 0; T_V: 0.02 TSF	
	V7			688 / 118 psf			31			V7 (21'): 17.5 in-lbs / 3 in-lbs with 25.4 x 50.8 mm vane.	
22.5	MU1 11D	24/3	22.00 - 24.00	Failed Shelby			28			MU1 (22.00' - 24.00'): Failed Shelby Tube. 11D (22.00' - 24.00') Gray with black streaks, wet, silty CLAY. Material that fell out of Shelby tube.	
							22				
	U1	24/24	24.00 - 26.00	Shelby Tube			OPEN			U1 (24.00' - 26.00'): Gray, wet, silty CLAY, one small piece of gravel on top.	
25											
	12D	28.8/24	26.00 - 28.40	WOR	Note 1					12D (26.00' - 28.40'): Gray with some dark gray streaks, wet, silty, CLAY, trace gravel. PP: 0; T_V: 0.5 (on dial) Spoon sunk to 28.4'.	#428403 WC: 42.4%
27.5	V8 V9			357 / 67 psf 402 / 67 psf						V8 (27'): 8 ft-lbs / 1.5 ft-lbs with 55 x 110 mm vane. V9 (28'): 9 ft-lbs / 1 ft-lbs with with 55 x 110 mm vane.	
	13D MV4	24/8.4	28.00 - 30.00	(WOH/1')/4/16 Would Not Push	4	4				13D (28.00' - 30.00'): Gray, wet, soft, silty CLAY, trace gravel, trace sand. MV4 (28.5'): Failed 55 x 110 mm vane, only pushed 0.5'.	
30											

Remarks:

1. Blow counts for spoons driven through a disturbed field vane test location cannot be used to determine N-value.
2. Water Levels: Water at 0.9' b.g.s. in casing at 15:54, water at 1.3' b.g.s. after casing pulled at 16:20.
3. Boring is located at the toe of the embankment slope on the south eastern side of the bridge, near the abutment. Refer to Figure 2 for approximate location.
4. Boring elevation estimated from basemap provided by VHB on April 27, 2017.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Marsh Bridge Supplemental Borings
Location: Tremomt / Southwest Harbor, ME

Boring No.: BB-MAR-205
WIN: 21702.00

Driller:	NEBC	**Elevation (ft.)**	4	**Auger ID/OD:**	4"
Operator:	Mike Porter	**Datum:**	NAVD 88	**Sampler:**	Standard Split Spoon/ 3" Split
Logged By:	LLM	**Rig Type:**	M-1	**Hammer Wt./Fall:**	140 lbs./ 30"
Date Start/Finish:	9:34 / 16:30 (9/27/17)	**Drilling Method:**	SSA / D+W	**Core Barrel:**	N/A
Boring Location:		**Casing ID/OD:**	4"	**Water Level*:**	1.3' b.g.s. at 16:20 (9/27/17)
Hammer Efficiency Factor:	0.6	**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: 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.
30											
								-28.20		32.20	
32.5										Drill grinding at 32.2', milky gray wash.	
35										Drill grinding at 35'.	
										Rig shaking and grinding at 37'. Gray-brown wash with sand.	
37.5											
								-34.10		38.10	
								-34.30		Top of Rock at 38.1' b.g.s., drill shaking.	
										38.30	
										Bottom of Exploration at 38.30 feet below ground surface.	
40											
42.5											
45											

Remarks:

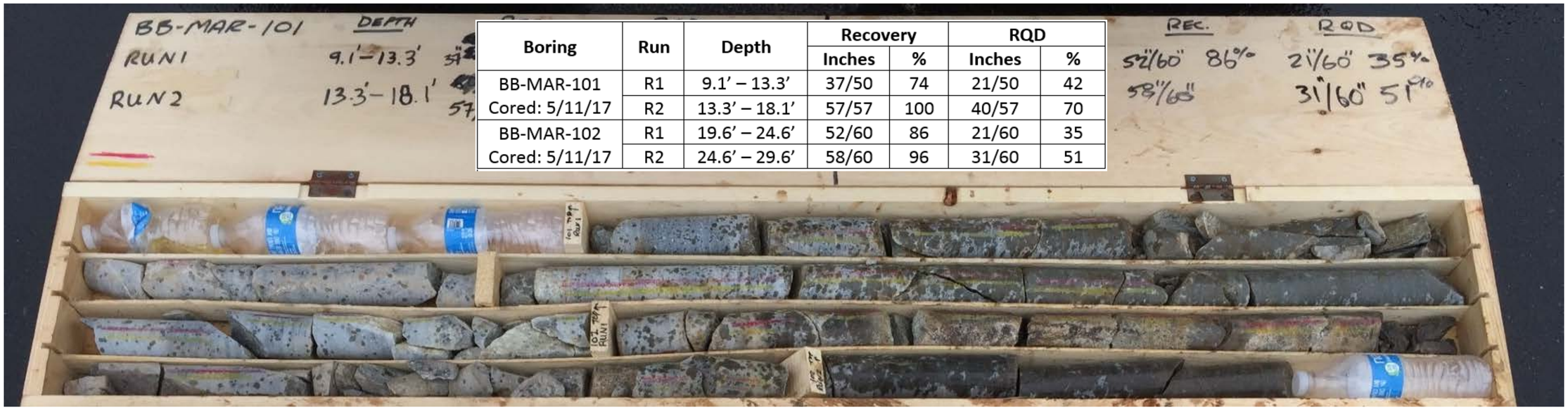
1. Blow counts for spoons driven through a disturbed field vane test location cannot be used to determine N-value.
2. Water Levels: Water at 0.9' b.g.s. in casing at 15:54, water at 1.3' b.g.s. after casing pulled at 16:20.
3. Boring is located at the toe of the embankment slope on the south eastern side of the bridge, near the abutment. Refer to Figure 2 for approximate location.
4. Boring elevation estimated from basemap provided by VHB on April 27, 2017.

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.

APPENDIX B
ROCK CORE PHOTOGRAPHS

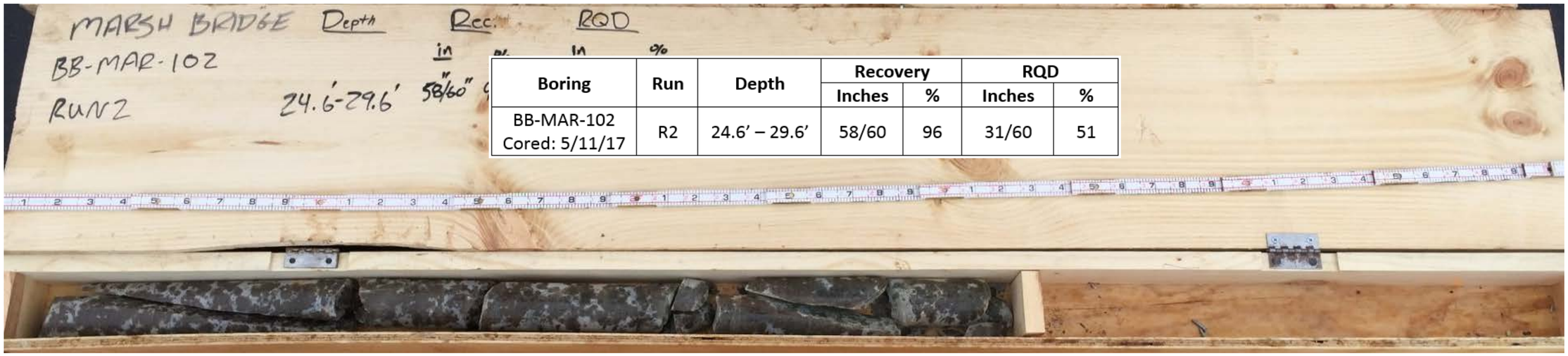
APPENDIX B
Rock Core Photos
Marsh Bridge Replacement
Tremont and Southwest Harbor, Maine
MaineDOT WIN: 21702.00



Boring	Run	Depth	Recovery		RQD	
			Inches	%	Inches	%
BB-MAR-101	R1	9.1' – 13.3'	37/50	74	21/50	42
Cored: 5/11/17	R2	13.3' – 18.1'	57/57	100	40/57	70
BB-MAR-102	R1	19.6' – 24.6'	52/60	86	21/60	35
Cored: 5/11/17	R2	24.6' – 29.6'	58/60	96	31/60	51

Rows 1,2 = BB-MAR-101 Run 1: 9.1 - 13.3 ft-bgs.
Rows 2,3 = BB-MAR-101 Run 2: 13.3 - 18.1 ft-bgs.
Rows 3,4 = BB-MAR-102 Run 1: 19.6 - 24.6 ft-bgs.
Rows 4 = BB-MAR-102 Run 2: 24.6 - 29.6 ft-bgs

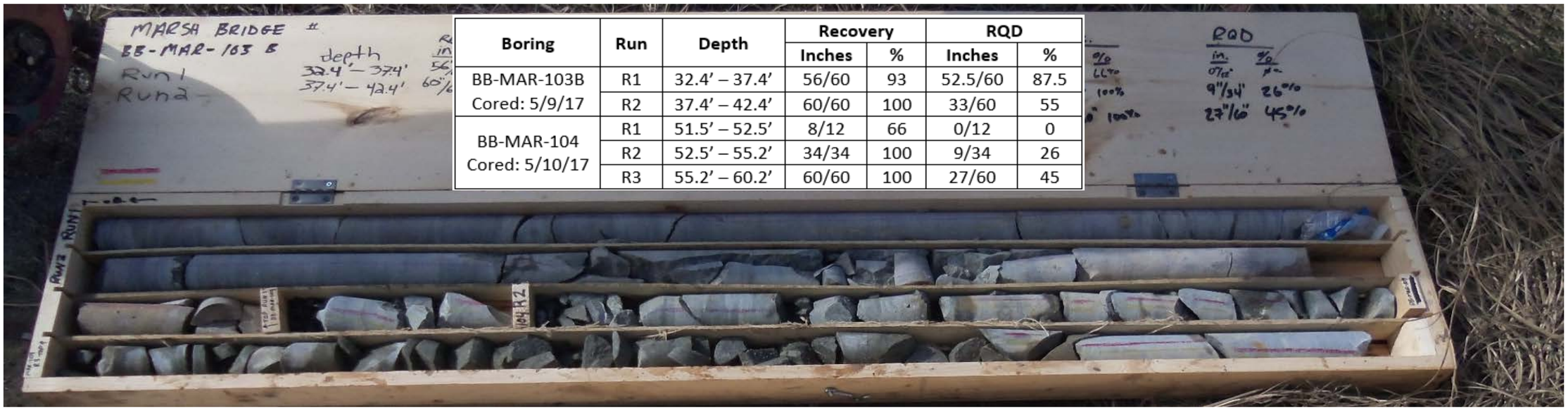
APPENDIX B
Rock Core Photos
Marsh Bridge Replacement
Tremont and Southwest Harbor, Maine
MaineDOT WIN: 21702.00



Boring	Run	Depth	Recovery		RQD	
			Inches	%	Inches	%
BB-MAR-102 Cored: 5/11/17	R2	24.6' – 29.6'	58/60	96	31/60	51

Rows 1= BB-MAR-102 Run 2: 24.6 - 29.6 ft-bgs.

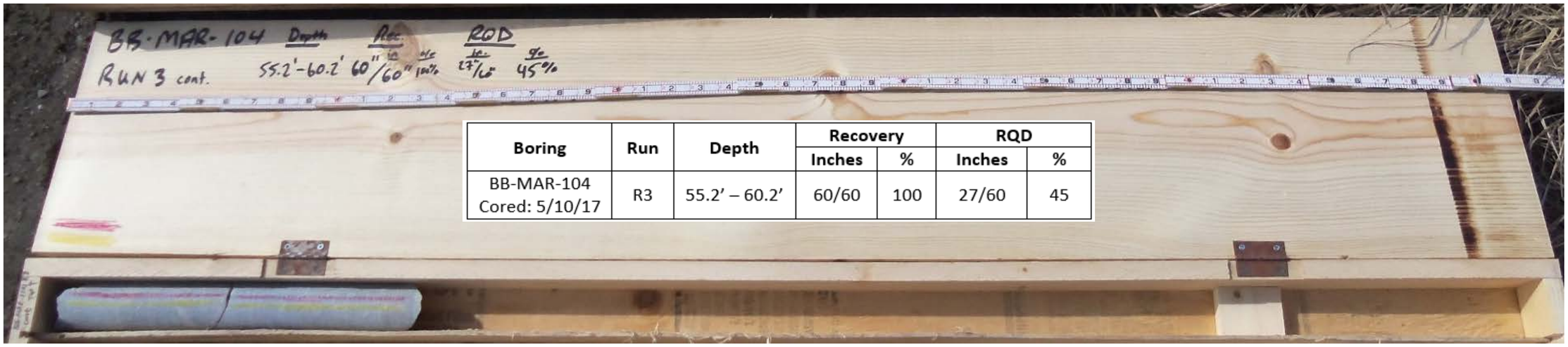
APPENDIX B
Rock Core Photos
Marsh Bridge Replacement
Tremont and Southwest Harbor, Maine
MaineDOT WIN: 21702.00



Boring	Run	Depth	Recovery		RQD	
			Inches	%	Inches	%
BB-MAR-103B Cored: 5/9/17	R1	32.4' – 37.4'	56/60	93	52.5/60	87.5
	R2	37.4' – 42.4'	60/60	100	33/60	55
BB-MAR-104 Cored: 5/10/17	R1	51.5' – 52.5'	8/12	66	0/12	0
	R2	52.5' – 55.2'	34/34	100	9/34	26
	R3	55.2' – 60.2'	60/60	100	27/60	45

Row 1 = BB-MAR-103B Run 1: 32.4 - 37.4 ft-bgs.
Rows 2,3 = BB-MAR-103B Run 2: 37.4 - 42.4 ft-bgs.
Row 3 = BB-MAR-104 Run 1: 51.5 - 52.5 ft-bgs.
Row 3 = BB-MAR-104 Run 2: 52.5 - 55.2 ft-bgs
Rows 4 = BB-MAR-104 Run 3: 55.2 - 60.2 ft-bgs

APPENDIX B
Rock Core Photos
Marsh Bridge Replacement
Tremont and Southwest Harbor, Maine
MaineDOT WIN: 21702.00



Boring	Run	Depth	Recovery		RQD	
			Inches	%	Inches	%
BB-MAR-104 Cored: 5/10/17	R3	55.2' – 60.2'	60/60	100	27/60	45

Row 1 = BB-MAR-104 Run 3: 55.2 - 60.2 ft-bgs.

APPENDIX C
LABORATORY TEST RESULTS

Client:	Golder Associates				
Project:	Marsh Bridge				
Location:	Tremont/Southwest Harbor, ME			Project No:	GTX-306503
Boring ID:	---	Sample Type:	---	Tested By:	jbr
Sample ID:	---	Test Date:	06/01/17	Checked By:	emm
Depth :	---	Test Id:	412806		

Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content, %
BB-MAR-101	1	1-3 ft	Moist, drak grayish brown clayey sand with gravel	9.8
BB-MAR-101	2	5-7 ft	Moist, dark gray clay with sand	23.5
BB-MAR-102	1	1-3 ft	Moist, dark brown silty sand and gravel	8.0
BB-MAR-102	2	5-7 ft	Moist, very dark brown clay with gravel	27.7
BB-MAR-102	3	12-14 ft	Moist, olive clay with sand	19.9
BB-MAR-102	4	15-17 ft	Moist, olive clay	26.1
BB-MAR-103A	2	2.5-4.5 ft	Moist, dark grayish brown silty sand and gravel	12.2

Notes: Temperature of Drying : 110° Celsius

Client:	Golder Associates				
Project:	Marsh Bridge				
Location:	Tremont/Southwest Harbor, ME			Project No:	GTX-306503
Boring ID:	---	Sample Type:	---	Tested By:	jbr
Sample ID:	---	Test Date:	06/01/17	Checked By:	emm
Depth :	---	Test Id:	412816		

Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content, %
BB-MAR-103B	5	8-10 ft	Moist, dark gray clay with sand and organics	62.0
BB-MAR-103B	10	18-20 ft	Moist, olive clay	25.4
BB-MAR-103B	11	20-22 ft	Moist, olive clay	37.2
BB-MAR-103B	12	22-24 ft	Moist, olive clay	41.7
BB-MAR-104	5	10.5-12.5 ft	Moist, very dark brown silt with organics	271.0
BB-MAR-104	6B	12.5-14.5 ft	Moist, olive clay	28.0
BB-MAR-104	7	15-17 ft	Moist, olive clay	35.5
BB-MAR-104	8	17-19 ft	Moist, olive clay	35.9
BB-MAR-104	9	25-27 ft	Moist, olive clay	43.6
BB-MAR-104	11	35-37 ft	Moist, olive clay	40.9

Notes: Temperature of Drying : 110° Celsius

Client:	Golder Associates				
Project:	Marsh Bridge				
Location:	Tremont/Southwest Harbor, ME			Project No:	GTX-306503
Boring ID:	---	Sample Type:	---	Tested By:	jbr
Sample ID:	---	Test Date:	10/13/17	Checked By:	emm
Depth :	---	Test Id:	428403		

Moisture Content of Soil and Rock - ASTM D2216

Boring ID	Sample ID	Depth	Description	Moisture Content, %
BB-MAR-203A	3D	8.5-10.5 ft	Moist, gray clay	27.3
BB-MAR-201B	3D	10-12 ft	Moist, gray clay	22.6
BB-MAR-204	4D	10-12 ft	Moist, gray clay	36.1
BB-MAR-203B	6D	12-14 ft	Moist, gray clay	36.9
BB-MAR-204	6D	14-16 ft	Moist, gray clay	37.8
BB-MAR-202	6D	15-17 ft	Moist, gray clay	25.1
BB-MAR-205	9D	18-20 ft	Moist, gray clay	39.5
BB-MAR-203A	6D (1/2)	21-23 ft	Moist, gray clay	43.6
BB-MAR-205	12D	26-28.4 ft	Moist, gray clay	42.4
BB-MAR-202	11D	28-30 ft	Moist, gray clay	42.7

Notes: Temperature of Drying : 110° Celsius

Client:	Golder Associates				
Project:	Marsh Bridge				
Location:	Tremont/Southwest Harbor, ME			Project No:	GTX-306503
Boring ID:	---	Sample Type:	---	Tested By:	cam
Sample ID:	---	Test Date:	06/06/17	Checked By:	emm
Depth :	---	Test Id:	412818		

Moisture, Ash, and Organic Matter - ASTM D2974

Boring ID	Sample ID	Depth	Description	Moisture Content, %	Ash Content, %	Organic Matter, %
BB-MAR-103B	5	8-10 ft	Moist, dark gray clay with sand and organics	62	92.3	7.7
BB-MAR-104	5	10.5-12.5 ft	Moist, very dark brown silt with organics	271	64.3	35.7

Notes: Moisture content determined by Method A and reported as a percentage of oven-dried mass; dried to a constant mass at temperature of 105º C
Ash content and organic matter determined by Method C; dried to constant mass at temperature 440º C

Client:	Golder Associates				
Project:	Marsh Bridge				
Location:	Tremont/Southwest Harbor, ME			Project No:	GTX-306503
Boring ID:	---	Sample Type:	---	Tested By:	cam
Sample ID:	---	Test Date:	10/26/17	Checked By:	emm
Depth :	---	Test Id:	428418		

Moisture, Ash, and Organic Matter - ASTM D2974

Boring ID	Sample ID	Depth	Description	Moisture Content, %	Ash Content, %	Organic Matter, %
BB-MAR-202	3DB	9-11 ft	Moist, dark brown silt with organics	193	64.8	35.2
BB-MAR-203A	2DA	2-4 ft	Moist, very dark brown silty sand with organics	159	84.9	15.1
BB-MAR-204	3D	8-10 ft	Moist, olive clay	29	99.4	.6

Notes: Moisture content determined by Method A and reported as a percentage of oven-dried mass;
dried to a constant mass at temperature of 105º C
Ash content and organic matter determined by Method C; dried to constant mass at temperature 440º C

Client:	Golder Associates				
Project:	Marsh Bridge				
Location:	Tremont/Southwest Harbor, ME			Project No:	GTX-306503
Boring ID:	BB-MAR-203A	Sample Type:	tube	Tested By:	cam
Sample ID:	U1	Test Date:	11/06/17	Checked By:	emm
Depth :	6.5-8.5 ft	Test Id:	428419		
Test Comment:	---				
Visual Description:	Wet, very dark brown silt with sand and organics				
Sample Comment:	---				

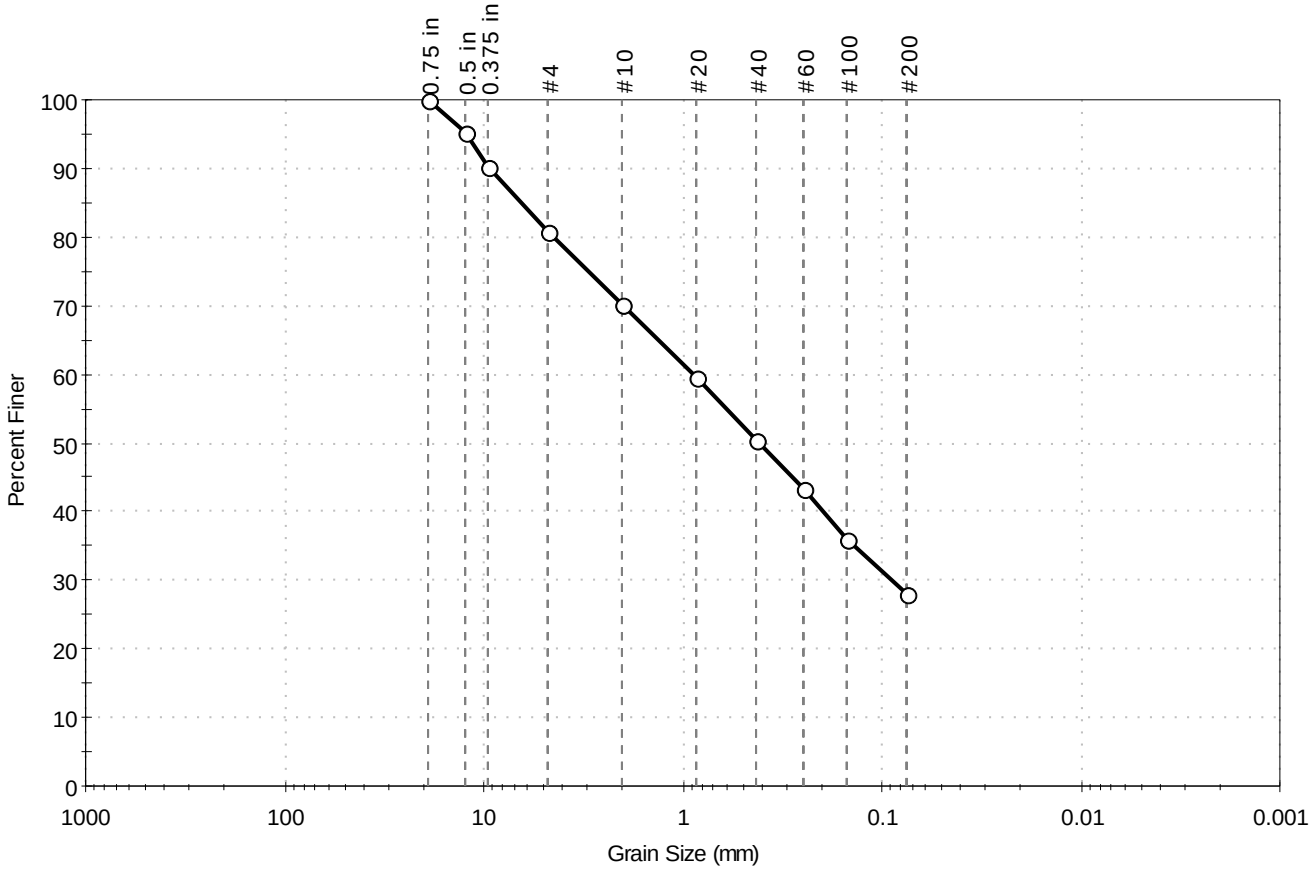
Moisture, Ash, and Organic Matter - ASTM D2974

Boring ID	Sample ID	Depth	Description	Moisture Content, %	Ash Content, %	Organic Matter, %
BB-MAR-203A	U1	6.5-8.5 ft	Wet, very dark brown silt with sand and organics	286	71.9	28.1

Notes: Moisture content determined by Method A and reported as a percentage of oven-dried mass;
dried to a constant mass at temperature of 105º C
Ash content and organic matter determined by Method C; dried to constant mass at temperature 440º C

Client:	Golder Associates				
Project:	Marsh Bridge				
Location:	Tremont/Southwest Harbor, ME			Project No:	GTX-306503
Boring ID:	BB-MAR-101	Sample Type:	jar	Tested By:	jbr
Sample ID:	1	Test Date:	06/02/17	Checked By:	emm
Depth :	1-3 ft	Test Id:	412791		
Test Comment:	---				
Visual Description:	Moist, drak grayish brown clayey sand with gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D422



%Cobble	%Gravel	%Sand	%Silt & Clay Size
---	19.1	53.0	27.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	95		
0.375 in	9.50	90		
#4	4.75	81		
#10	2.00	70		
#20	0.85	60		
#40	0.42	50		
#60	0.25	43		
#100	0.15	36		
#200	0.075	28		

Coefficients

D_{85} = 6.4141 mm	D_{30} = 0.0896 mm
D_{60} = 0.8805 mm	D_{15} = N/A
D_{50} = 0.4119 mm	D_{10} = N/A
C_u = N/A	C_c = N/A

Classification

ASTM N/A

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

Sample / Test Description

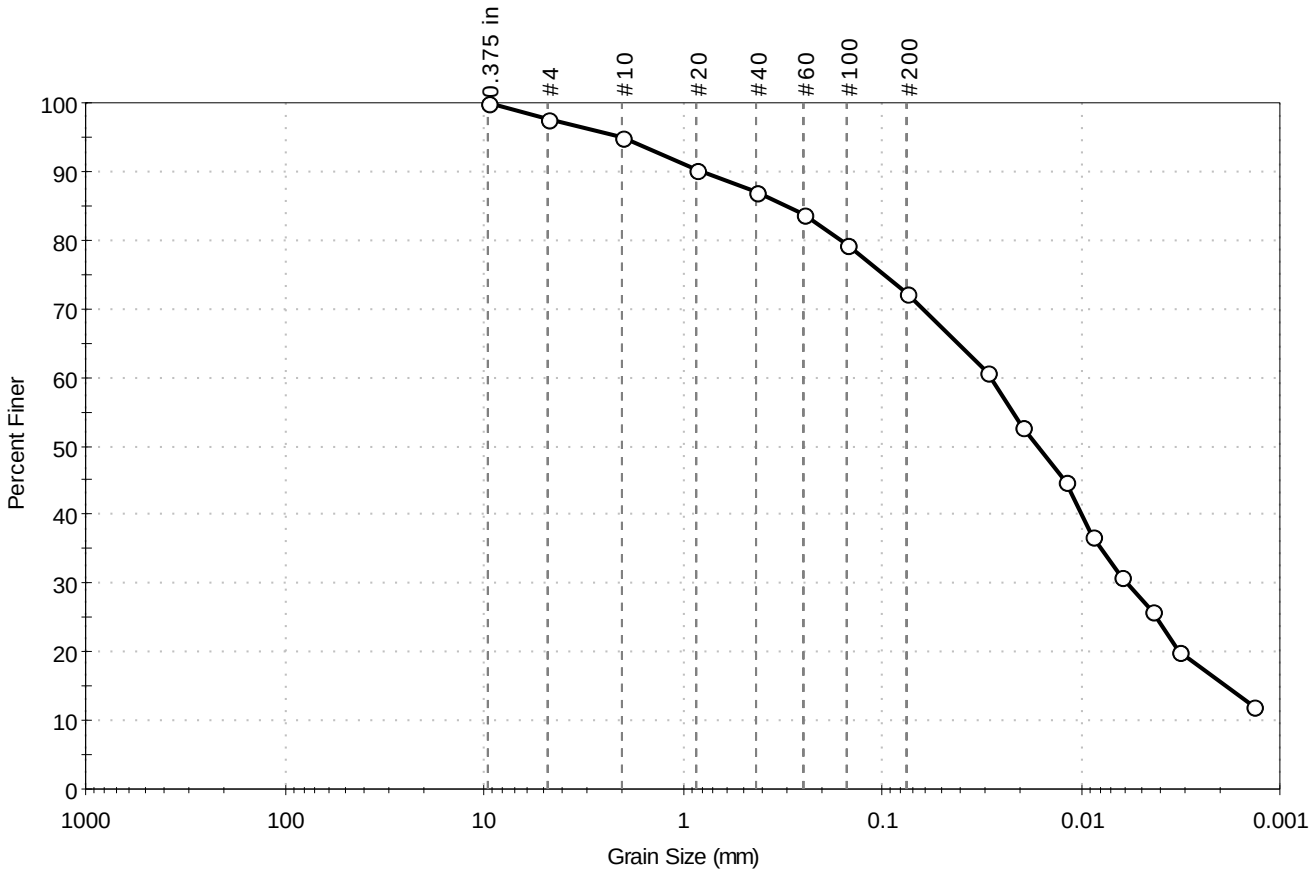
Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

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Client:	Golder Associates				
Project:	Marsh Bridge				
Location:	Tremont/Southwest Harbor, ME			Project No:	GTX-306503
Boring ID:	BB-MAR-101	Sample Type:	jar	Tested By:	jbr
Sample ID:	2	Test Date:	06/06/17	Checked By:	emm
Depth :	5-7 ft	Test Id:	412794		
Test Comment:	---				
Visual Description:	Moist, dark gray clay with sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	2.2	25.5	72.3

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	98		
#10	2.00	95		
#20	0.85	90		
#40	0.42	87		
#60	0.25	84		
#100	0.15	79		
#200	0.075	72		
---	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0299	61		
---	0.0199	53		
---	0.0121	45		
---	0.0087	37		
---	0.0062	31		
---	0.0044	26		
---	0.0032	20		
---	0.0014	12		

Coefficients

D_{85} = 0.3041 mm	D_{30} = 0.0059 mm
D_{60} = 0.0286 mm	D_{15} = 0.0019 mm
D_{50} = 0.0166 mm	D_{10} = N/A
C_u = N/A	C_c = N/A

Classification

ASTM Lean clay with sand (CL)

AASHTO Clayey Soils (A-6 (6))

Sample / Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

Dispersion Device : Apparatus A - Mech Mixer

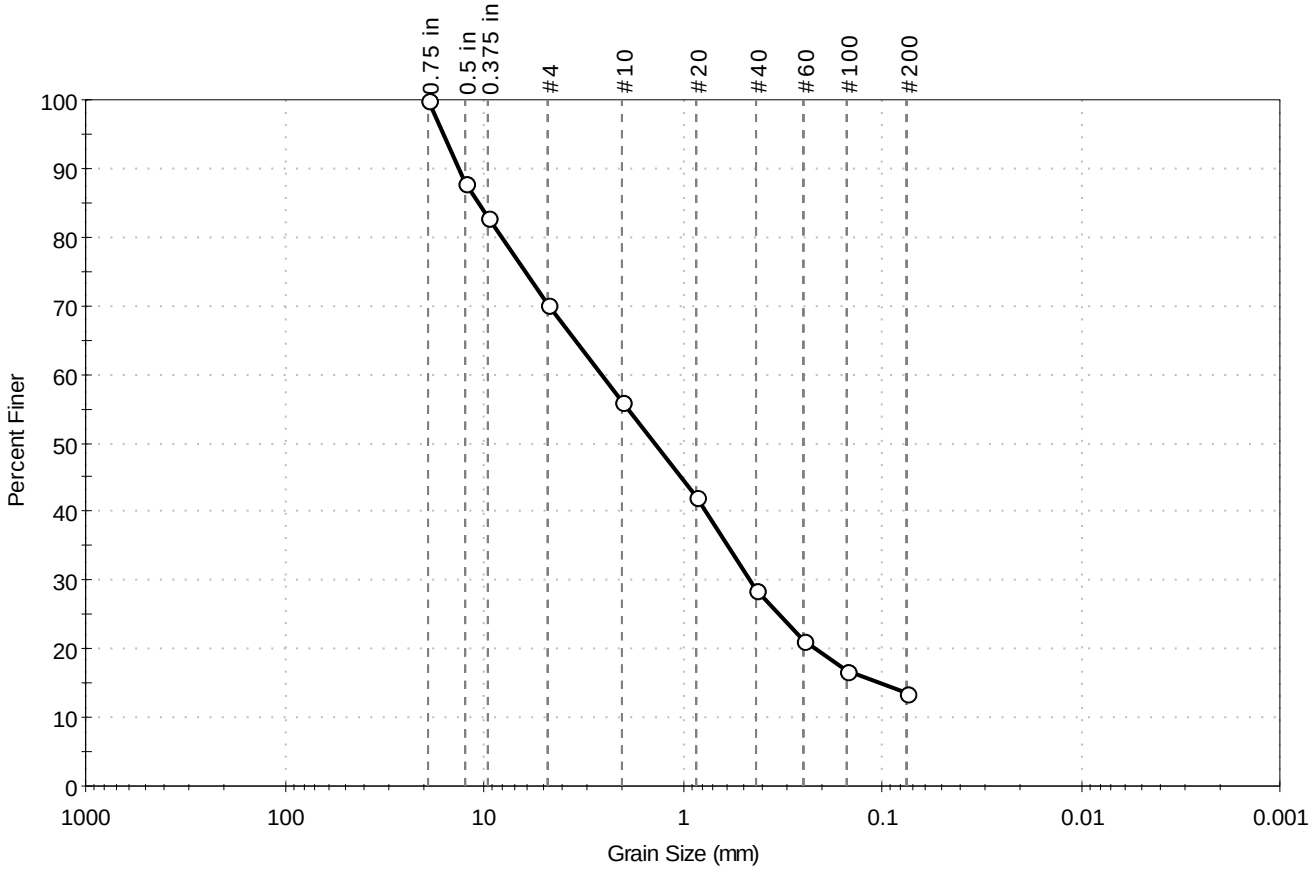
Dispersion Period : 1 minute

Specific Gravity : 2.65

Separation of Sample: #200 Sieve

Client:	Golder Associates				
Project:	Marsh Bridge				
Location:	Tremont/Southwest Harbor, ME			Project No:	GTX-306503
Boring ID:	BB-MAR-102	Sample Type:	jar	Tested By:	jbr
Sample ID:	1	Test Date:	06/02/17	Checked By:	emm
Depth :	1-3 ft	Test Id:	412792		
Test Comment:	---				
Visual Description:	Moist, dark brown silty sand and gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	29.9	56.5	13.6

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	88		
0.375 in	9.50	83		
#4	4.75	70		
#10	2.00	56		
#20	0.85	42		
#40	0.42	29		
#60	0.25	21		
#100	0.15	17		
#200	0.075	14		

Coefficients

D_{85} = 10.6959 mm
D_{30} = 0.4530 mm
D_{60} = 2.5470 mm
D_{15} = 0.1006 mm
D_{50} = 1.3739 mm
D_{10} = N/A
C_u = N/A
C_c = N/A

Classification

ASTM N/A

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

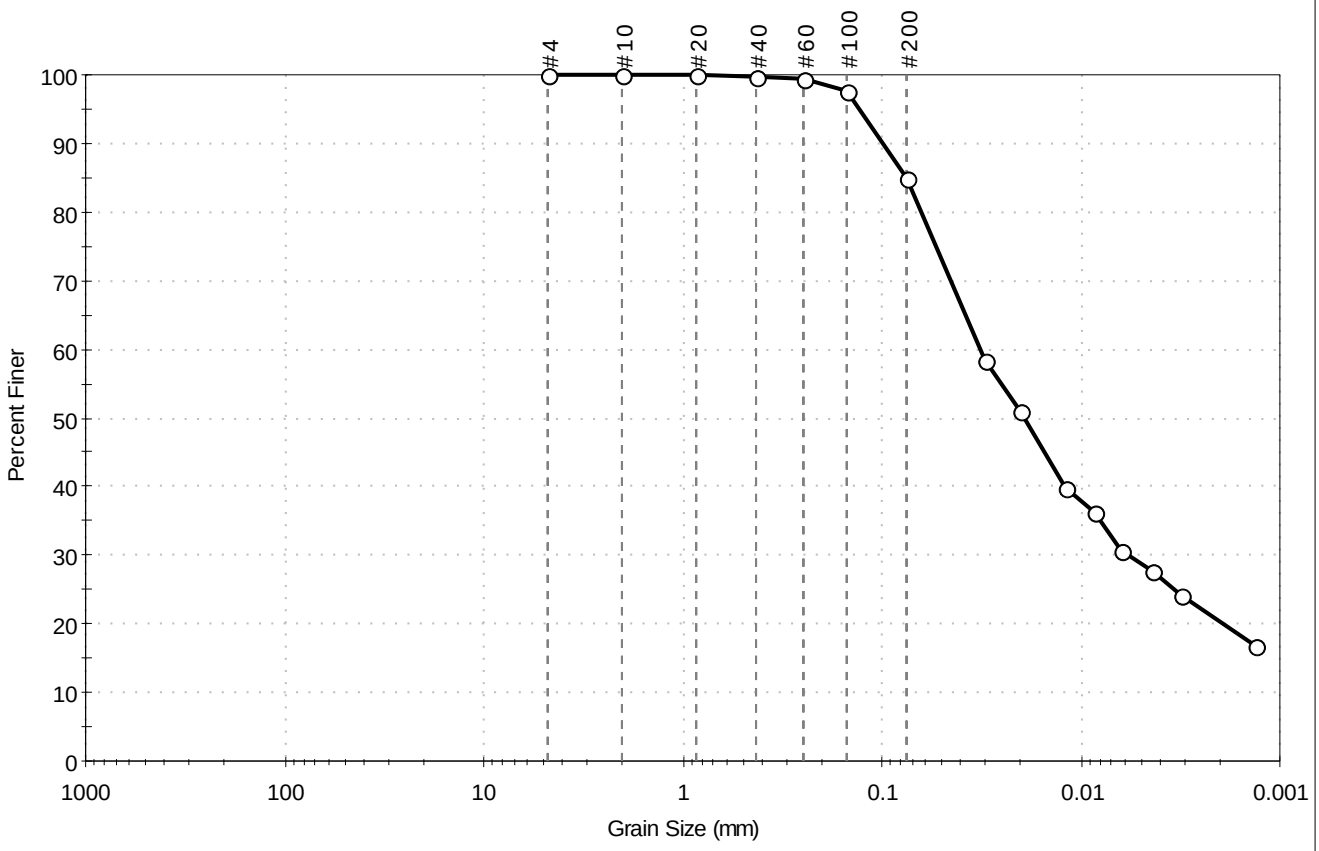
Sample / Test Description

Sand/Gravel Particle Shape : ANGULAR

Sand/Gravel Hardness : HARD

Client:	Golder Associates				
Project:	Marsh Bridge				
Location:	Tremont/Southwest Harbor, ME			Project No:	GTX-306503
Boring ID:	BB-MAR-102	Sample Type:	jar	Tested By:	jbr
Sample ID:	3	Test Date:	06/06/17	Checked By:	emm
Depth :	12-14 ft	Test Id:	412795		
Test Comment:	---				
Visual Description:	Moist, olive clay with sand				
Sample Comment:	---				

Particle Size Analysis - ASTM D422



%Cobble	%Gravel	%Sand	%Silt & Clay Size
---	0.0	15.0	85.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	100		
#60	0.25	99		
#100	0.15	98		
#200	0.075	85		
---	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0304	58		
---	0.0200	51		
---	0.0121	40		
---	0.0085	36		
---	0.0062	31		
---	0.0044	28		
---	0.0032	24		
---	0.0013	17		

Coefficients

D_{85} = 0.0750 mm	D_{30} = 0.0058 mm
D_{60} = 0.0321 mm	D_{15} = N/A
D_{50} = 0.0192 mm	D_{10} = N/A
C_u = N/A	C_c = N/A

Classification

ASTM Lean clay with sand (CL)

AASHTO Clayey Soils (A-6 (7))

Sample / Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

Dispersion Device : Apparatus A - Mech Mixer

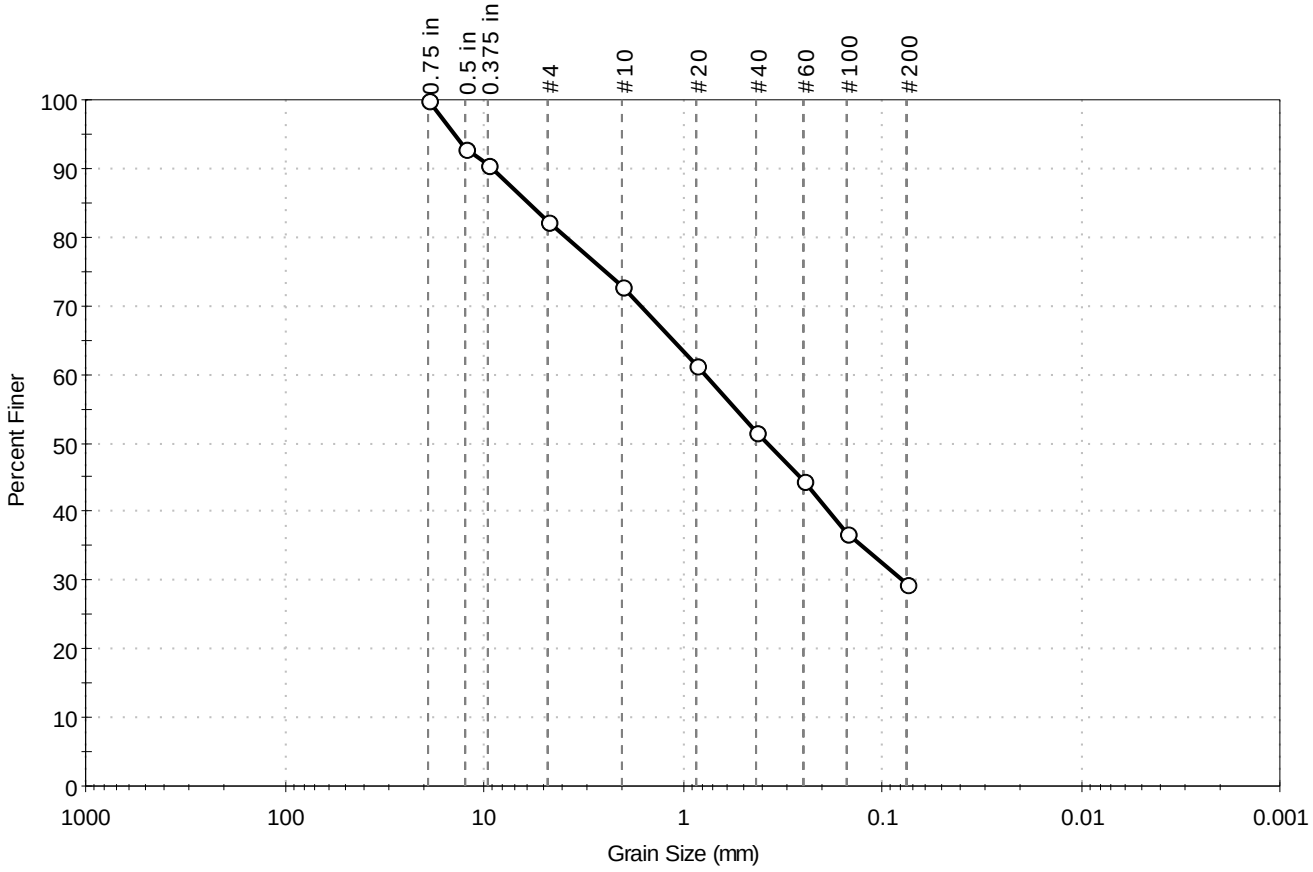
Dispersion Period : 1 minute

Specific Gravity : 2.65

Separation of Sample: #200 Sieve

Client:	Golder Associates				
Project:	Marsh Bridge				
Location:	Tremont/Southwest Harbor, ME			Project No:	GTX-306503
Boring ID:	BB-MAR-103A	Sample Type:	jar	Tested By:	jbr
Sample ID:	2	Test Date:	06/02/17	Checked By:	emm
Depth :	2.5-4.5 ft	Test Id:	412793		
Test Comment:	---				
Visual Description:	Moist, dark grayish brown silty sand and gravel				
Sample Comment:	---				

Particle Size Analysis - ASTM D422



%Cobble	%Gravel	%Sand	%Silt & Clay Size
---	17.6	52.9	29.5

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	93		
0.375 in	9.50	91		
#4	4.75	82		
#10	2.00	73		
#20	0.85	61		
#40	0.42	52		
#60	0.25	45		
#100	0.15	37		
#200	0.075	29		

Coefficients

D_{85} = 5.9275 mm D_{30} = 0.0787 mm

D_{60} = 0.7709 mm D_{15} = N/A

D_{50} = 0.3775 mm D_{10} = N/A

C_u = N/A C_c = N/A

Classification

ASTM N/A

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

Sample / Test Description

Sand/Gravel Particle Shape : ANGULAR

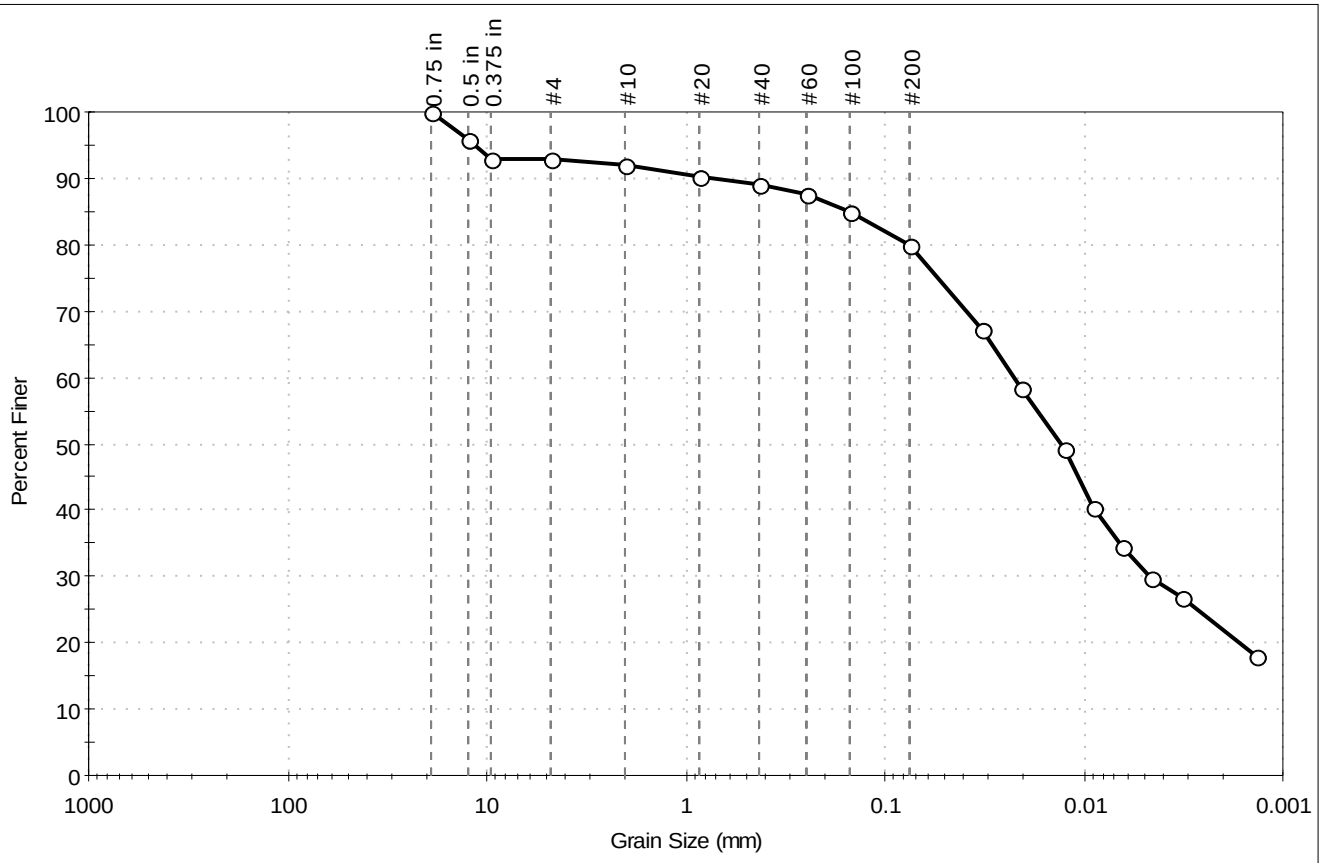
Sand/Gravel Hardness : HARD

GeoTesting EXPRESS

Client:	Golder Associates				
Project:	Marsh Bridge				
Location:	Tremont/Southwest Harbor, ME			Project No:	GTX-306503
Boring ID:	BB-MAR-103B	Sample Type:	jar	Tested By:	jbr
Sample ID:	5	Test Date:	06/06/17	Checked By:	emm
Depth :	8-10 ft	Test Id:	412796		

Test Comment: ---
Visual Description: Moist, dark gray clay with sand and organics
Sample Comment: ---

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	7.1	13.0	79.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.75 in	19.00	100		
0.5 in	12.50	96		
0.375 in	9.50	93		
#4	4.75	93		
#10	2.00	92		
#20	0.85	90		
#40	0.42	89		
#60	0.25	88		
#100	0.15	85		
#200	0.075	80		
---	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0329	67		
---	0.0209	58		
---	0.0126	49		
---	0.0089	40		
---	0.0065	34		
---	0.0046	30		
---	0.0033	27		
---	0.0014	18		

Coefficients

D_{85} = 0.1482 mm	D_{30} = 0.0046 mm
D_{60} = 0.0227 mm	D_{15} = N/A
D_{50} = 0.0130 mm	D_{10} = N/A
C_u = N/A	C_c = N/A

Classification

ASTM Lean clay with sand (CL)

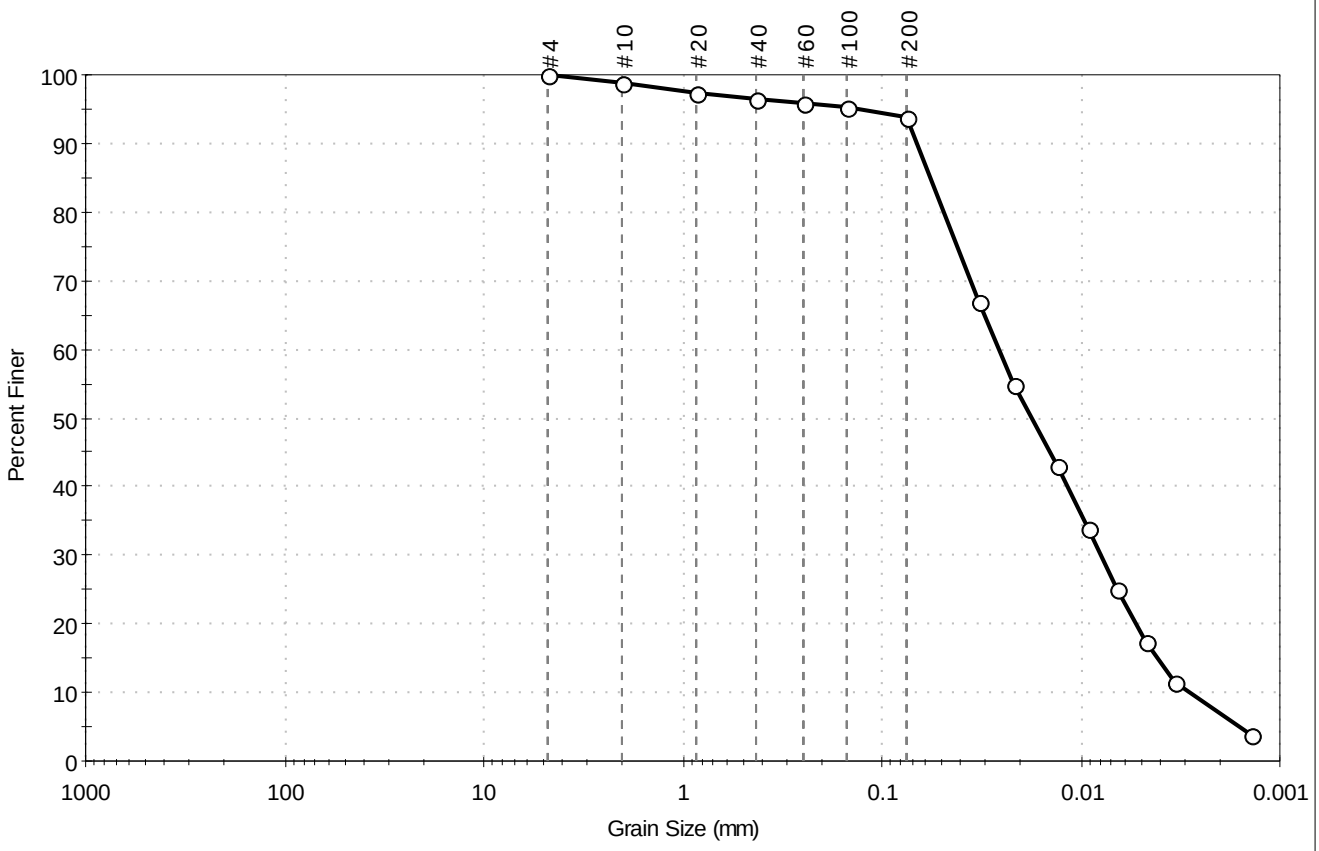
AASHTO Clayey Soils (A-6 (9))

Sample / Test Description

Sand/Gravel Particle Shape : ANGULAR
Sand/Gravel Hardness : HARD
Dispersion Device : Apparatus A - Mech Mixer
Dispersion Period : 1 minute
Specific Gravity : 2.65
Separation of Sample: #200 Sieve

Client:	Golder Associates				
Project:	Marsh Bridge				
Location:	Tremont/Southwest Harbor, ME			Project No:	GTX-306503
Boring ID:	BB-MAR-104	Sample Type:	jar	Tested By:	jbr
Sample ID:	5	Test Date:	06/06/17	Checked By:	emm
Depth :	10.5-12.5 ft	Test Id:	412797		
Test Comment:	---				
Visual Description:	Moist, very dark brown silt with organics				
Sample Comment:	---				

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	0.0	6.1	93.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	99		
#20	0.85	97		
#40	0.42	96		
#60	0.25	96		
#100	0.15	95		
#200	0.075	94		
---	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0327	67		
---	0.0220	55		
---	0.0132	43		
---	0.0093	34		
---	0.0066	25		
---	0.0047	18		
---	0.0034	12		
---	0.0014	4		

Coefficients

D_{85} = 0.0570 mm	D_{30} = 0.0080 mm
D_{60} = 0.0260 mm	D_{15} = 0.0041 mm
D_{50} = 0.0178 mm	D_{10} = 0.0028 mm
C_u = 9.286	C_c = 0.879

Classification

ASTM N/A

AASHTO Silty Soils (A-4 (0))

Sample / Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

Dispersion Device : Apparatus A - Mech Mixer

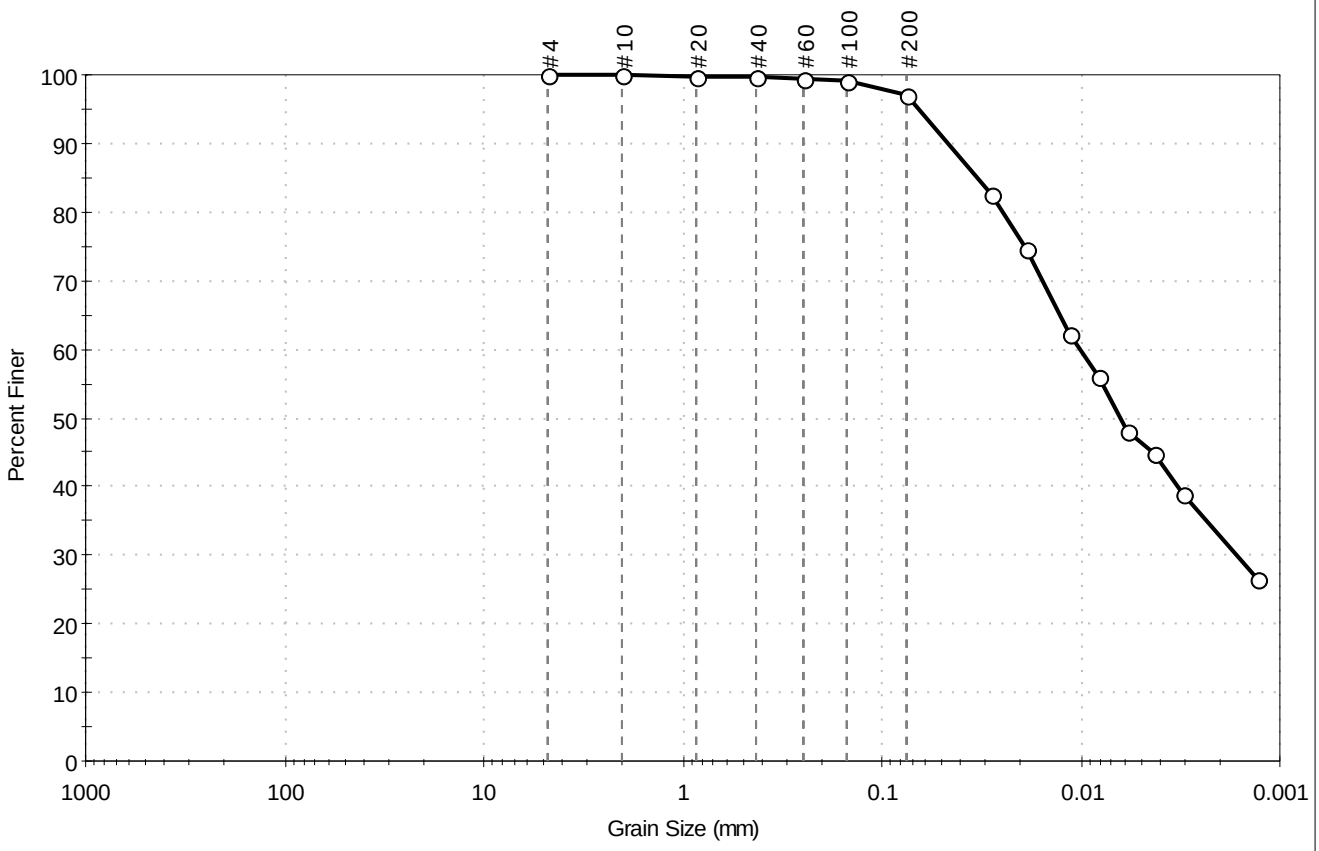
Dispersion Period : 1 minute

Specific Gravity : 2.65

Separation of Sample: #200 Sieve

Client:	Golder Associates				
Project:	Marsh Bridge				
Location:	Tremont/Southwest Harbor, ME			Project No:	GTX-306503
Boring ID:	BB-MAR-104	Sample Type:	jar	Tested By:	jbr
Sample ID:	6B	Test Date:	06/06/17	Checked By:	emm
Depth :	12.5-14.5 ft	Test Id:	412798		
Test Comment:	---				
Visual Description:	Moist, olive clay				
Sample Comment:	---				

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	0.0	3.1	96.9

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	100		
#60	0.25	100		
#100	0.15	99		
#200	0.075	97		
---	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0285	83		
---	0.0187	75		
---	0.0114	62		
---	0.0082	56		
---	0.0059	48		
---	0.0042	45		
---	0.0031	39		
---	0.0013	27		

Coefficients

D_{85} = 0.0333 mm	D_{30} = 0.0017 mm
D_{60} = 0.0101 mm	D_{15} = N/A
D_{50} = 0.0064 mm	D_{10} = N/A
C_u = N/A	C_c = N/A

Classification

ASTM Lean clay (CL)

AASHTO Clayey Soils (A-6 (17))

Sample / Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

Dispersion Device : Apparatus A - Mech Mixer

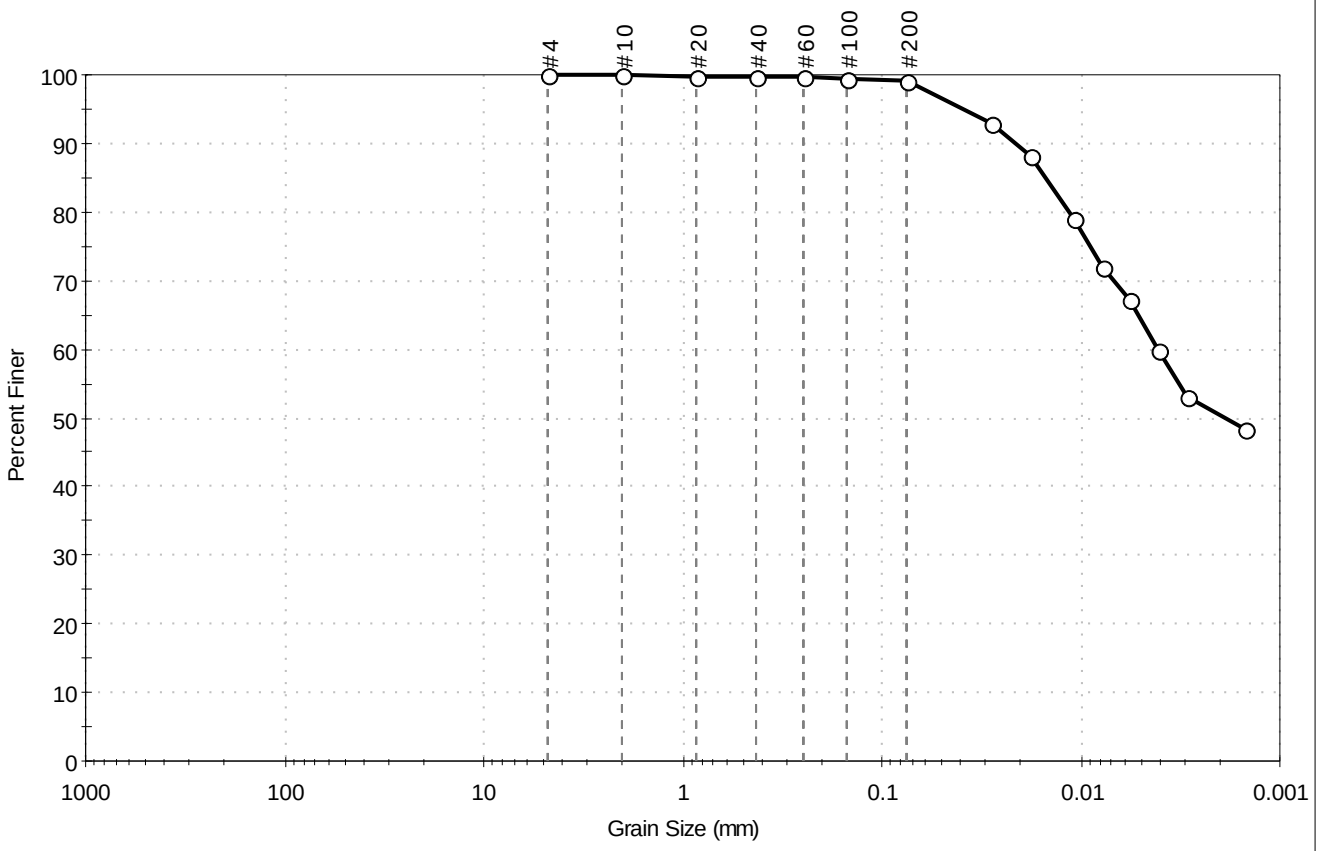
Dispersion Period : 1 minute

Specific Gravity : 2.65

Separation of Sample: #200 Sieve

Client:	Golder Associates				
Project:	Marsh Bridge				
Location:	Tremont/Southwest Harbor, ME			Project No:	GTX-306503
Boring ID:	BB-MAR-104	Sample Type:	jar	Tested By:	jbr
Sample ID:	9	Test Date:	06/06/17	Checked By:	emm
Depth :	25-27 ft	Test Id:	412799		
Test Comment:	---				
Visual Description:	Moist, olive clay				
Sample Comment:	---				

Particle Size Analysis - ASTM D422



% Cobble	% Gravel	% Sand	% Silt & Clay Size
---	0.0	1.0	99.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
#4	4.75	100		
#10	2.00	100		
#20	0.85	100		
#40	0.42	100		
#60	0.25	100		
#100	0.15	99		
#200	0.075	99		
---	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0284	93		
---	0.0179	88		
---	0.0109	79		
---	0.0079	72		
---	0.0056	67		
---	0.0041	60		
---	0.0030	53		
---	0.0015	48		

Coefficients

D_{85} = 0.0151 mm	D_{30} = N/A
D_{60} = 0.0041 mm	D_{15} = N/A
D_{50} = 0.0019 mm	D_{10} = N/A
C_u = N/A	C_c = N/A

Classification

ASTM Lean clay (CL)

AASHTO Clayey Soils (A-7-6 (26))

Sample / Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

Dispersion Device : Apparatus A - Mech Mixer

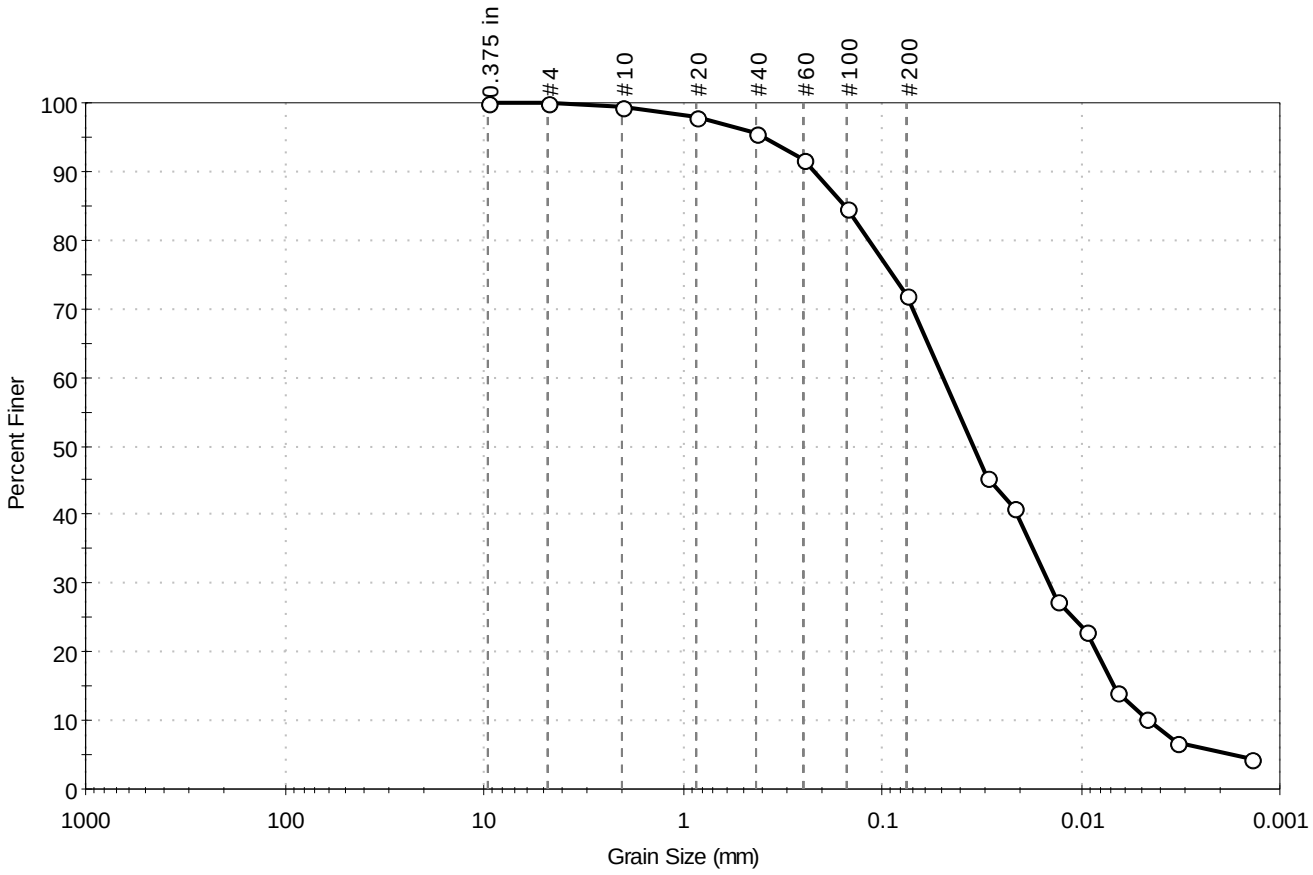
Dispersion Period : 1 minute

Specific Gravity : 2.65

Separation of Sample: #200 Sieve

Client:	Golder Associates				
Project:	Marsh Bridge				
Location:	Tremont/Southwest Harbor, ME			Project No:	GTX-306503
Boring ID:	BB-MAR-203A	Sample Type:	tube	Tested By:	jbr
Sample ID:	U1	Test Date:	11/06/17	Checked By:	emm
Depth :	6.5-8.5 ft	Test Id:	428420		
Test Comment:	---				
Visual Description:	Wet, very dark brown silt with sand and organics				
Sample Comment:	---				

Particle Size Analysis - ASTM D422



%Cobble	%Gravel	%Sand	%Silt & Clay Size
---	0.0	28.0	72.0

Sieve Name	Sieve Size, mm	Percent Finer	Spec. Percent	Complies
0.375 in	9.50	100		
#4	4.75	100		
#10	2.00	100		
#20	0.85	98		
#40	0.42	96		
#60	0.25	92		
#100	0.15	85		
#200	0.075	72		
---	Particle Size (mm)	Percent Finer	Spec. Percent	Complies
---	0.0294	45		
---	0.0215	41		
---	0.0131	27		
---	0.0094	23		
---	0.0067	14		
---	0.0047	10		
---	0.0033	7		
---	0.0014	4		

Coefficients

D_{85} = 0.1545 mm	D_{30} = 0.0144 mm
D_{60} = 0.0493 mm	D_{15} = 0.0069 mm
D_{50} = 0.0347 mm	D_{10} = 0.0045 mm
C_u = 10.956	C_c = 0.935

Classification

ASTM Elastic silt with sand (MH)

AASHTO Clayey Soils (A-7-5 (130))

Sample / Test Description

Sand/Gravel Particle Shape : ---

Sand/Gravel Hardness : ---

Dispersion Device : Apparatus A - Mech Mixer

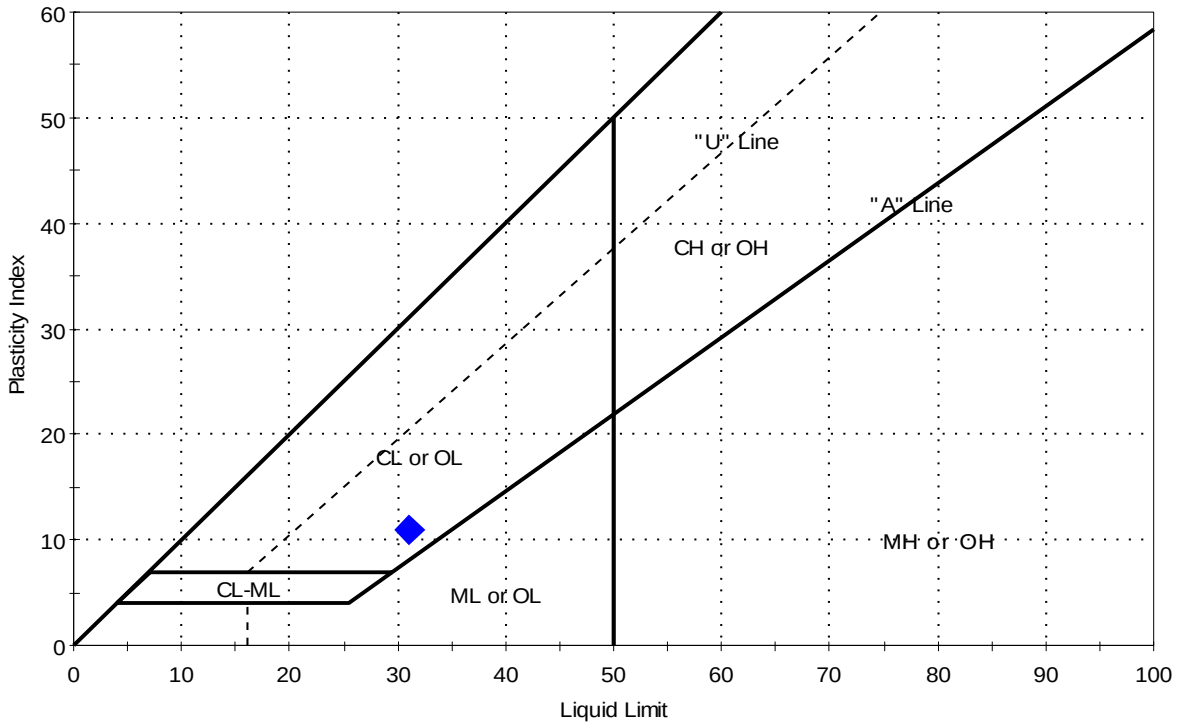
Dispersion Period : 1 minute

Specific Gravity : 2.65

Separation of Sample: #200 Sieve

Client:	Golder Associates				
Project:	Marsh Bridge				
Location:	Tremont/Southwest Harbor, ME			Project No:	GTX-306503
Boring ID:	BB-MAR-101	Sample Type:	jar	Tested By:	cam
Sample ID:	2	Test Date:	06/05/17	Checked By:	emm
Depth :	5-7 ft	Test Id:	412781		
Test Comment:	---				
Visual Description:	Moist, dark gray clay with sand				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



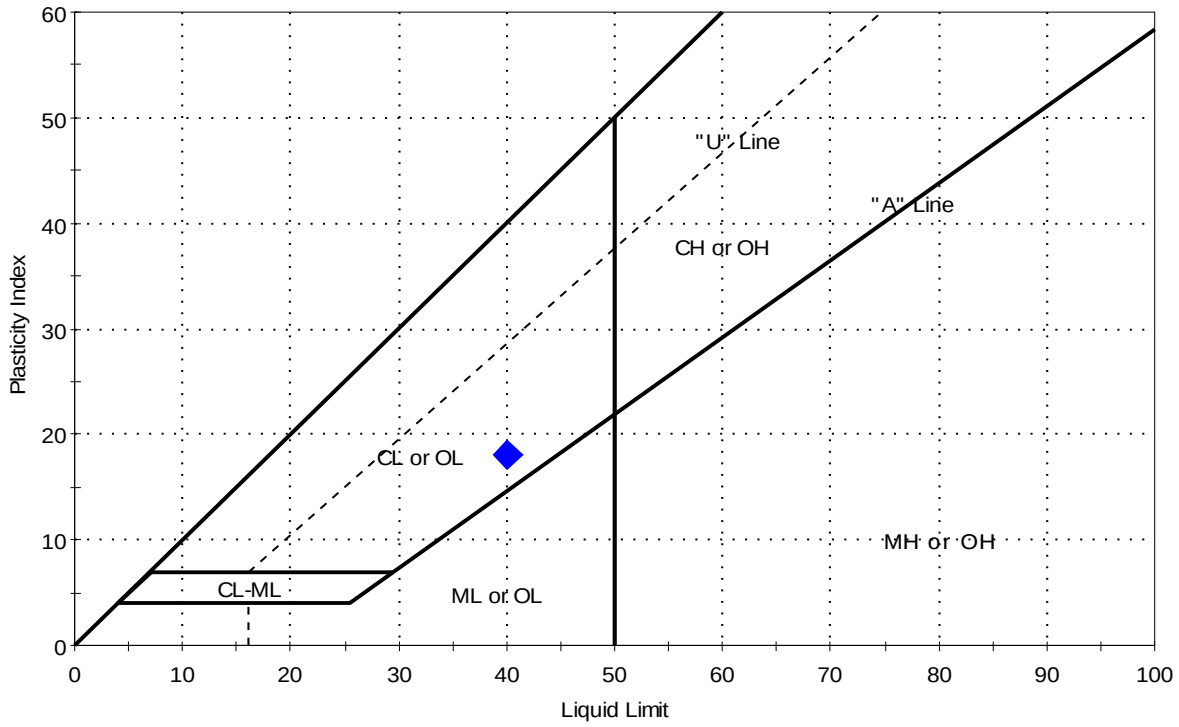
Symbol	Sample ID	Boring	Depth	Natural Moisture Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	2	B-MAR-10	5-7 ft	24	31	20	11	0.3	Lean clay with sand (CL)

Sample Prepared using the WET method
13% Retained on #40 Sieve
Dry Strength: VERY HIGH
Dilatancy: SLOW
Toughness: LOW

Client:	Golder Associates				
Project:	Marsh Bridge				
Location:	Tremont/Southwest Harbor, ME			Project No:	GTX-306503
Boring ID:	BB-MAR-102	Sample Type:	jar	Tested By:	cam
Sample ID:	2	Test Date:	06/06/17	Checked By:	emm
Depth :	5-7 ft	Test Id:	412782		
Test Comment:	---				
Visual Description:	Moist, very dark brown clay with gravel				
Sample Comment:	---				

Atterberg Limits - ASTM D4318

Plasticity Chart



Symbol	Sample ID	Boring	Depth	Natural Moisture Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	2	B-MAR-10	5-7 ft	28	40	22	18	0.3	

Sample Prepared using the WET method

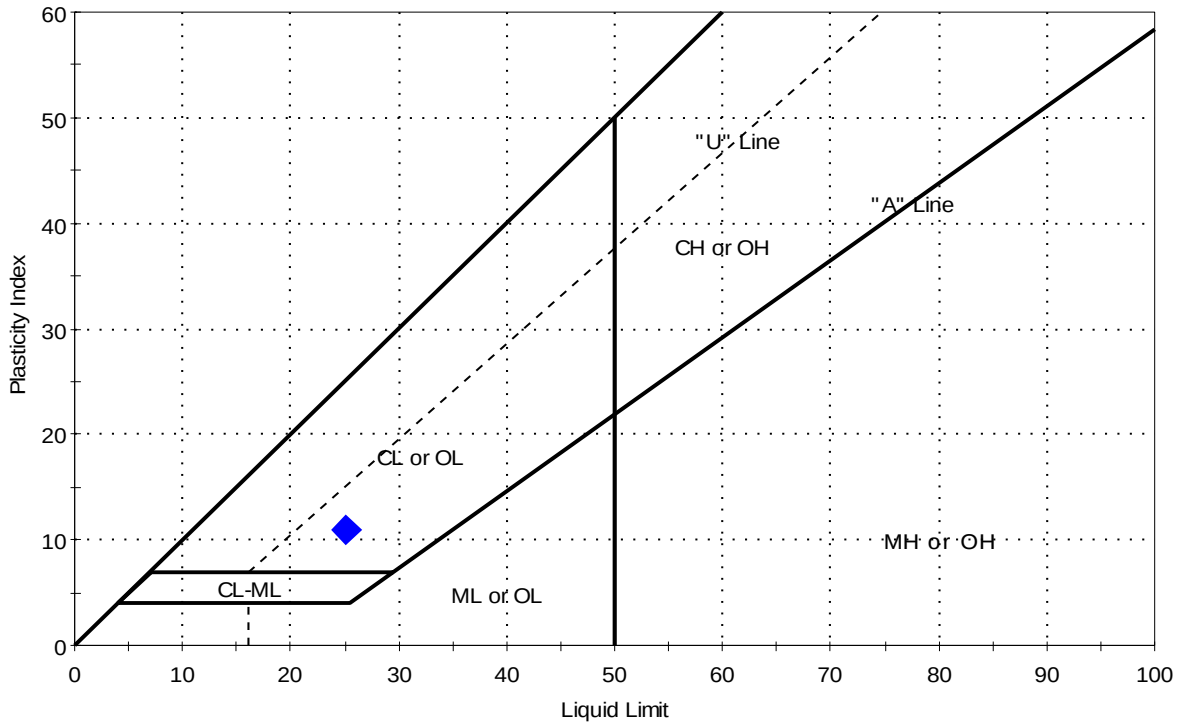
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	Golder Associates				
Project:	Marsh Bridge				
Location:	Tremont/Southwest Harbor, ME			Project No:	GTX-306503
Boring ID:	BB-MAR-102	Sample Type:	jar	Tested By:	cam
Sample ID:	3	Test Date:	06/05/17	Checked By:	emm
Depth :	12-14 ft	Test Id:	412783		
Test Comment:	---				
Visual Description:	Moist, olive clay with sand				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



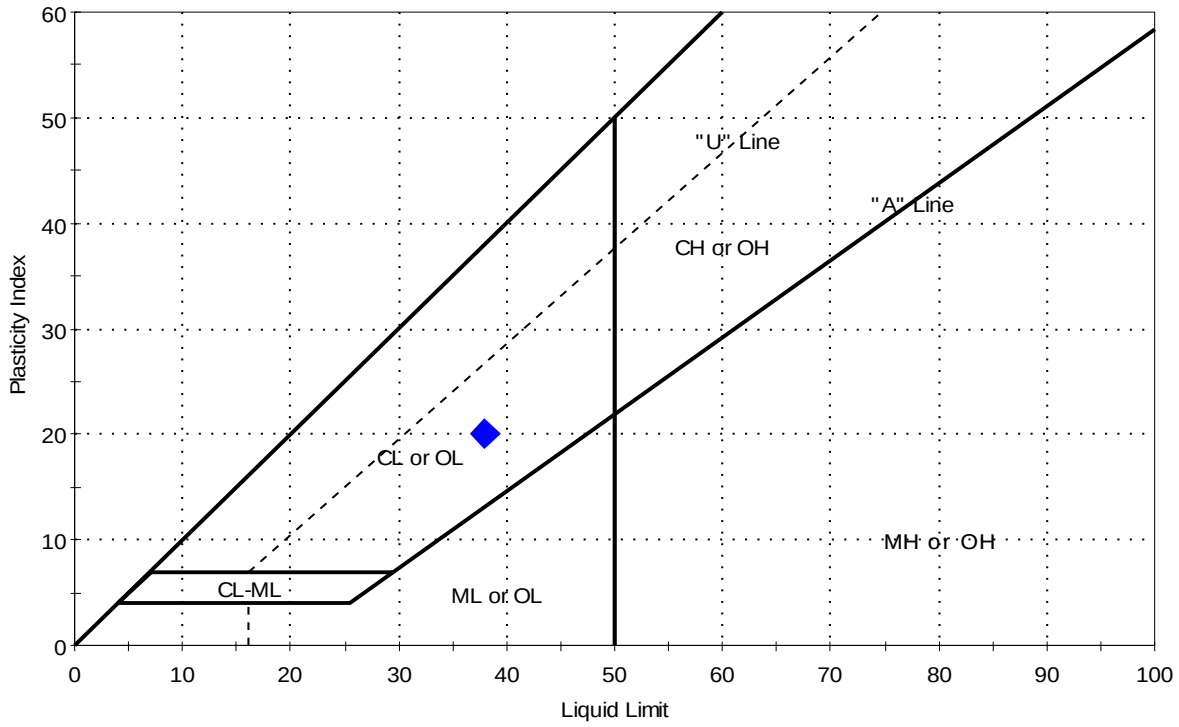
Symbol	Sample ID	Boring	Depth	Natural Moisture Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	3	B-MAR-10	12-14 ft	20	25	14	11	0.5	Lean clay with sand (CL)

Sample Prepared using the WET method
0% Retained on #40 Sieve
Dry Strength: VERY HIGH
Dilatancy: SLOW
Toughness: LOW

Client:	Golder Associates				
Project:	Marsh Bridge				
Location:	Tremont/Southwest Harbor, ME			Project No:	GTX-306503
Boring ID:	BB-MAR-102	Sample Type:	jar	Tested By:	cam
Sample ID:	4	Test Date:	06/02/17	Checked By:	emm
Depth :	15-17 ft	Test Id:	412784		
Test Comment:	---				
Visual Description:	Moist, olive clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318

Plasticity Chart



Symbol	Sample ID	Boring	Depth	Natural Moisture Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	4	B-MAR-10	15-17 ft	26	38	18	20	0.4	

Sample Prepared using the WET method

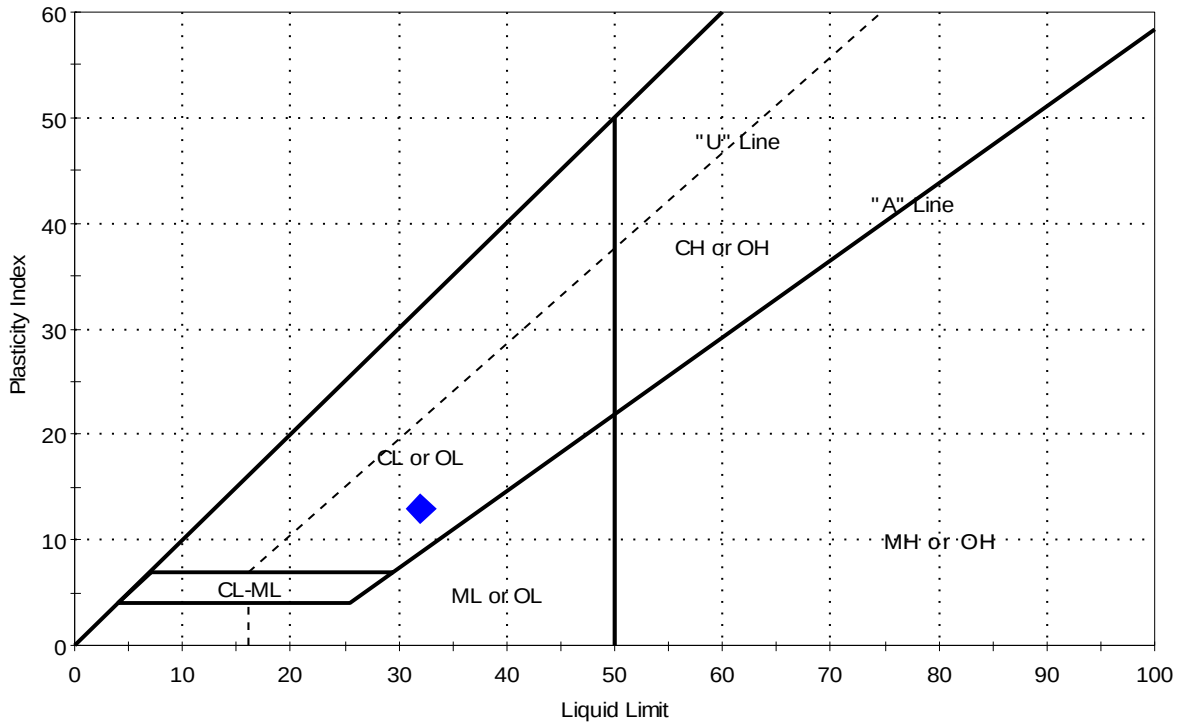
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	Golder Associates				
Project:	Marsh Bridge				
Location:	Tremont/Southwest Harbor, ME			Project No:	GTX-306503
Boring ID:	BB-MAR-103B	Sample Type:	jar	Tested By:	cam
Sample ID:	5	Test Date:	06/06/17	Checked By:	emm
Depth :	8-10 ft	Test Id:	412785		
Test Comment:	---				
Visual Description:	Moist, dark gray clay with sand and organics				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	5	-MAR-10	8-10 ft	62	32	19	13	3.3	Lean clay with sand (CL)

Sample Prepared using the WET method

11% Retained on #40 Sieve

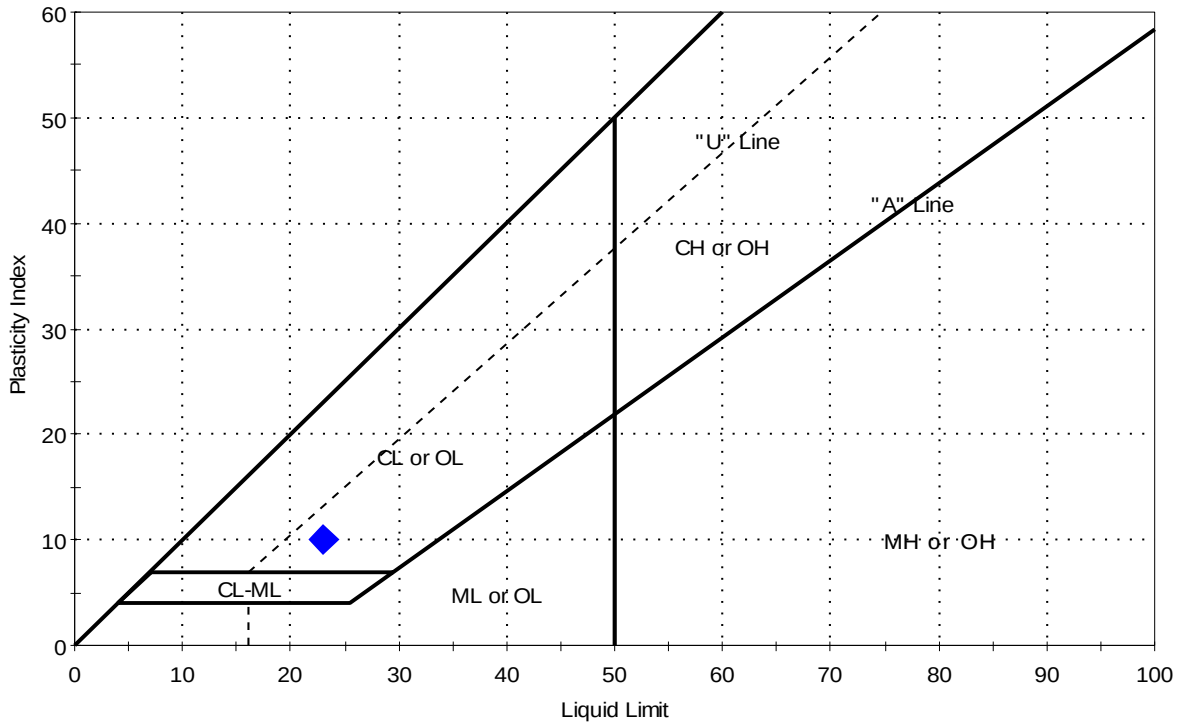
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	Golder Associates				
Project:	Marsh Bridge				
Location:	Tremont/Southwest Harbor, ME			Project No:	GTX-306503
Boring ID:	BB-MAR-103B	Sample Type:	jar	Tested By:	cam
Sample ID:	10	Test Date:	06/06/17	Checked By:	emm
Depth :	18-20 ft	Test Id:	412786		
Test Comment:	---				
Visual Description:	Moist, olive clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	10	-MAR-10	18-20 ft	25	23	13	10	1.2	

Sample Prepared using the WET method

Dry Strength: VERY HIGH

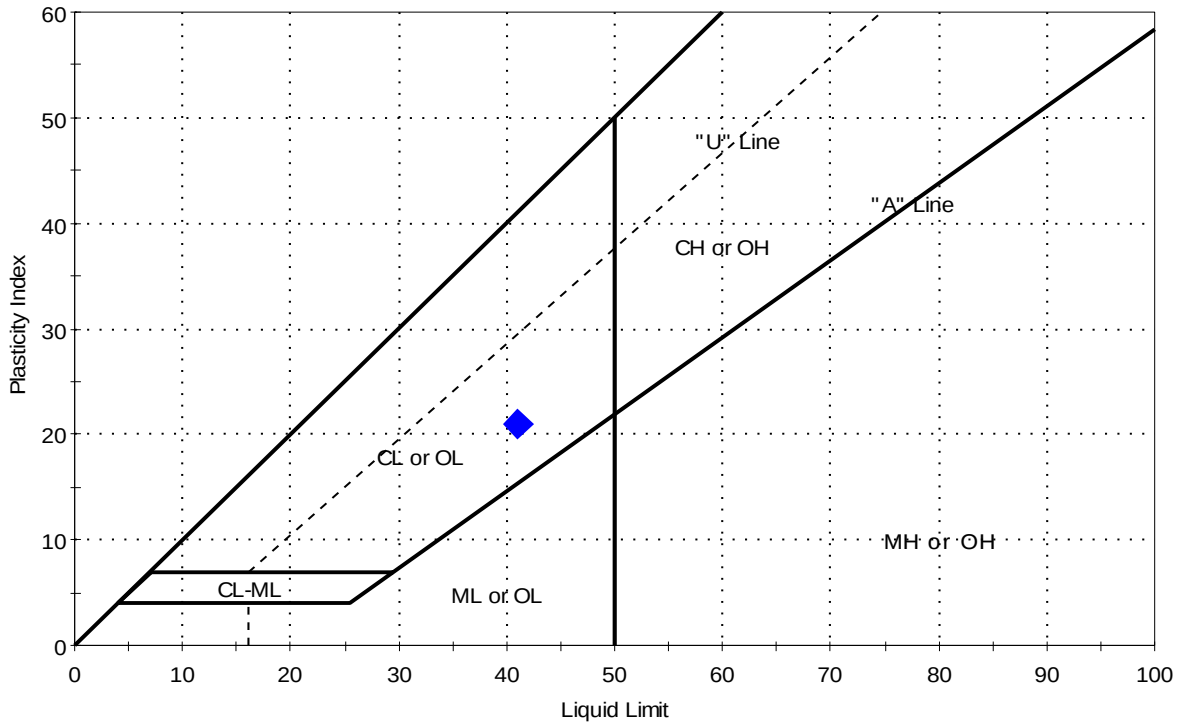
Dilatancy: SLOW

Toughness: LOW

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Client:	Golder Associates				
Project:	Marsh Bridge				
Location:	Tremont/Southwest Harbor, ME			Project No:	GTX-306503
Boring ID:	BB-MAR-103B	Sample Type:	jar	Tested By:	cam
Sample ID:	12	Test Date:	06/05/17	Checked By:	emm
Depth :	22-24 ft	Test Id:	412787		
Test Comment:	---				
Visual Description:	Moist, olive clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	12	-MAR-10	22-24 ft	42	41	20	21	1	

Sample Prepared using the WET method

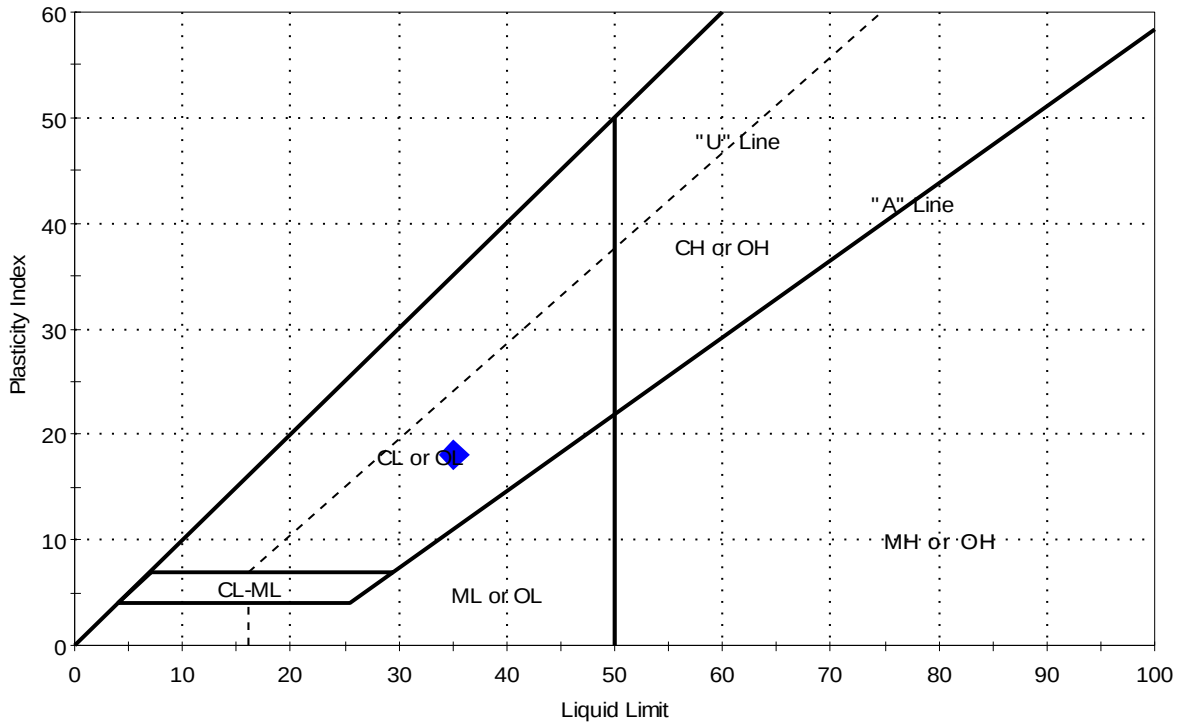
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	Golder Associates				
Project:	Marsh Bridge				
Location:	Tremont/Southwest Harbor, ME			Project No:	GTX-306503
Boring ID:	BB-MAR-104	Sample Type:	jar	Tested By:	cam
Sample ID:	6B	Test Date:	06/05/17	Checked By:	emm
Depth :	12.5-14.5 ft	Test Id:	412788		
Test Comment:	---				
Visual Description:	Moist, olive clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	6B	B-MAR-10	12.5-14.5 ft	28	35	17	18	0.6	Lean clay (CL)

Sample Prepared using the WET method

0% Retained on #40 Sieve

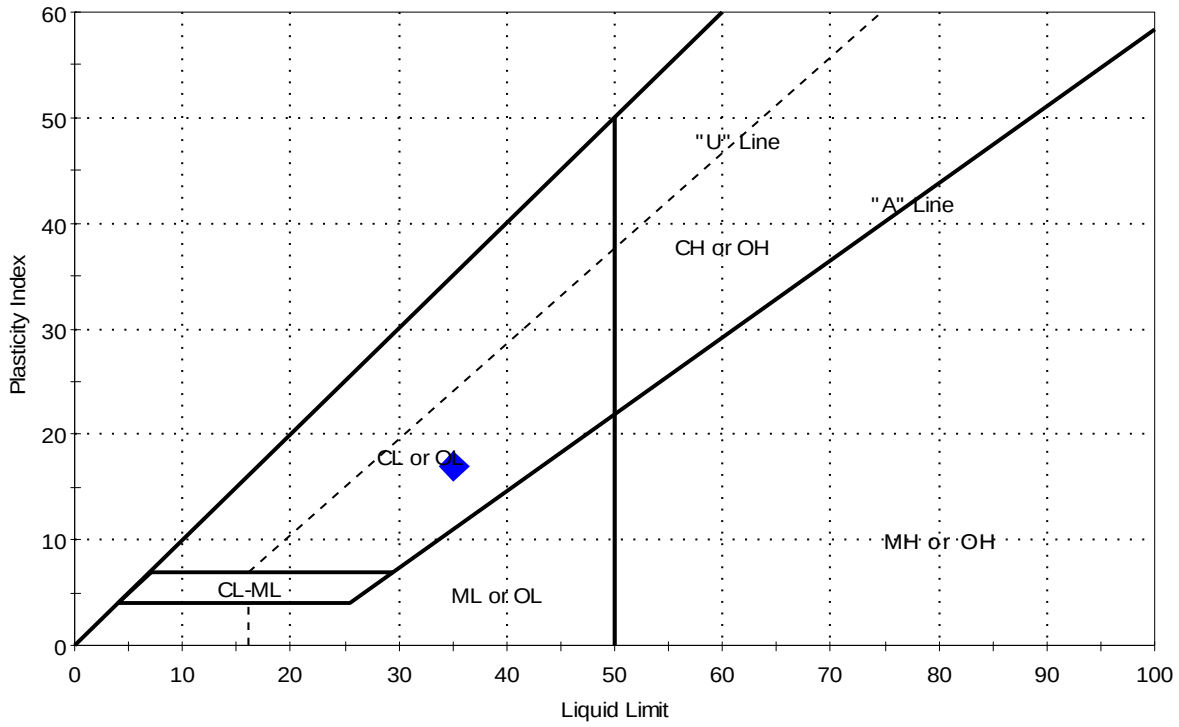
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	Golder Associates				
Project:	Marsh Bridge				
Location:	Tremont/Southwest Harbor, ME			Project No:	GTX-306503
Boring ID:	BB-MAR-104	Sample Type:	jar	Tested By:	cam
Sample ID:	8	Test Date:	06/05/17	Checked By:	emm
Depth :	17-19 ft	Test Id:	412789		
Test Comment:	---				
Visual Description:	Moist, olive clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	8	B-MAR-10	17-19 ft	36	35	18	17	1.1	

Sample Prepared using the WET method

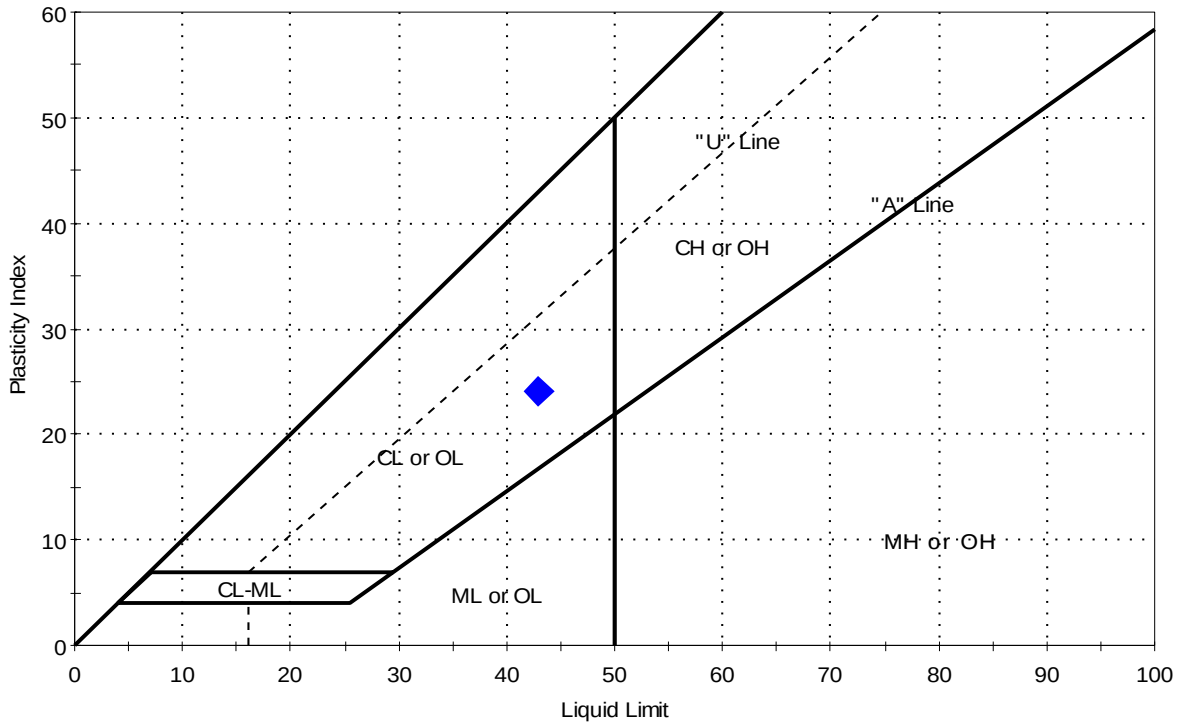
Dry Strength: VERY HIGH

Dilatancy: SLOW

Toughness: LOW

Client:	Golder Associates				
Project:	Marsh Bridge				
Location:	Tremont/Southwest Harbor, ME			Project No:	GTX-306503
Boring ID:	BB-MAR-104	Sample Type:	jar	Tested By:	cam
Sample ID:	9	Test Date:	06/05/17	Checked By:	emm
Depth :	25-27 ft	Test Id:	412790		
Test Comment:	---				
Visual Description:	Moist, olive clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



Symbol	Sample ID	Boring	Depth	Natural Moisture Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	9	B-MAR-10	25-27 ft	44	43	19	24	1	Lean clay (CL)

Sample Prepared using the WET method

0% Retained on #40 Sieve

Dry Strength: VERY HIGH

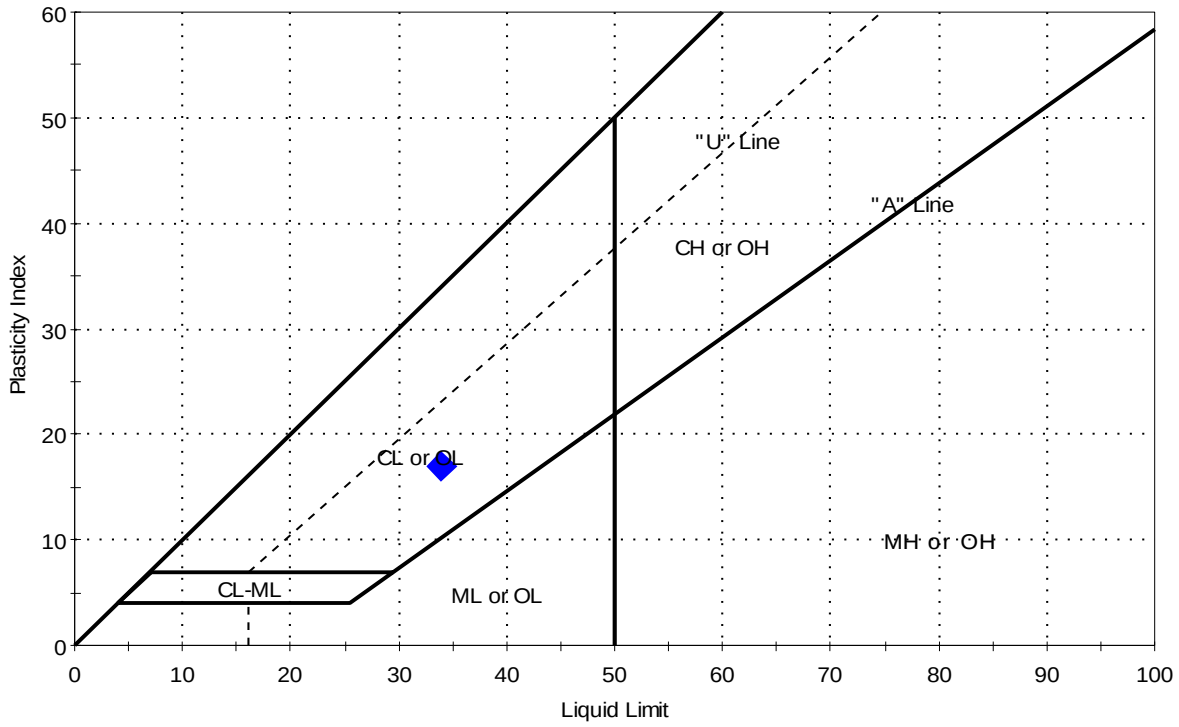
Dilatancy: SLOW

Toughness: LOW

Client:	Golder Associates				
Project:	Marsh Bridge				
Location:	Tremont/Southwest Harbor, ME			Project No:	GTX-306503
Boring ID:	BB-MAR-202	Sample Type:	jar	Tested By:	GA
Sample ID:	6D (2/2)	Test Date:	10/19/17	Checked By:	emm
Depth :	15-17 ft	Test Id:	428412		
Test Comment:	---				
Visual Description:	Moist, gray clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318

Plasticity Chart



Symbol	Sample ID	Boring	Depth	Natural Moisture Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	6D (2/2)	B-MAR-20	15-17 ft	25	34	17	17	0.5	

Sample Prepared using the WET method

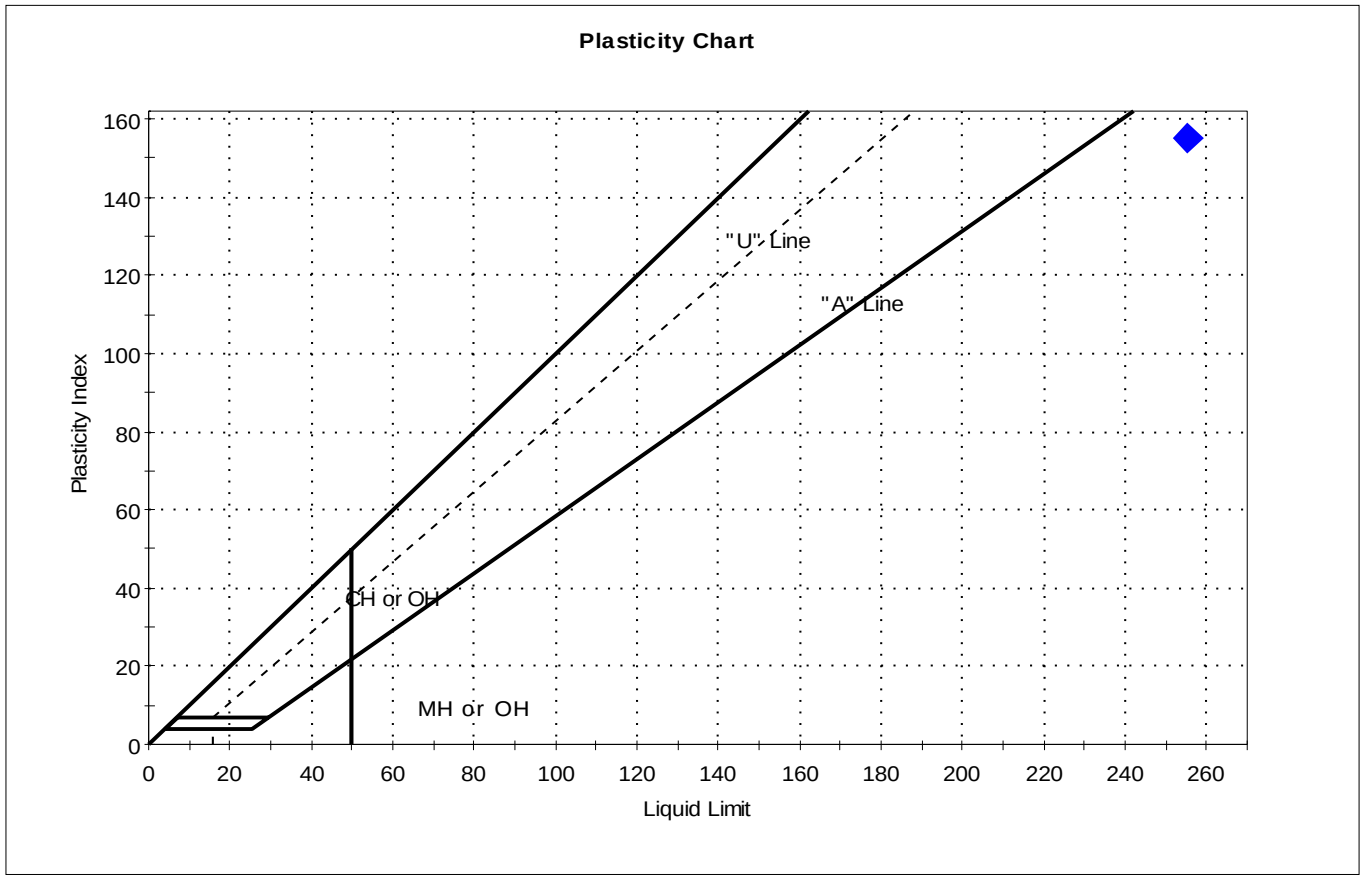
Dry Strength: MEDIUM

Dilatancy: NONE

Toughness: MEDIUM

Client:	Golder Associates				
Project:	Marsh Bridge				
Location:	Tremont/Southwest Harbor, ME			Project No:	GTX-306503
Boring ID:	BB-MAR-203A	Sample Type:	tube	Tested By:	cam
Sample ID:	U1	Test Date:	11/02/17	Checked By:	emm
Depth :	6.5-8.5 ft	Test Id:	428415		
Test Comment:	---				
Visual Description:	Wet, very dark brown silt with sand and organics				
Sample Comment:	---				

Atterberg Limits - ASTM D4318



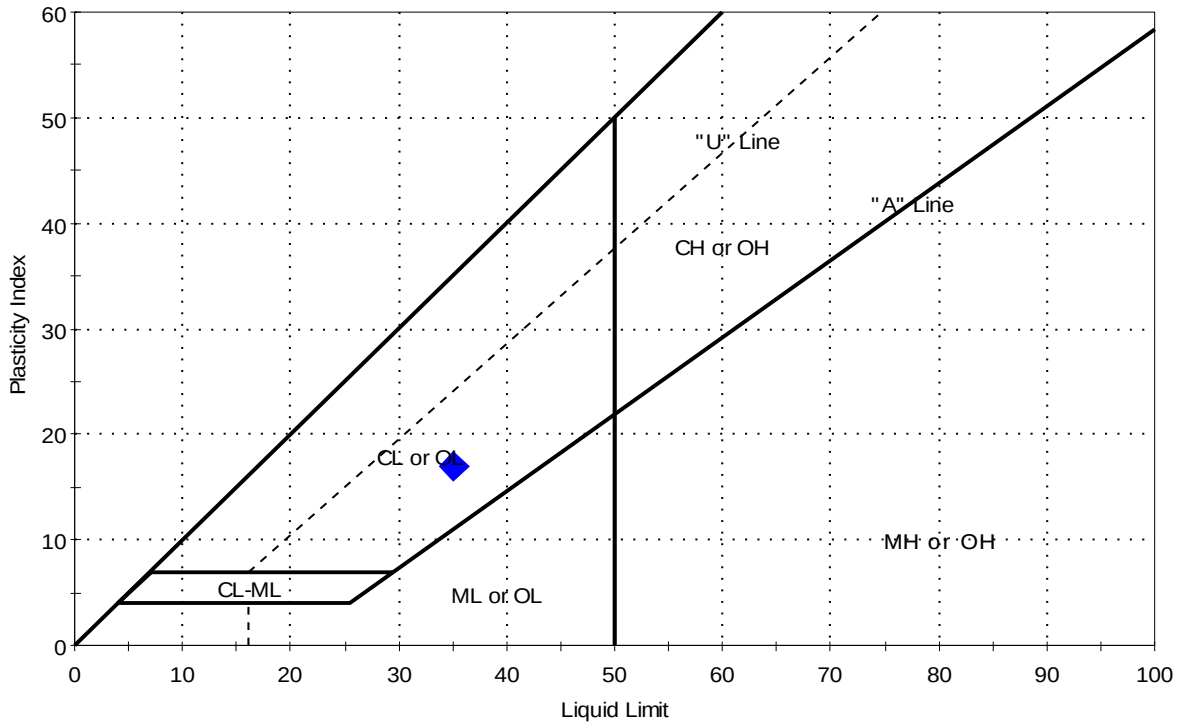
Symbol	Sample ID	Boring	Depth	Natural Moisture Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	U1	-MAR-20	6.5-8.5 ft	286	255	100	155	1.2	Elastic silt with sand (MH)

Sample Prepared using the WET method
4% Retained on #40 Sieve
Dry Strength: VERY HIGH
Dilatancy: SLOW
Toughness: LOW

Client:	Golder Associates				
Project:	Marsh Bridge				
Location:	Tremont/Southwest Harbor, ME			Project No:	GTX-306503
Boring ID:	BB-MAR-203A	Sample Type:	jar	Tested By:	GA
Sample ID:	4D	Test Date:	10/19/17	Checked By:	emm
Depth :	11.5-13.5 ft	Test Id:	428413		
Test Comment:	---				
Visual Description:	Moist, dark gray clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318

Plasticity Chart



Symbol	Sample ID	Boring	Depth	Natural Moisture Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	4D	-MAR-20	11.5-13.5 ft	32	35	18	17	0.8	

Sample Prepared using the WET method

Dry Strength: HIGH

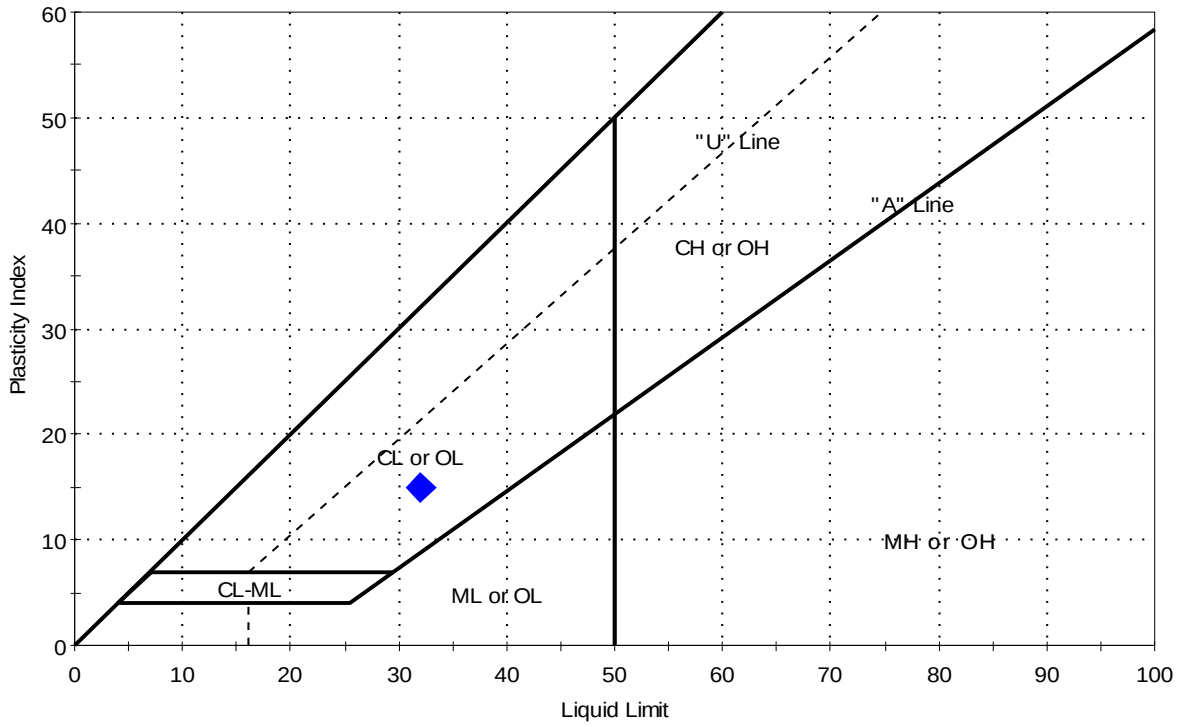
Dilatancy: NONE

Toughness: MEDIUM

Client:	Golder Associates				
Project:	Marsh Bridge				
Location:	Tremont/Southwest Harbor, ME			Project No:	GTX-306503
Boring ID:	BB-MAR-205	Sample Type:	jar	Tested By:	GA
Sample ID:	4DB	Test Date:	10/19/17	Checked By:	emm
Depth :	8-10 ft	Test Id:	428414		
Test Comment:	---				
Visual Description:	Moist, gray clay				
Sample Comment:	---				

Atterberg Limits - ASTM D4318

Plasticity Chart



Symbol	Sample ID	Boring	Depth	Natural Moisture Content, %	Liquid Limit	Plastic Limit	Plasticity Index	Liquidity Index	Soil Classification
◆	4DB	B-MAR-20	8-10 ft	25	32	17	15	0.5	

Sample Prepared using the WET method

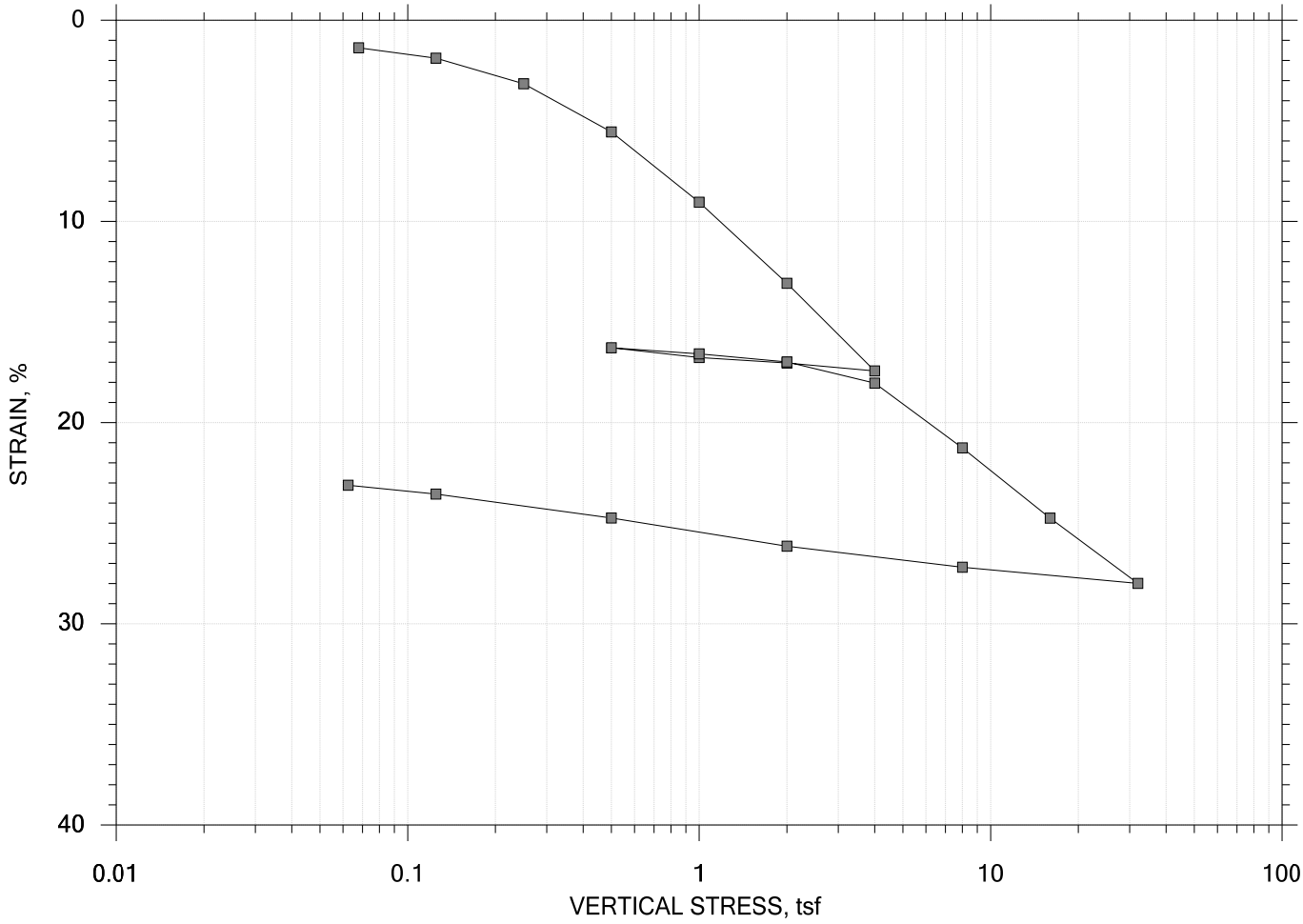
Dry Strength: HIGH

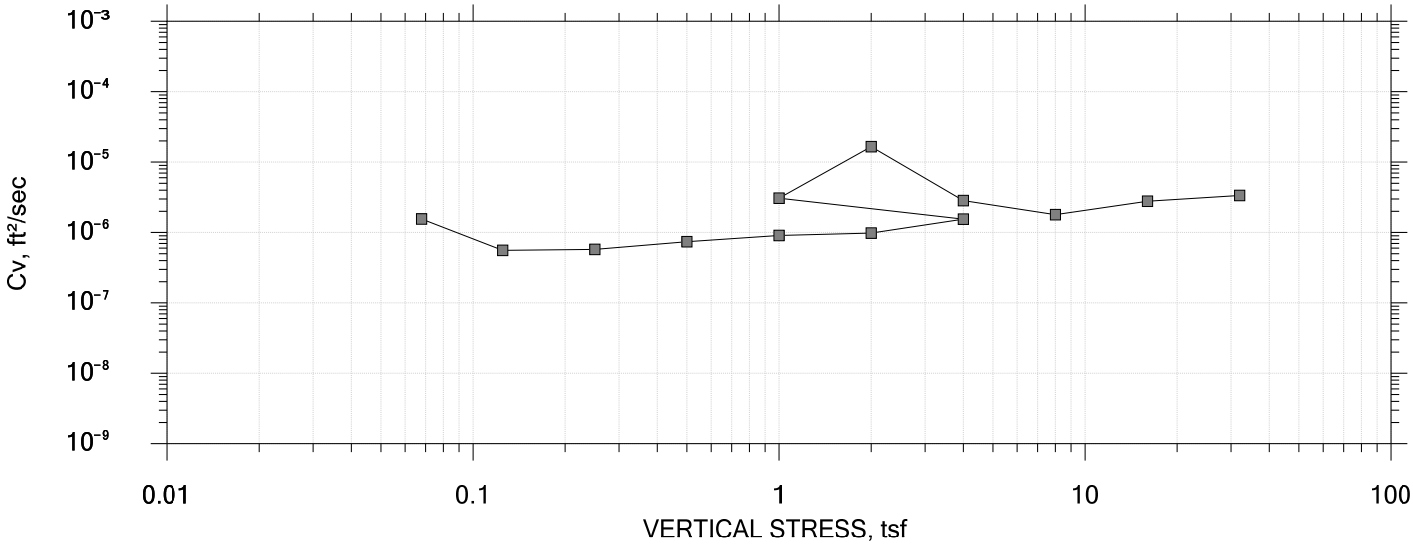
Dilatancy: NONE

Toughness: MEDIUM

One-Dimensional Consolidation by ASTM D2435 - Method B

SUMMARY REPORT

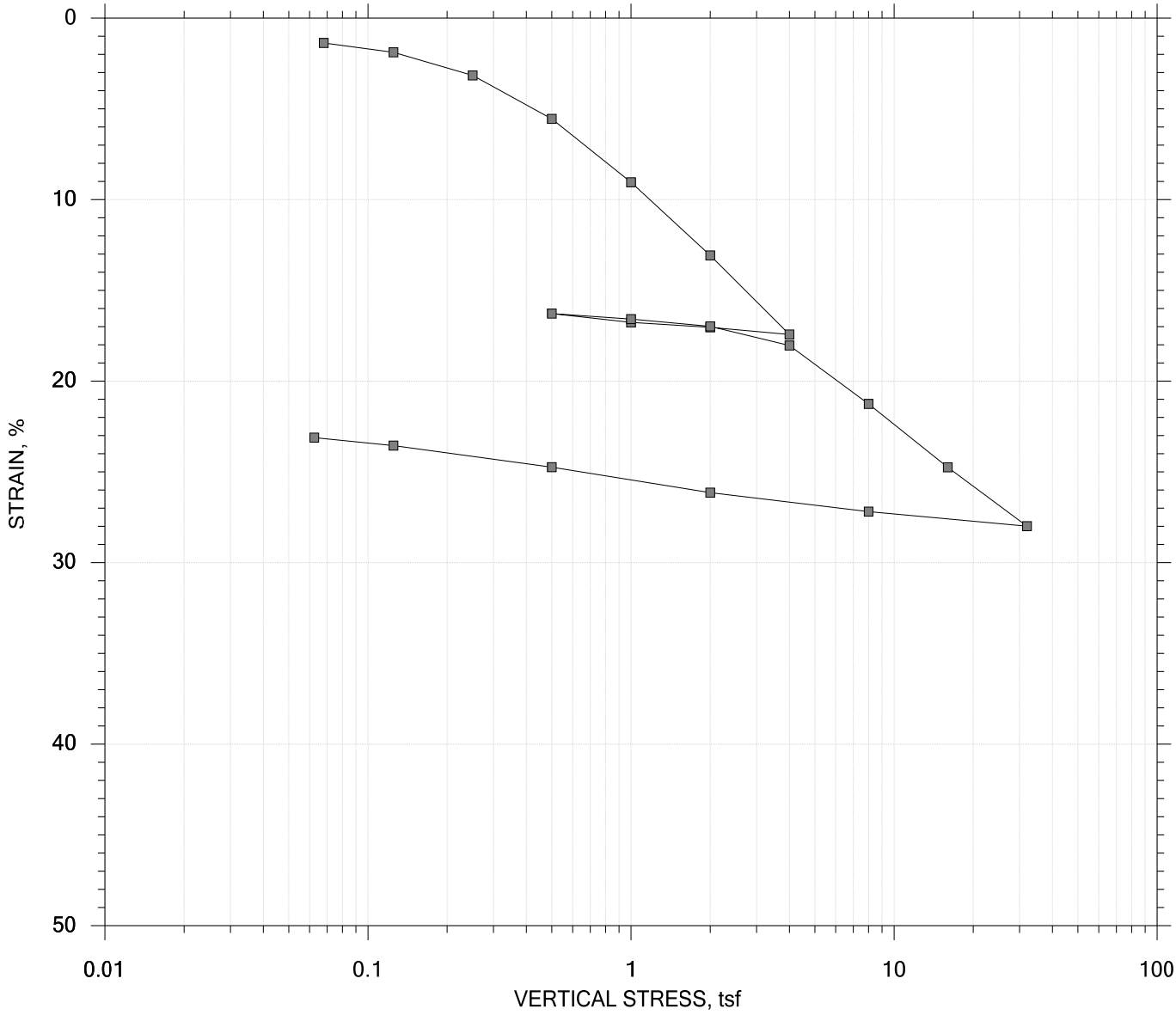




GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-104	Tested By: md	Checked By: njh
	Sample No.: U-1	Test Date: 05/30/17	Test No.: IP-1
	Depth: 20-21.4 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System O, Swell Pressure = 0.0679 tsf		
	Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method B

SUMMARY REPORT



				Before Test	After Test
Current Vertical Effective Stress: ---			Water Content, %	36.34	20.35
Preconsolidation Stress: ---			Dry Unit Weight, pcf	85.751	109.94
Compression Ratio: ---			Saturation, %	99.91	100.00
Diameter: 2.5 in		Height: 1 in	Void Ratio	1.00	0.56
LL: ---	PL: ---	PI: ---	GS: 2.74		

GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-104	Tested By: md	Checked By: njh
	Sample No.: U-1	Test Date: 05/30/17	Test No.: IP-1
	Depth: 20-21.4 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System O, Swell Pressure = 0.0679 tsf		
	Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method B

Project: Marsh Bridge | Location: Tremont/Southwest Harbor, ME | Project No.: GTX-306503
Boring No.: BB-MAR-104 | Tested By: md | Checked By: njh
Sample No.: U-1 | Test Date: 05/30/17 | Depth: 20-21.4 ft
Test No.: IP-1 | Sample Type: intact | Elevation: ---

Soil Description: Moist, gray silty clay
Remarks: System O, Swell Pressure = 0.0679 tsf

Estimated Specific Gravity: 2.74 | Liquid Limit: --- | Specimen Diameter: 2.50 in
Initial Void Ratio: 0.998 | Plastic Limit: --- | Initial Height: 1.00 in
Final Void Ratio: 0.559 | Plasticity Index: --- | Final Height: 0.78 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	B-696	RING		B528
Wt. Container + Wet Soil, gm	328.57	261.89	244.23	140.74
Wt. Container + Dry Soil, gm	240.38	221.74	221.74	118.31
Wt. Container, gm	8.5800	111.25	111.25	8.1000
Wt. Dry Soil, gm	231.80	110.49	110.49	110.21
Water Content, %	38.05	36.34	20.35	20.35
Void Ratio	---	0.998	0.559	---
Degree of Saturation, %	---	99.91	100.00	---
Dry Unit Weight, pcf	---	85.751	109.94	---

Note: Specific Gravity and Void Ratios are calculated assuming the degree of saturation equals 100% at the end of the test. Therefore, values may not represent actual values for the specimen.

One-Dimensional Consolidation by ASTM D2435 - Method B

Project: Marsh Bridge | Location: Tremont/Southwest Harbor, ME | Project No.: GTX-306503
Boring No.: BB-MAR-104 | Tested By: md | Checked By: njh
Sample No.: U-1 | Test Date: 05/30/17 | Depth: 20-21.4 ft
Test No.: IP-1 | Sample Type: intact | Elevation: ---

Soil Description: Moist, gray silty clay
Remarks: System O, Swell Pressure = 0.0679 tsf

Displacement at End of Increment

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	Sq.Rt T90 min	Cv ft²/sec	Mv 1/tsf	k ft/day
1	0.0679	0.01369	0.971	1.37	17.695	1.37e-006	2.02e-001	7.44e-004
2	0.125	0.01884	0.961	1.88	35.089	6.77e-007	9.02e-002	1.65e-004
3	0.250	0.03153	0.935	3.15	45.890	5.08e-007	1.01e-001	1.39e-004
4	0.500	0.05549	0.887	5.55	29.017	7.74e-007	9.58e-002	2.00e-004
5	1.00	0.09041	0.818	9.04	23.539	8.96e-007	6.98e-002	1.69e-004
6	2.00	0.1307	0.737	13.1	17.539	1.11e-006	4.03e-002	1.20e-004
7	4.00	0.1743	0.650	17.4	12.027	1.47e-006	2.18e-002	8.61e-005
8	2.00	0.1703	0.658	17.0	0.816	2.06e-005	1.95e-003	1.08e-004
9	1.00	0.1676	0.663	16.8	2.893	5.86e-006	2.74e-003	4.33e-005
10	0.500	0.1627	0.673	16.3	6.508	2.63e-006	9.72e-003	6.89e-005
11	1.00	0.1658	0.667	16.6	6.002	2.86e-006	6.17e-003	4.75e-005
12	2.00	0.1698	0.659	17.0	1.250	1.36e-005	4.00e-003	1.47e-004
13	4.00	0.1804	0.638	18.0	7.026	2.38e-006	5.26e-003	3.37e-005
14	8.00	0.2126	0.573	21.3	8.784	1.80e-006	8.05e-003	3.92e-005
15	16.0	0.2475	0.504	24.7	5.570	2.61e-006	4.36e-003	3.07e-005
16	32.0	0.2799	0.439	28.0	3.468	3.84e-006	2.03e-003	2.10e-005
17	8.00	0.2719	0.455	27.2	0.897	1.43e-005	3.34e-004	1.29e-005
18	2.00	0.2614	0.476	26.1	5.553	2.38e-006	1.75e-003	1.12e-005
19	0.500	0.2474	0.504	24.7	19.055	7.16e-007	9.31e-003	1.80e-005
20	0.125	0.2355	0.528	23.5	110.022	1.28e-007	3.18e-002	1.10e-005
21	0.0625	0.2311	0.536	23.1	226.284	6.37e-008	7.00e-002	1.20e-005

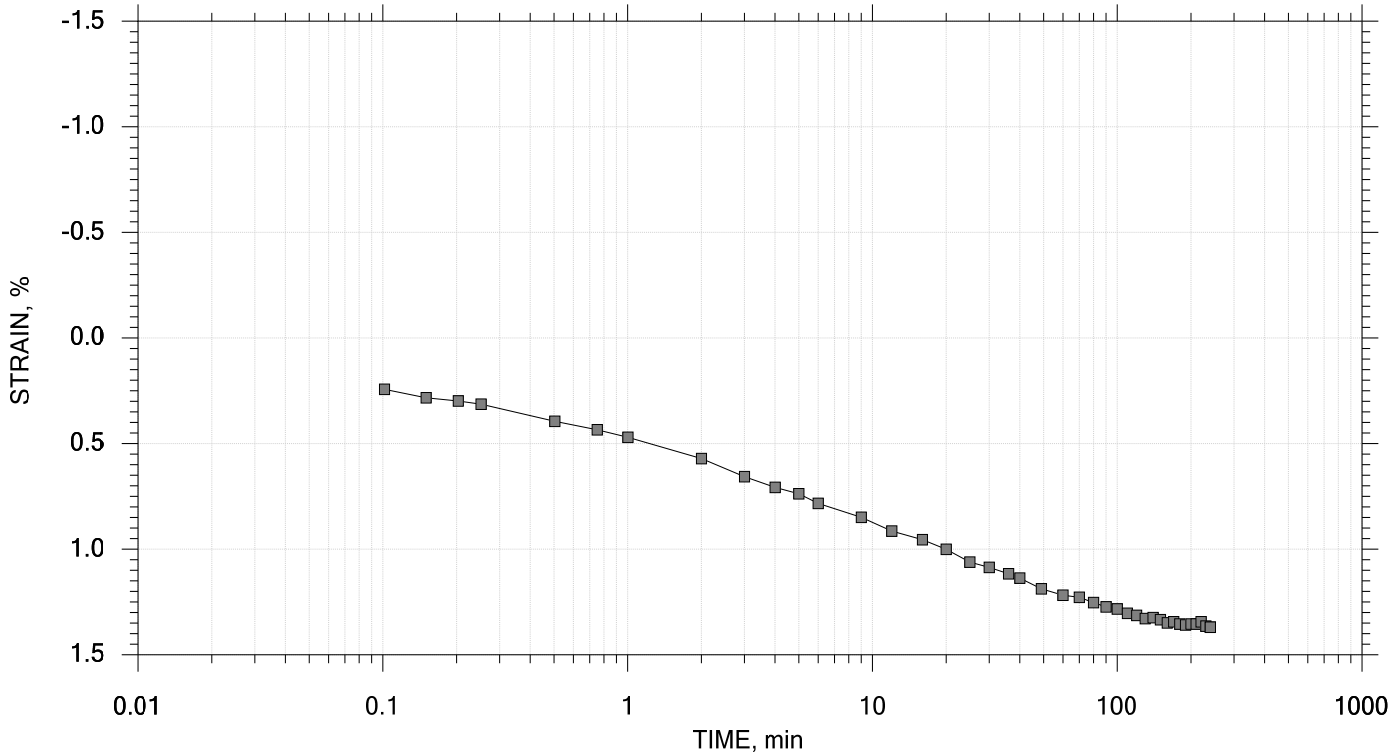
	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	Log T50 min	Cv ft²/sec	Mv 1/tsf	k ft/day	Ca %
1	0.0679	0.01369	0.971	1.37	0.000	0.00e+000	2.02e-001	0.00e+000	0.00e+000
2	0.125	0.01884	0.961	1.88	10.787	5.11e-007	9.02e-002	1.24e-004	0.00e+000
3	0.250	0.03153	0.935	3.15	0.000	0.00e+000	1.01e-001	0.00e+000	0.00e+000
4	0.500	0.05549	0.887	5.55	7.112	7.33e-007	9.58e-002	1.90e-004	0.00e+000
5	1.00	0.09041	0.818	9.04	5.336	9.18e-007	6.98e-002	1.73e-004	0.00e+000
6	2.00	0.1307	0.737	13.1	5.039	8.95e-007	4.03e-002	9.72e-005	0.00e+000
7	4.00	0.1743	0.650	17.4	2.535	1.62e-006	2.18e-002	9.49e-005	0.00e+000
8	2.00	0.1703	0.658	17.0	0.000	0.00e+000	1.95e-003	0.00e+000	0.00e+000
9	1.00	0.1676	0.663	16.8	0.000	0.00e+000	2.74e-003	0.00e+000	0.00e+000
10	0.500	0.1627	0.673	16.3	1.210	3.28e-006	9.72e-003	8.61e-005	0.00e+000
11	1.00	0.1658	0.667	16.6	0.000	0.00e+000	6.17e-003	0.00e+000	0.00e+000
12	2.00	0.1698	0.659	17.0	0.000	0.00e+000	4.00e-003	0.00e+000	0.00e+000
13	4.00	0.1804	0.638	18.0	0.000	0.00e+000	5.26e-003	0.00e+000	0.00e+000
14	8.00	0.2126	0.573	21.3	2.154	1.71e-006	8.05e-003	3.71e-005	0.00e+000
15	16.0	0.2475	0.504	24.7	1.235	2.74e-006	4.36e-003	3.22e-005	0.00e+000
16	32.0	0.2799	0.439	28.0	1.097	2.82e-006	2.03e-003	1.54e-005	0.00e+000
17	8.00	0.2719	0.455	27.2	0.000	0.00e+000	3.34e-004	0.00e+000	0.00e+000
18	2.00	0.2614	0.476	26.1	0.000	0.00e+000	1.75e-003	0.00e+000	0.00e+000
19	0.500	0.2474	0.504	24.7	0.000	0.00e+000	9.31e-003	0.00e+000	0.00e+000
20	0.125	0.2355	0.528	23.5	0.000	0.00e+000	3.18e-002	0.00e+000	0.00e+000
21	0.0625	0.2311	0.536	23.1	0.000	0.00e+000	7.00e-002	0.00e+000	0.00e+000

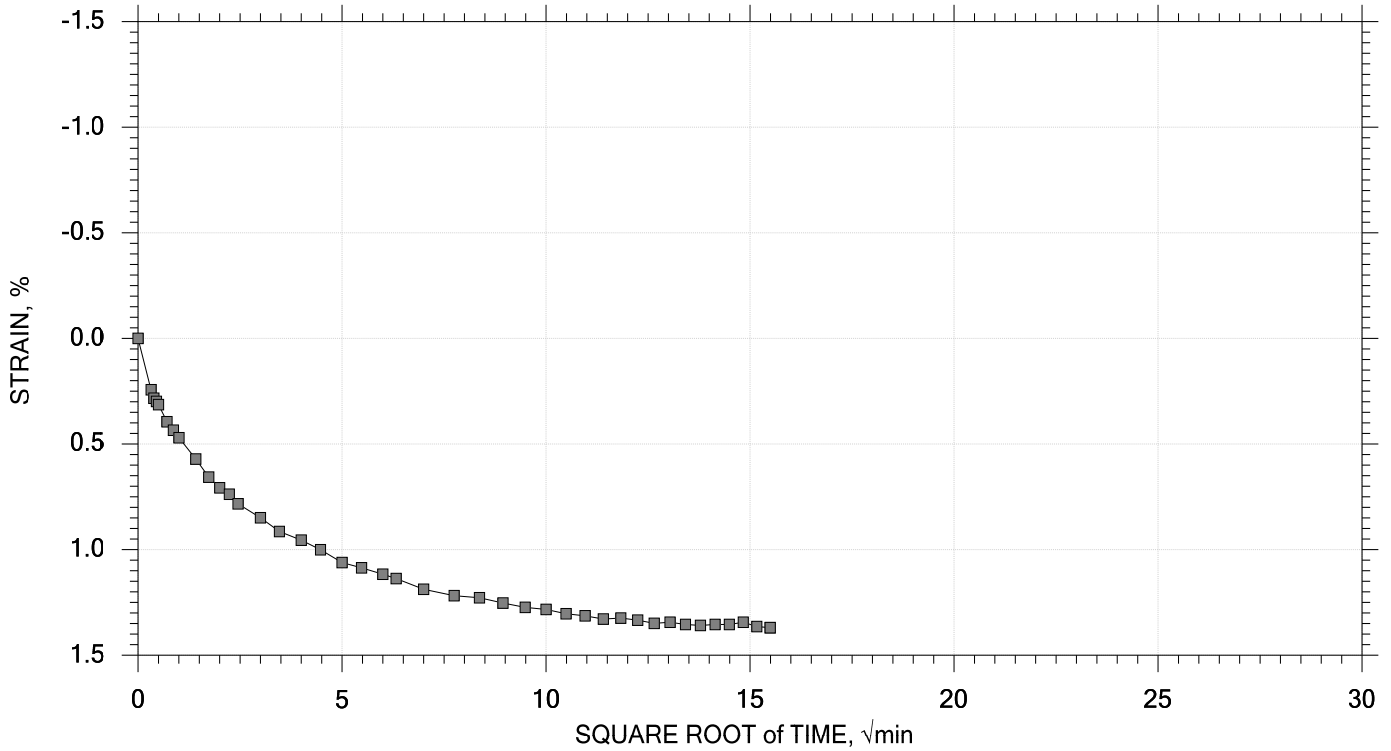
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Volume Step 1 of 21

Stress: 0.06788 tsf





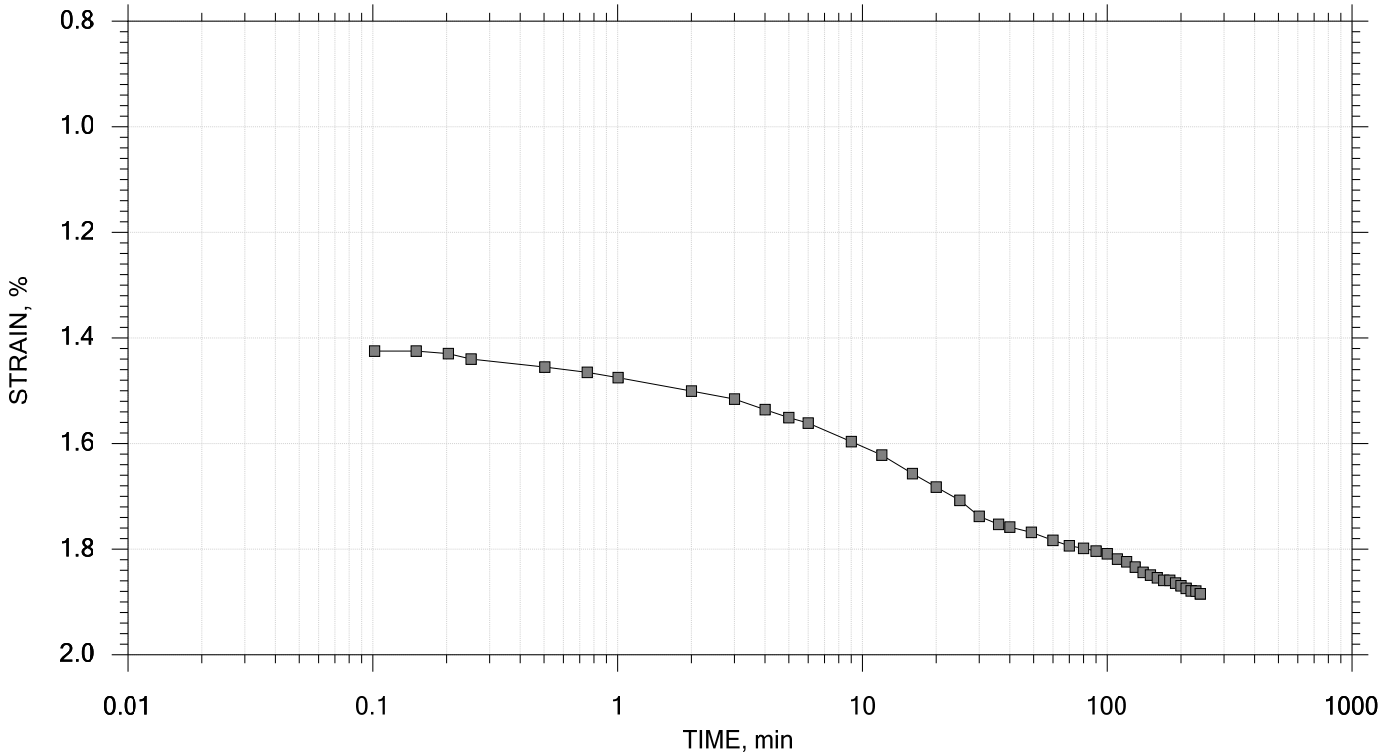
GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-104	Tested By: md	Checked By: njh
	Sample No.: U-1	Test Date: 05/30/17	Test No.: IP-1
	Depth: 20-21.4 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System O, Swell Pressure = 0.0679 tsf		

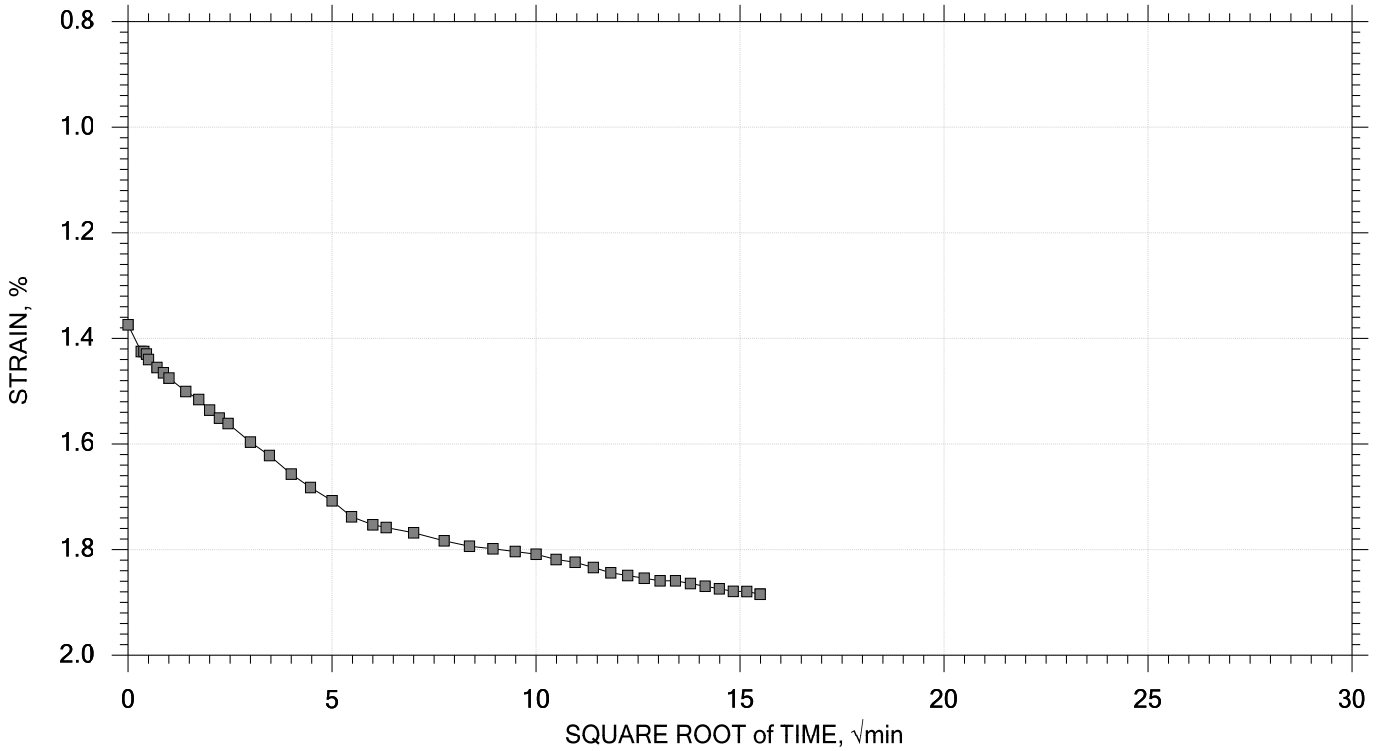
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 2 of 21

Stress: 0.125 tsf





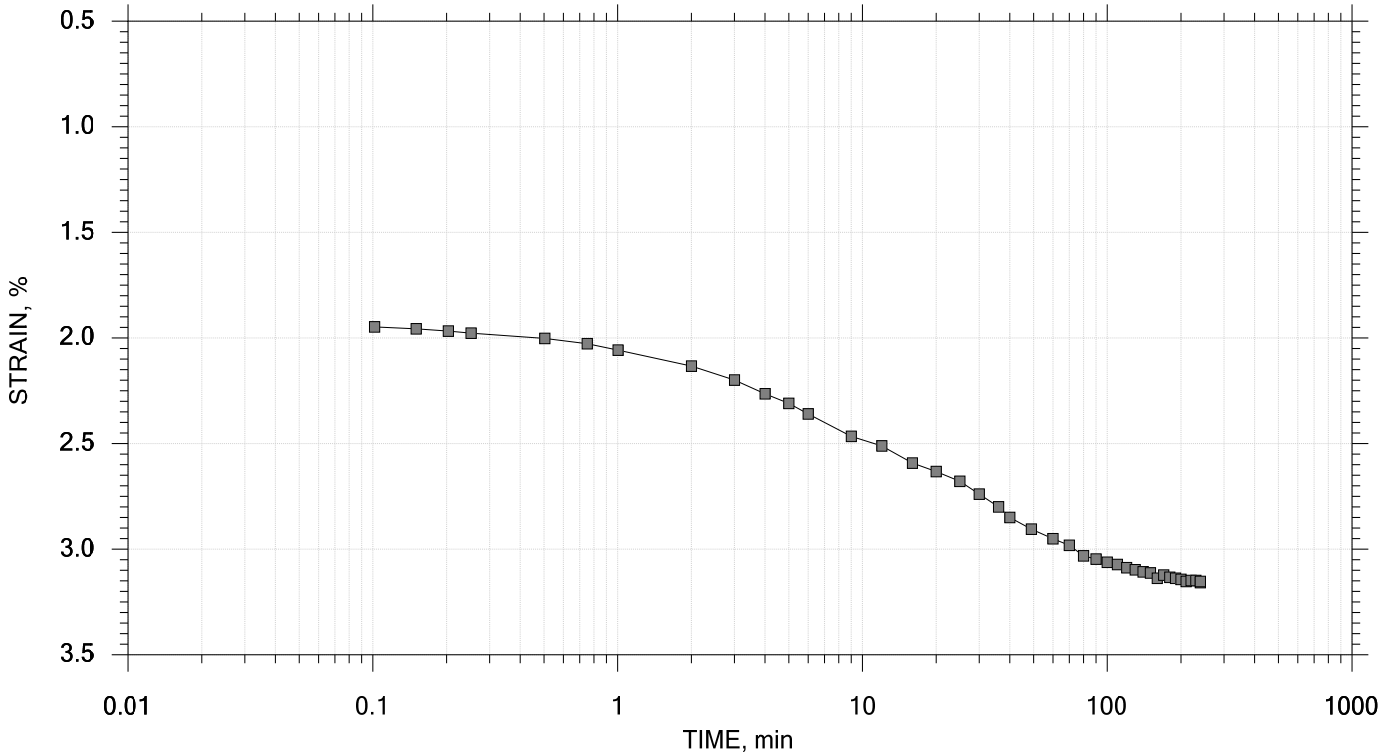
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	Boring No.: BB-MAR-104	Tested By: md	Checked By: njh
	Sample No.: U-1	Test Date: 05/30/17	Test No.: IP-1
	Depth: 20-21.4 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System O, Swell Pressure = 0.0679 tsf		

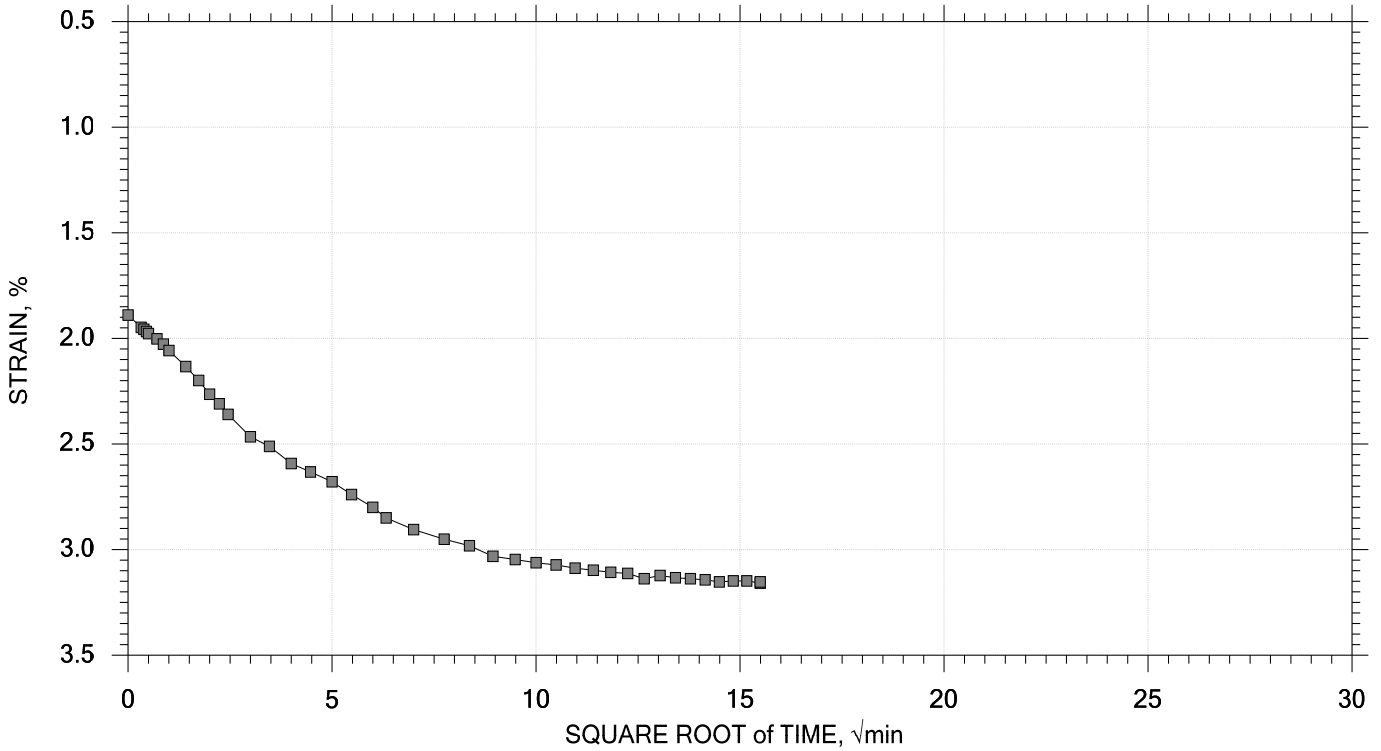
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 3 of 21

Stress: 0.25 tsf





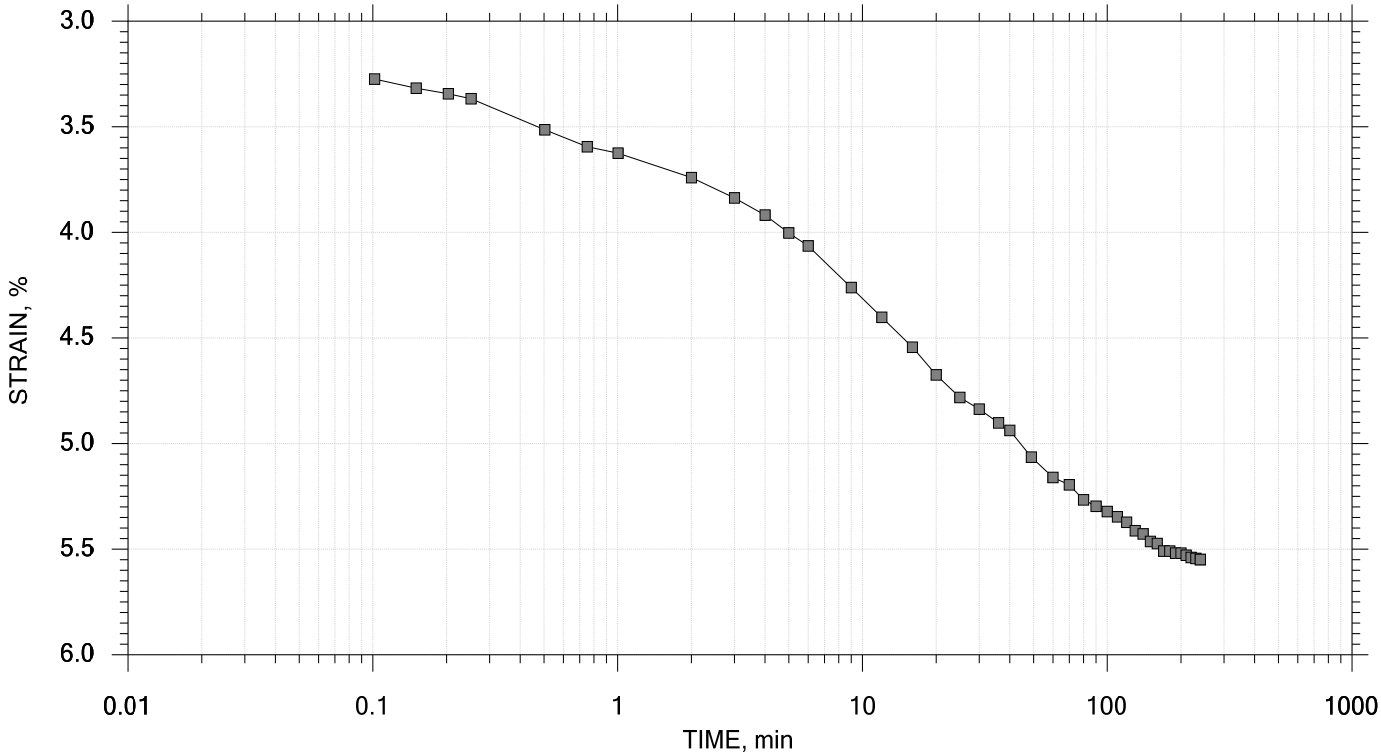
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	Boring No.: BB-MAR-104	Tested By: md	Checked By: njh
	Sample No.: U-1	Test Date: 05/30/17	Test No.: IP-1
	Depth: 20-21.4 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System O, Swell Pressure = 0.0679 tsf		

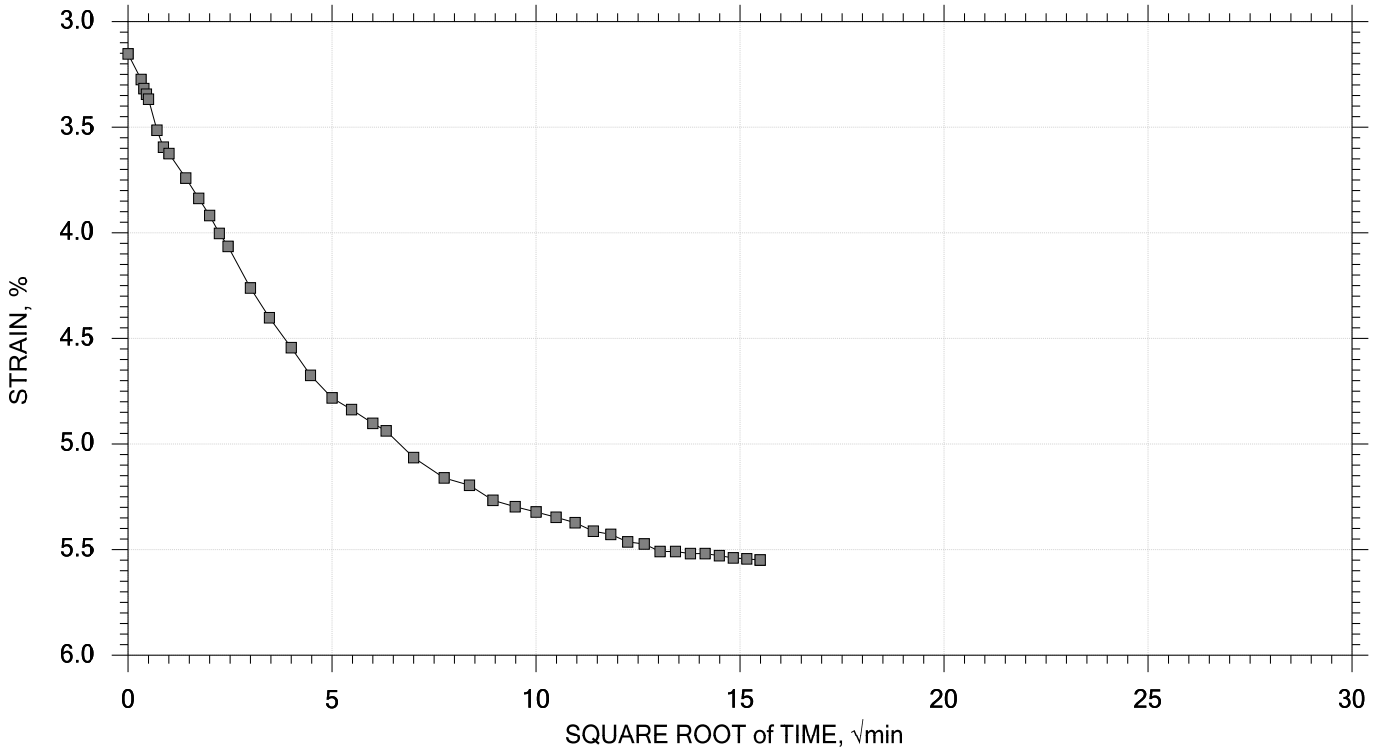
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 4 of 21

Stress: 0.5 tsf





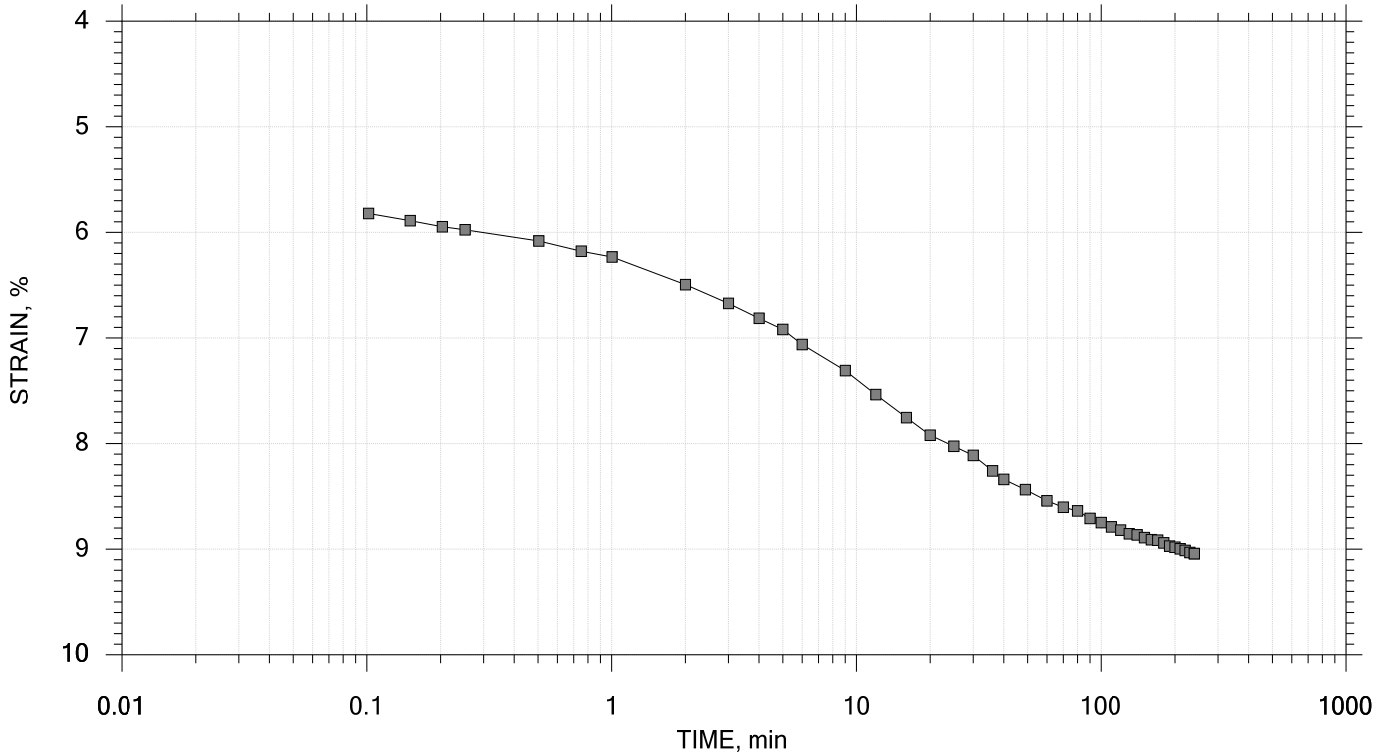
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	Boring No.: BB-MAR-104	Tested By: md	Checked By: njh
	Sample No.: U-1	Test Date: 05/30/17	Test No.: IP-1
	Depth: 20-21.4 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System O, Swell Pressure = 0.0679 tsf		

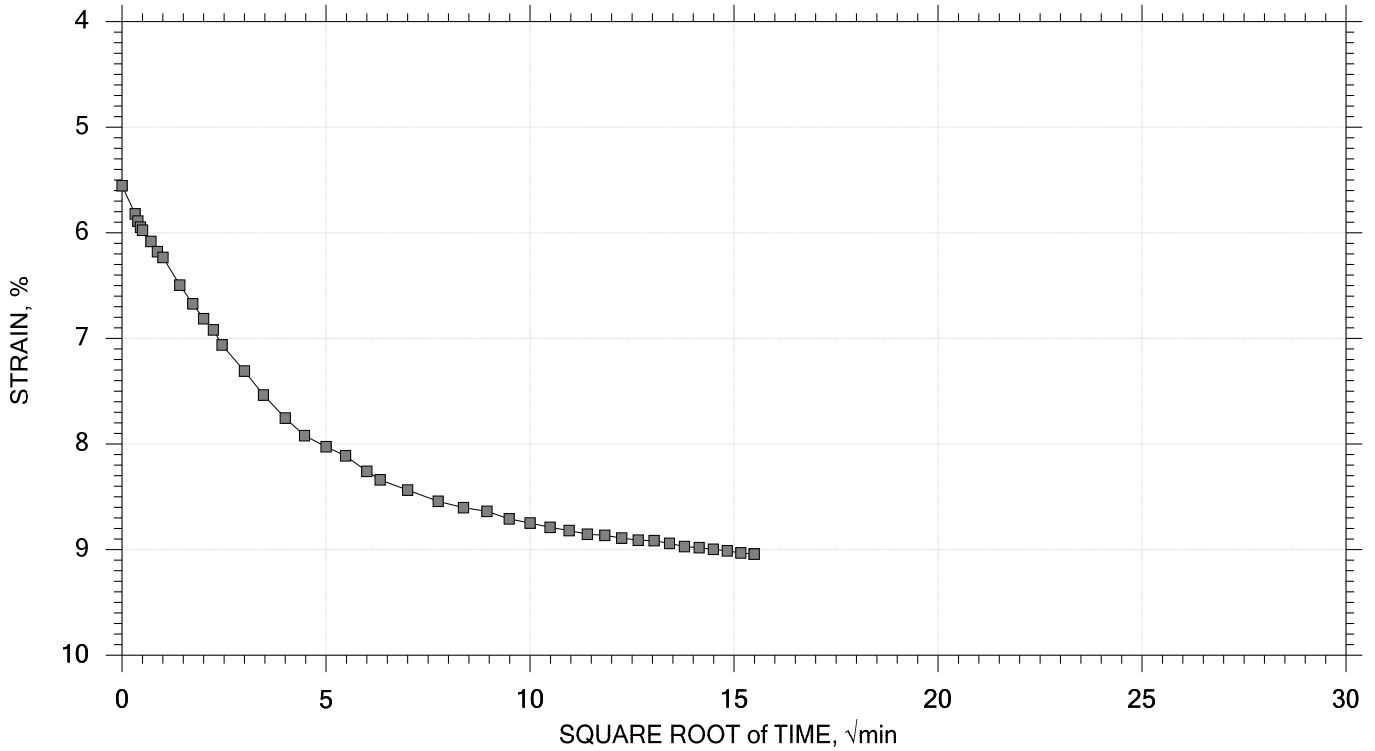
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 5 of 21

Stress: 1 tsf





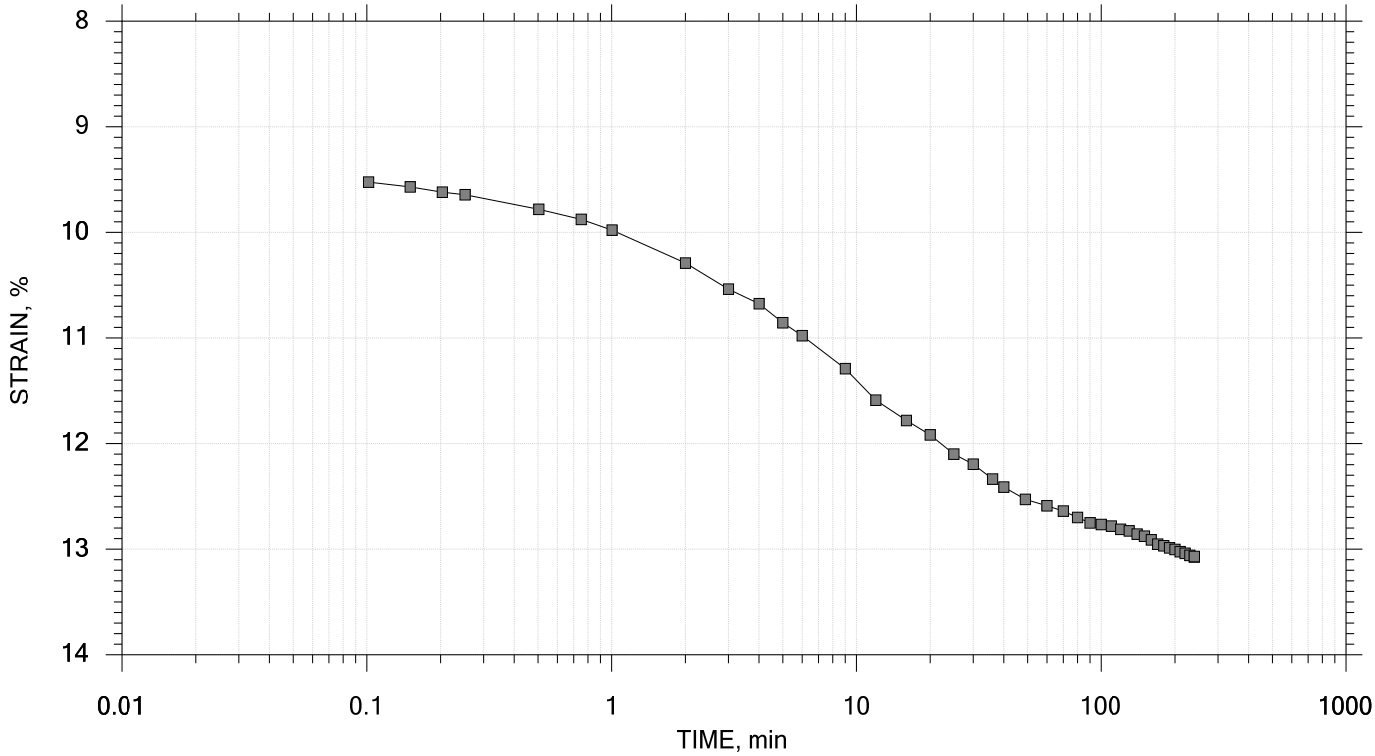
GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-104	Tested By: md	Checked By: njh
	Sample No.: U-1	Test Date: 05/30/17	Test No.: IP-1
	Depth: 20-21.4 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System O, Swell Pressure = 0.0679 tsf		

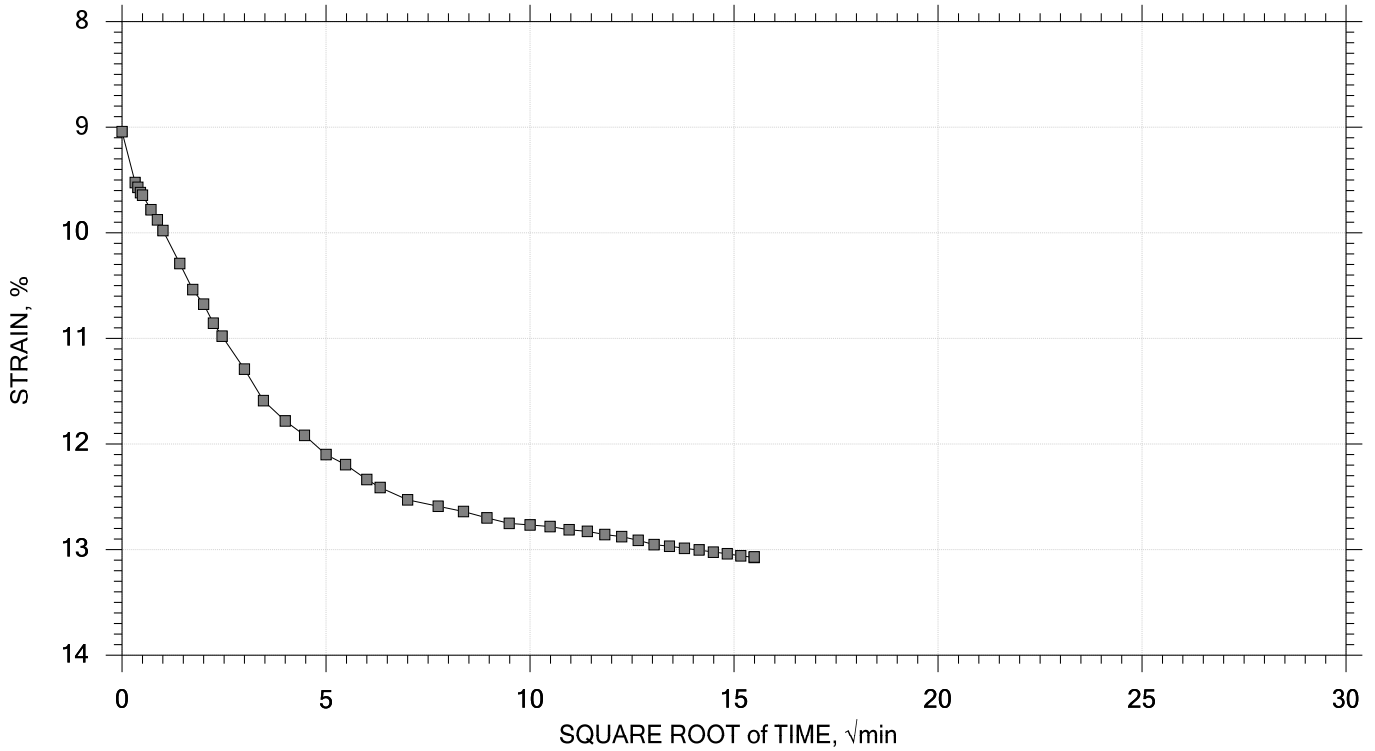
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 6 of 21

Stress: 2 tsf





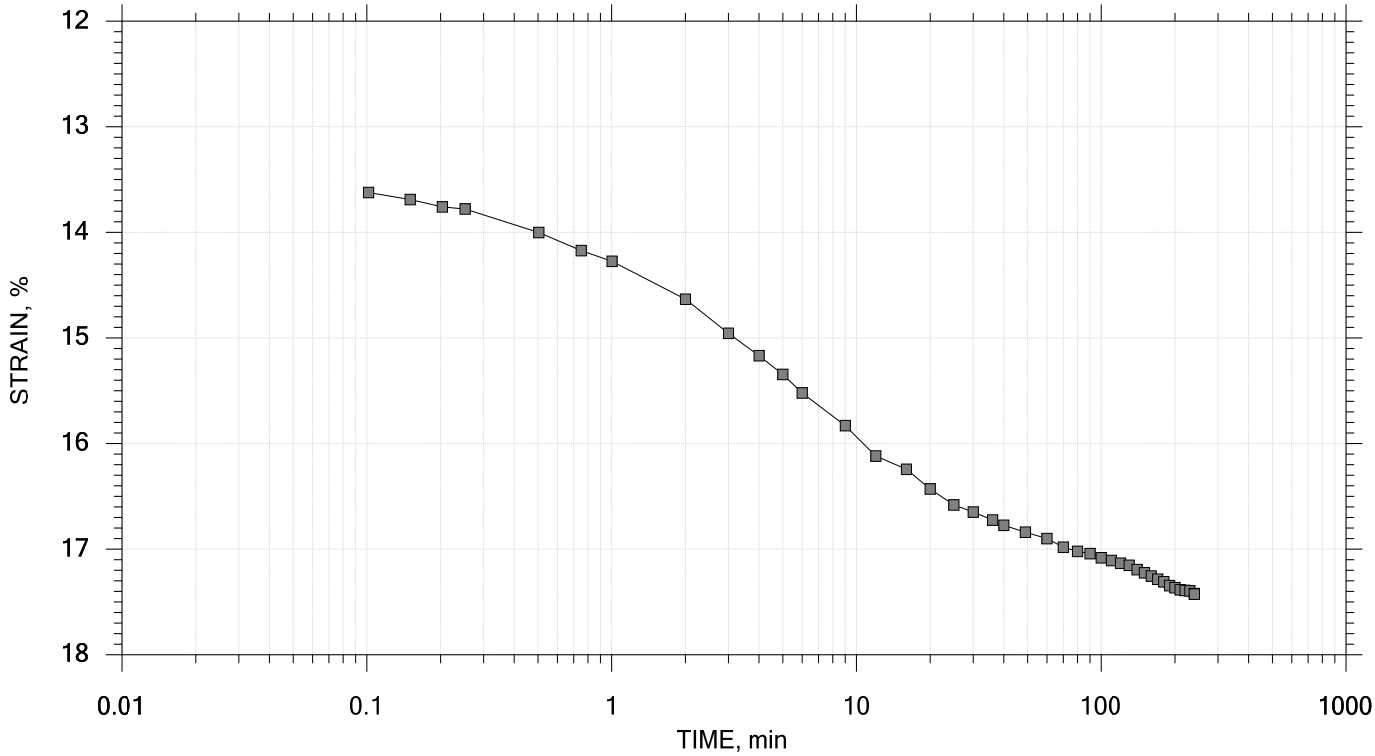
GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-104	Tested By: md	Checked By: njh
	Sample No.: U-1	Test Date: 05/30/17	Test No.: IP-1
	Depth: 20-21.4 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System O, Swell Pressure = 0.0679 tsf		

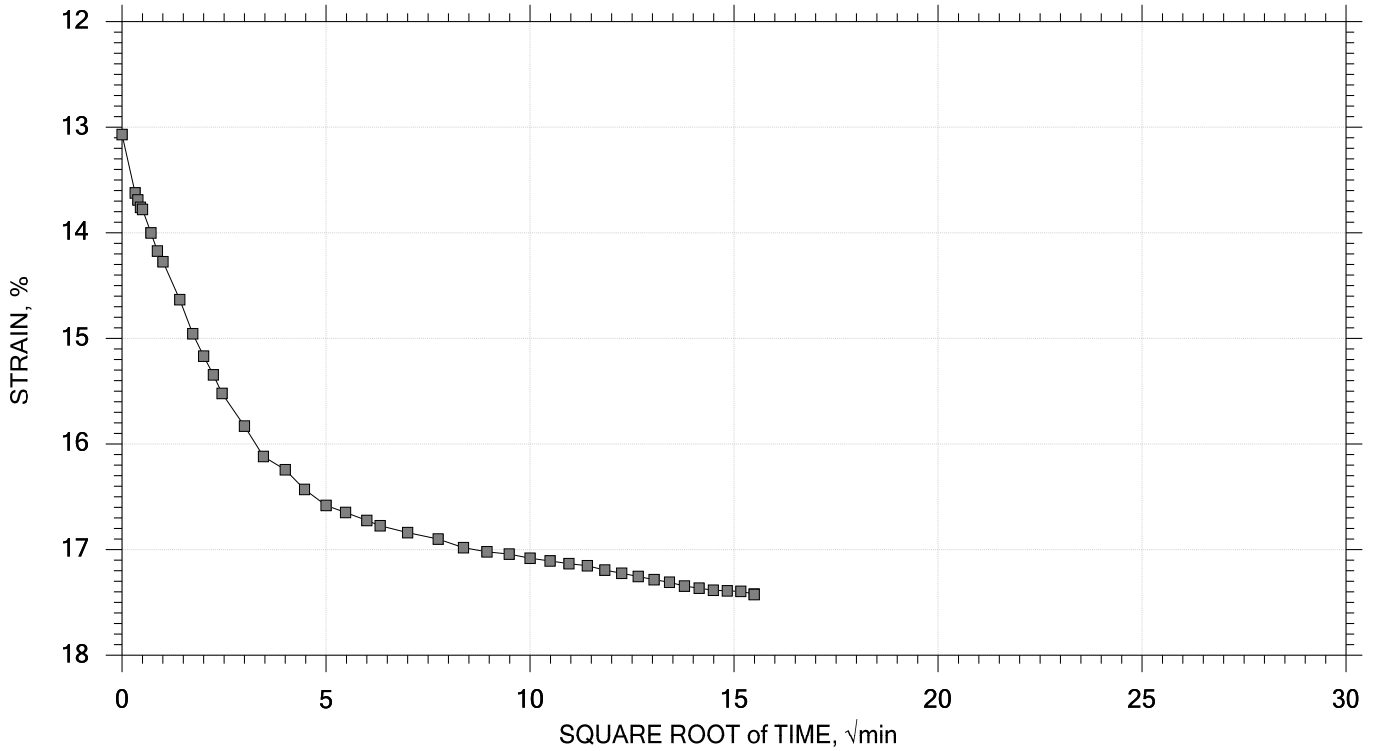
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 7 of 21

Stress: 4 tsf





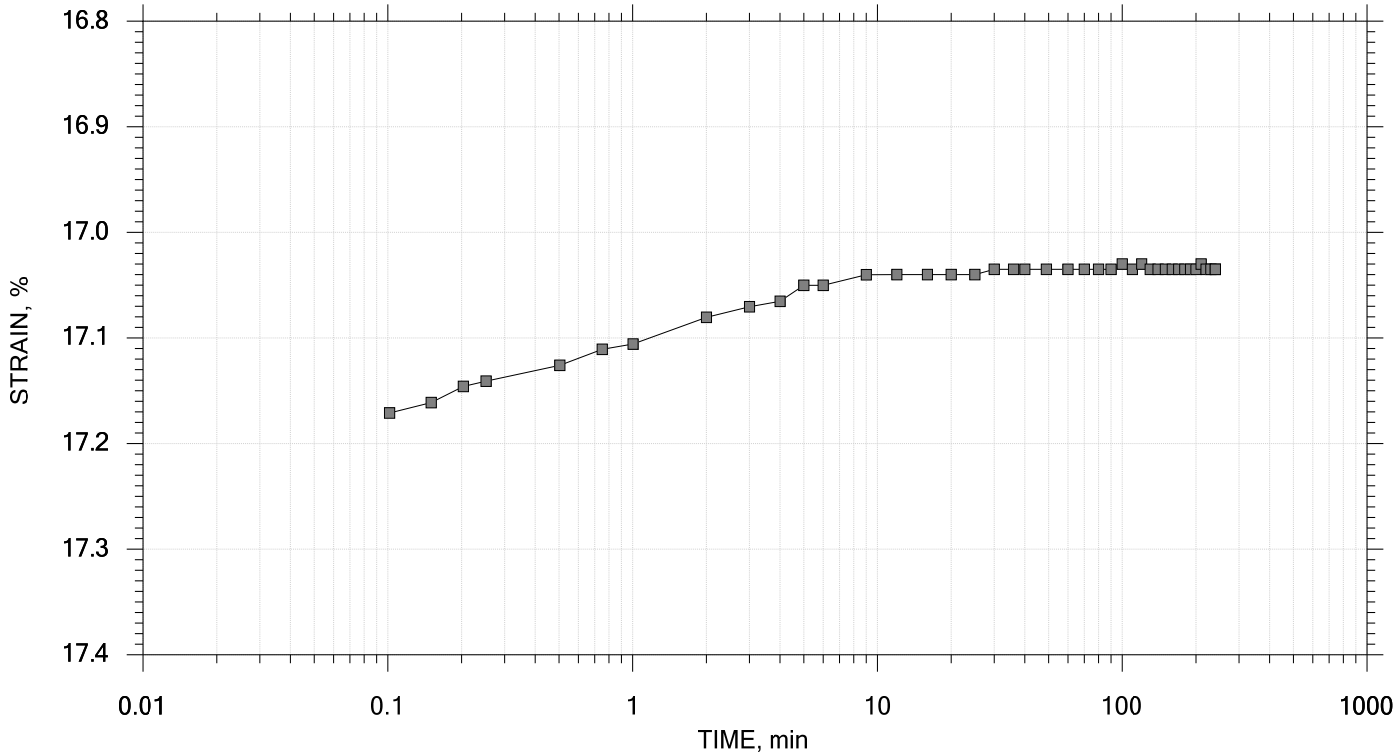
GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-104	Tested By: md	Checked By: njh
	Sample No.: U-1	Test Date: 05/30/17	Test No.: IP-1
	Depth: 20-21.4 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System O, Swell Pressure = 0.0679 tsf		

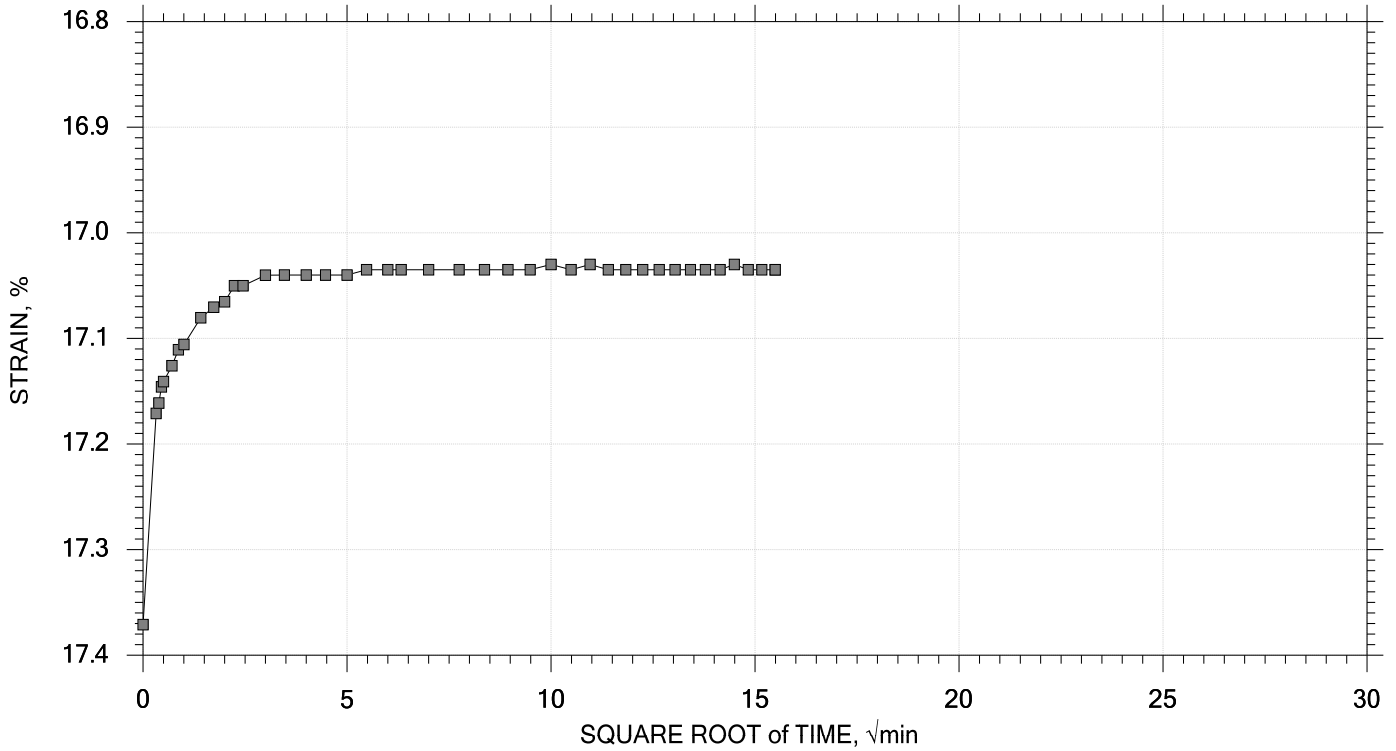
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 8 of 21

Stress: 2 tsf





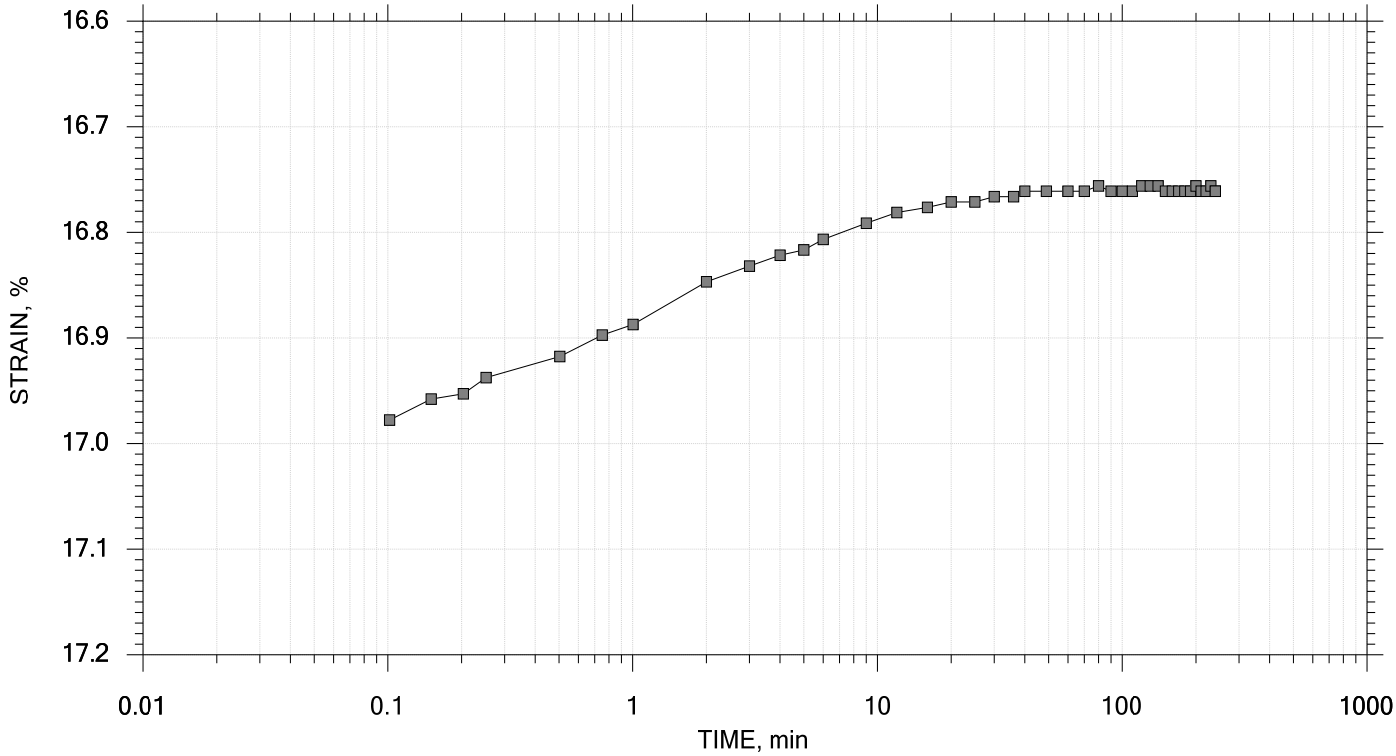
GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-104	Tested By: md	Checked By: njh
	Sample No.: U-1	Test Date: 05/30/17	Test No.: IP-1
	Depth: 20-21.4 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System O, Swell Pressure = 0.0679 tsf		

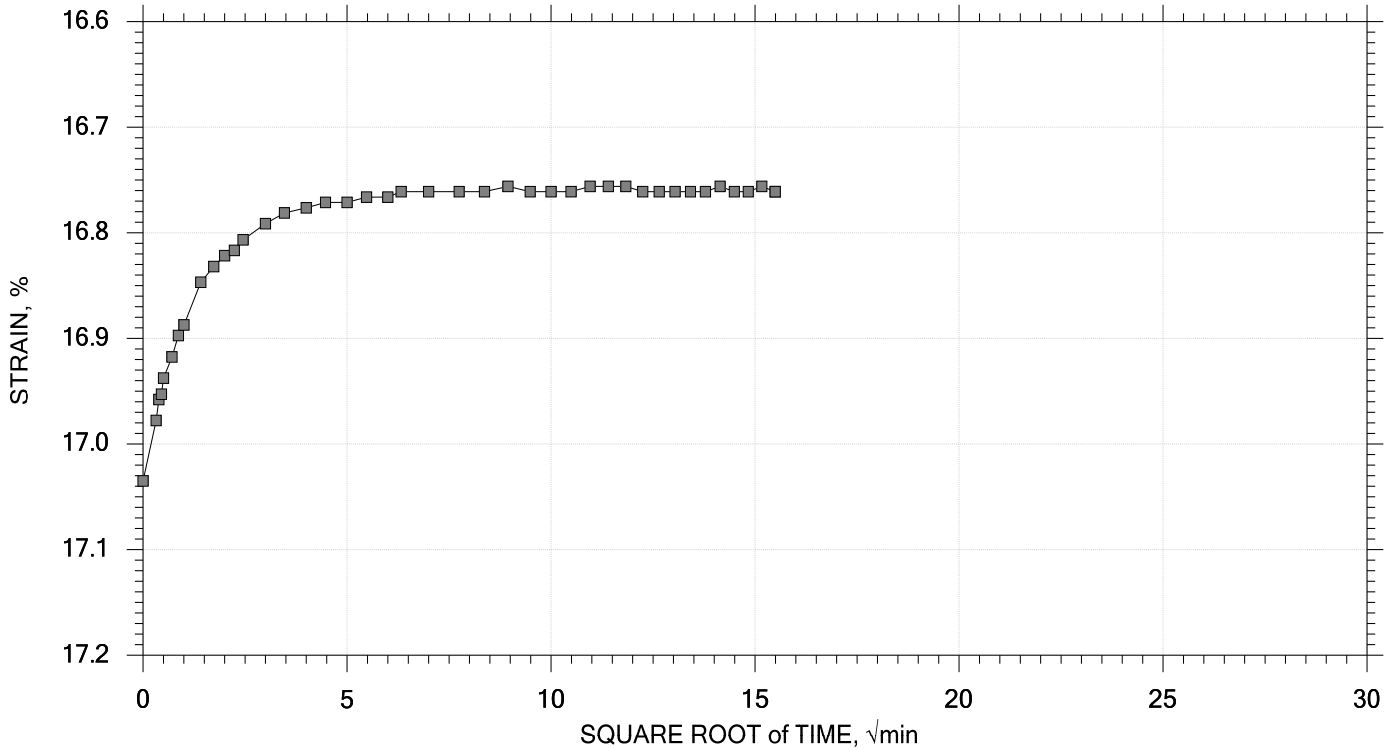
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 9 of 21

Stress: 1 tsf





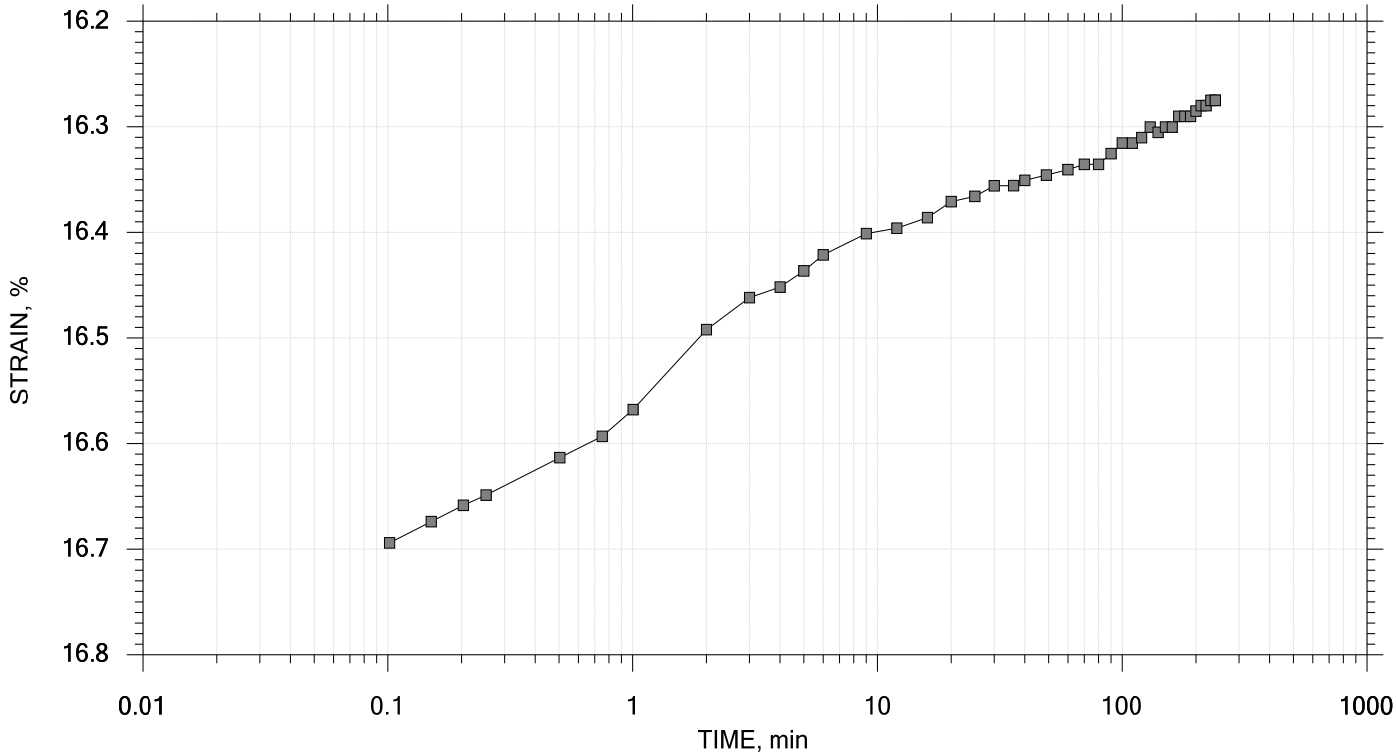
GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-104	Tested By: md	Checked By: njh
	Sample No.: U-1	Test Date: 05/30/17	Test No.: IP-1
	Depth: 20-21.4 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System O, Swell Pressure = 0.0679 tsf		

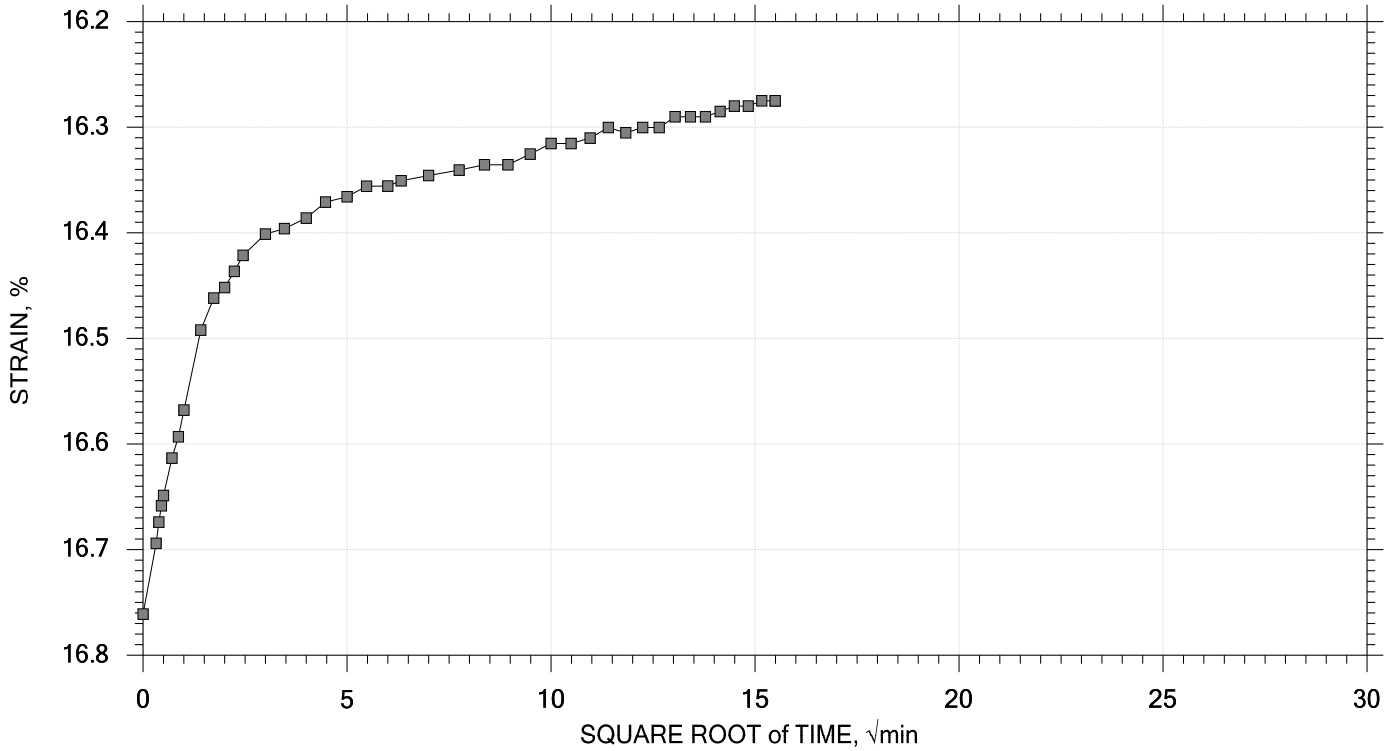
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 10 of 21

Stress: 0.5 tsf





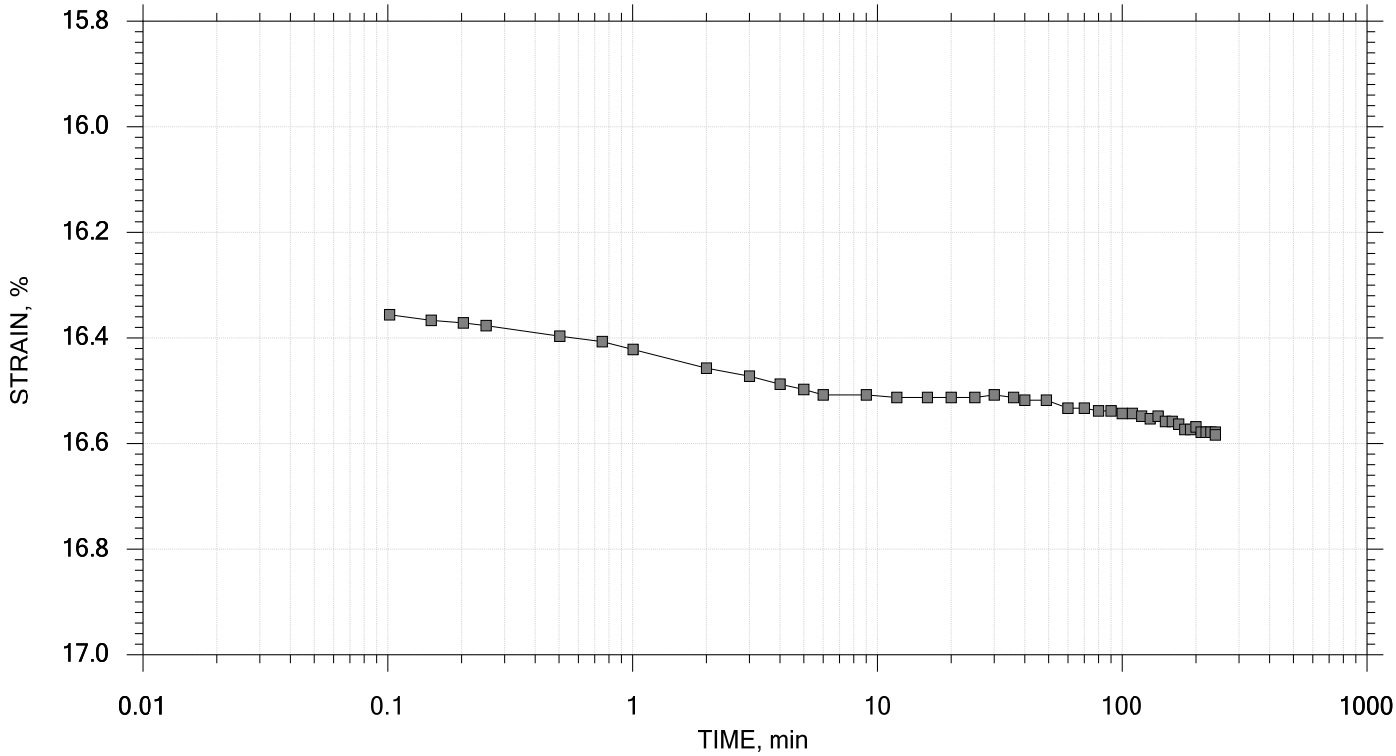
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	Boring No.: BB-MAR-104	Tested By: md	Checked By: njh
	Sample No.: U-1	Test Date: 05/30/17	Test No.: IP-1
	Depth: 20-21.4 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System O, Swell Pressure = 0.0679 tsf		

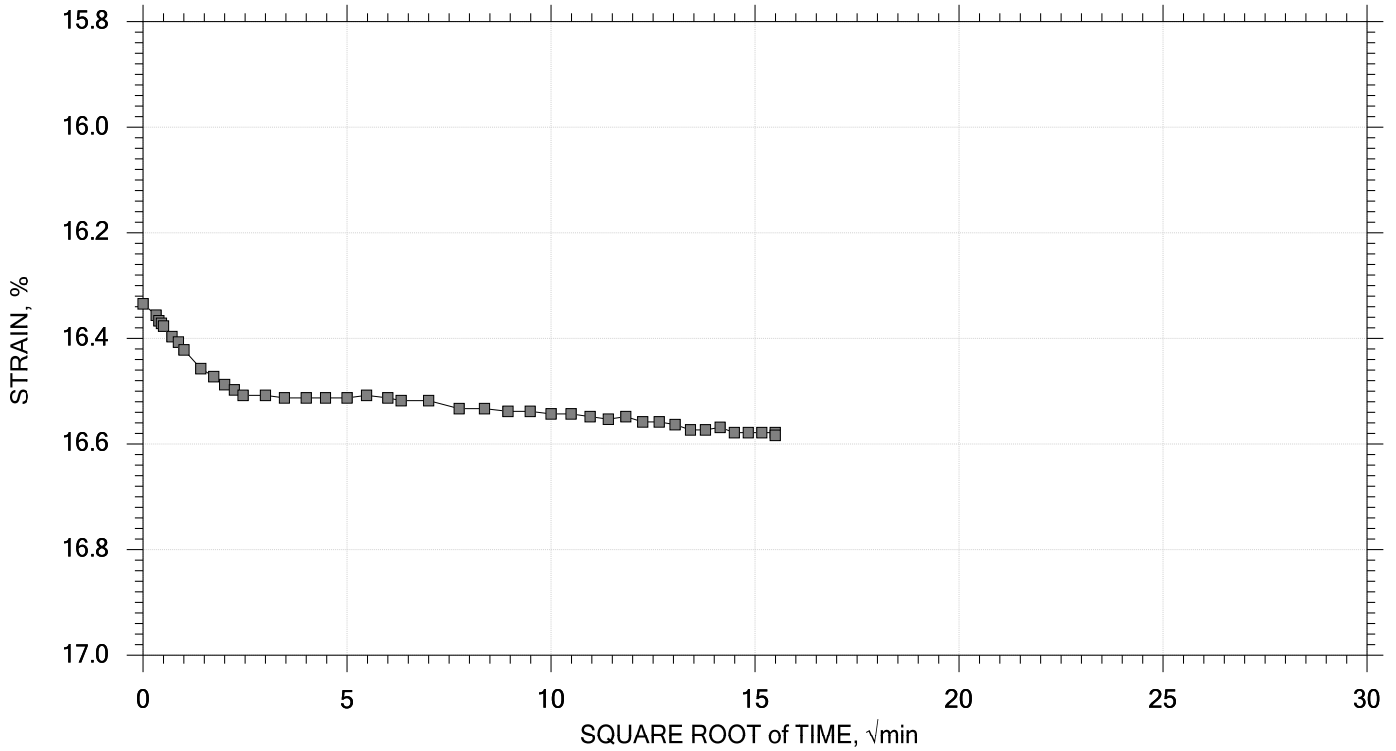
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 11 of 21

Stress: 1 tsf





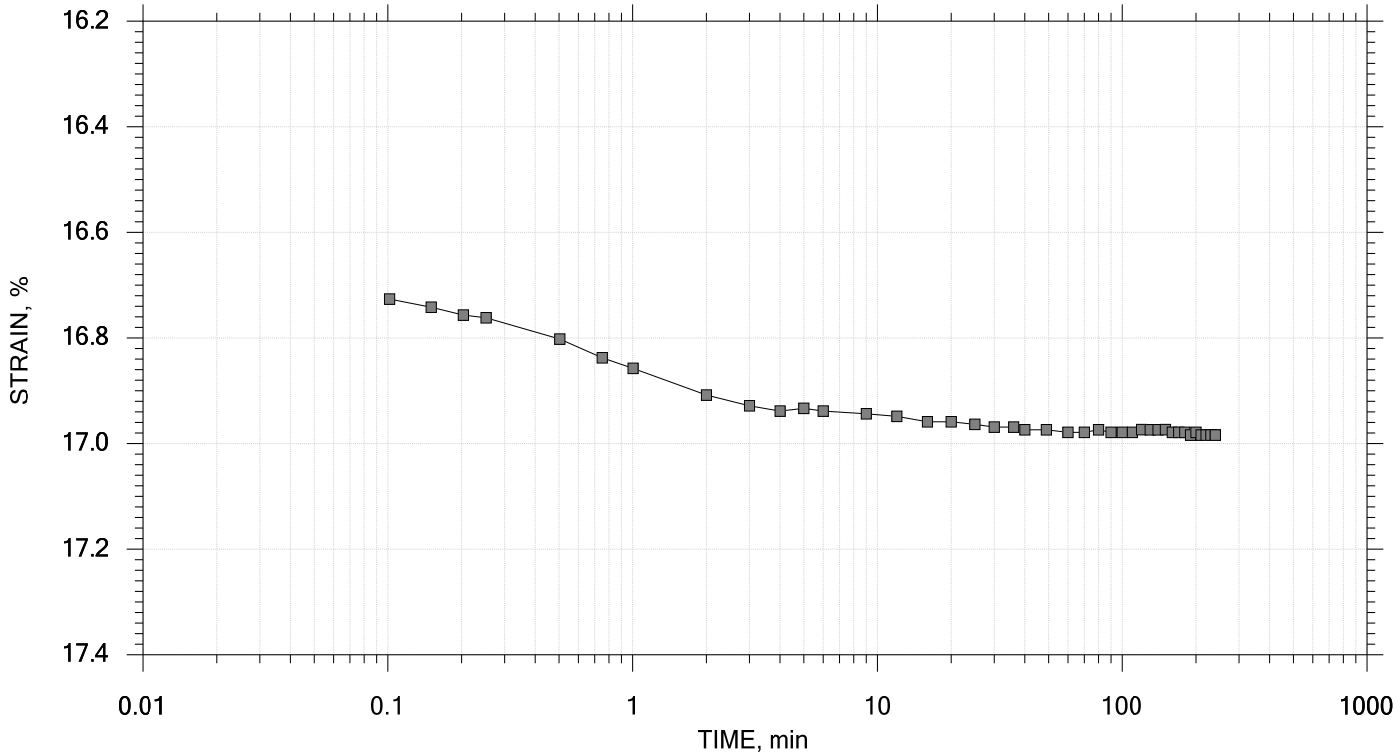
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	Boring No.: BB-MAR-104	Tested By: md	Checked By: njh
	Sample No.: U-1	Test Date: 05/30/17	Test No.: IP-1
	Depth: 20-21.4 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System O, Swell Pressure = 0.0679 tsf		

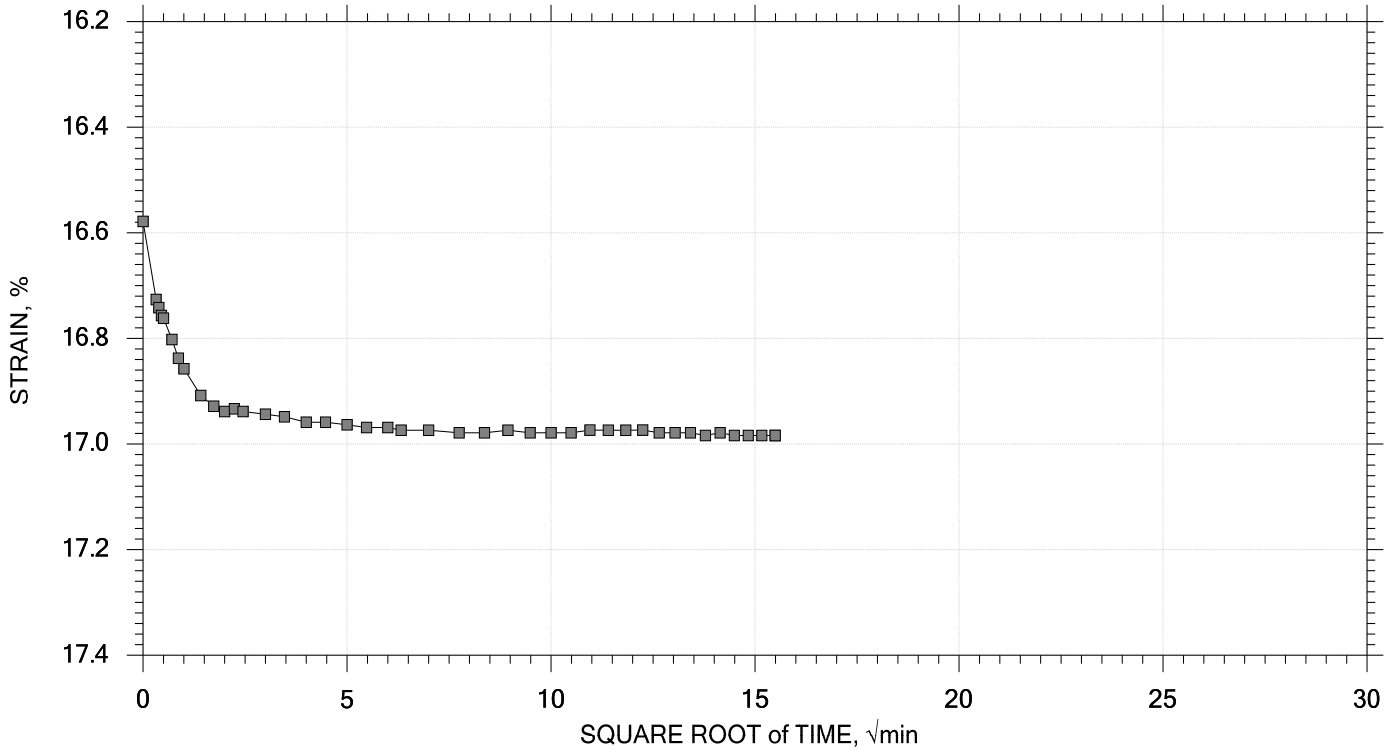
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 12 of 21

Stress: 2 tsf





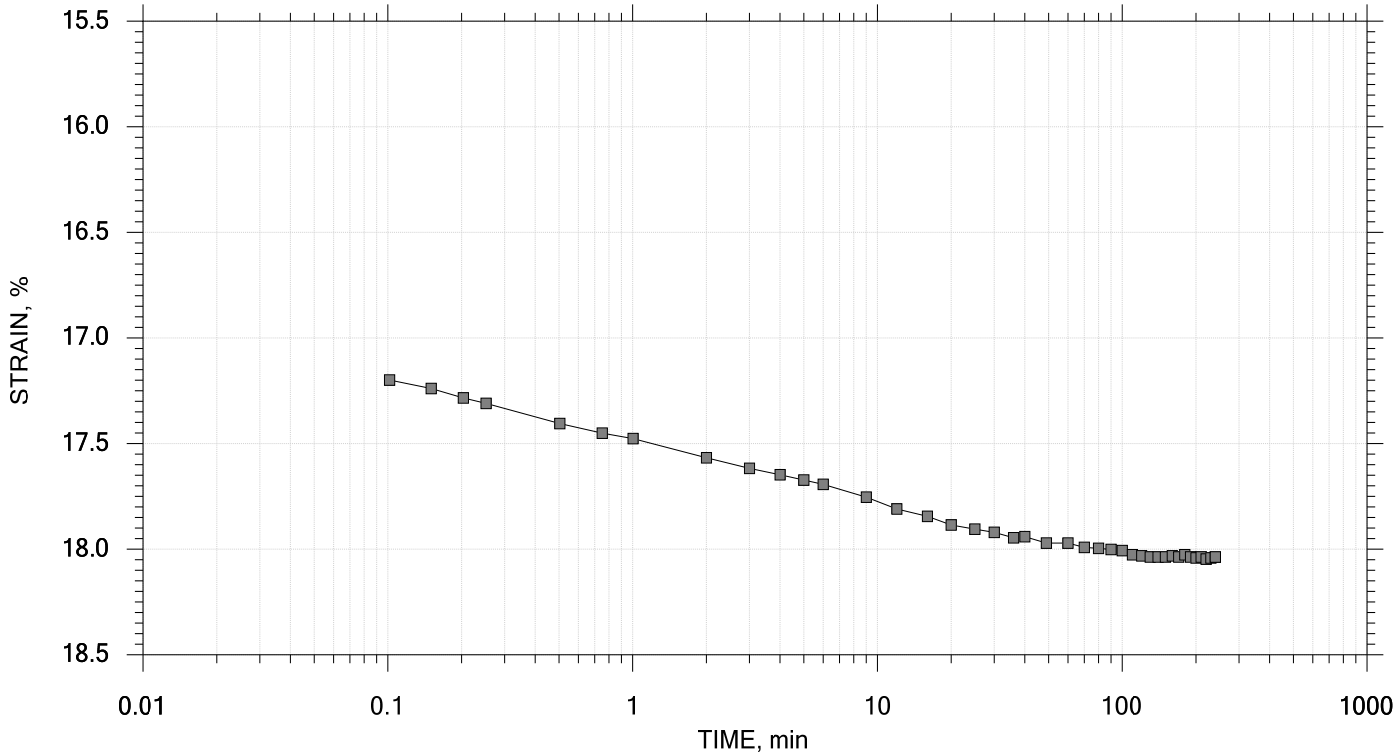
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	Boring No.: BB-MAR-104	Tested By: md	Checked By: njh
	Sample No.: U-1	Test Date: 05/30/17	Test No.: IP-1
	Depth: 20-21.4 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System O, Swell Pressure = 0.0679 tsf		

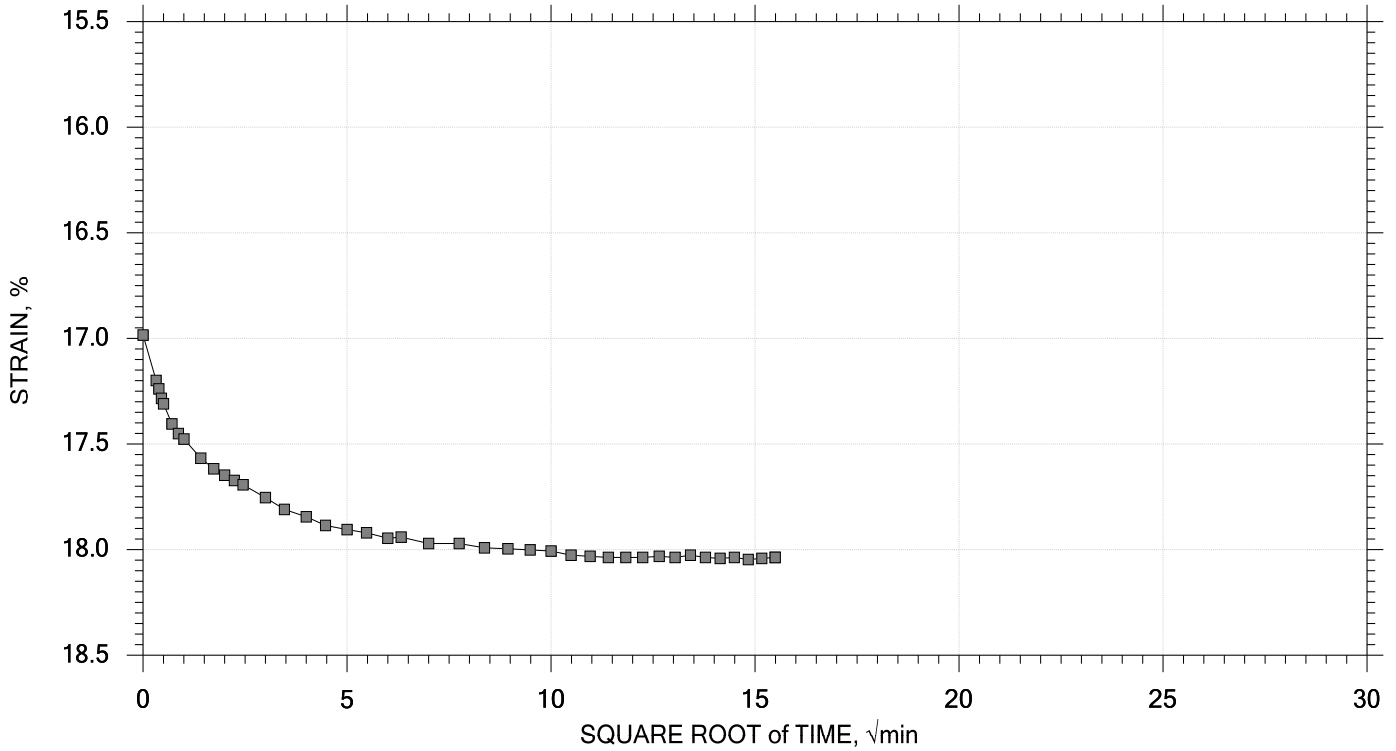
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 13 of 21

Stress: 4 tsf





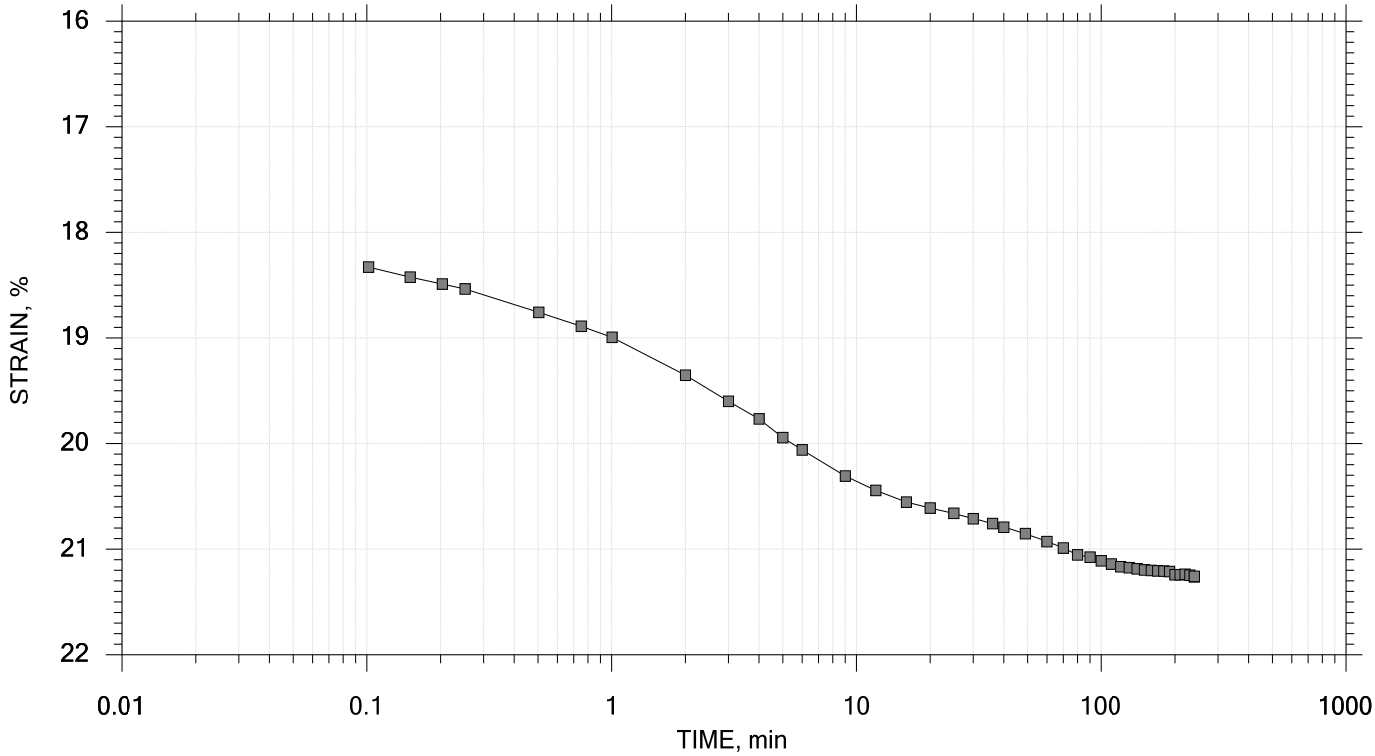
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	Boring No.: BB-MAR-104	Tested By: md	Checked By: njh
	Sample No.: U-1	Test Date: 05/30/17	Test No.: IP-1
	Depth: 20-21.4 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System O, Swell Pressure = 0.0679 tsf		

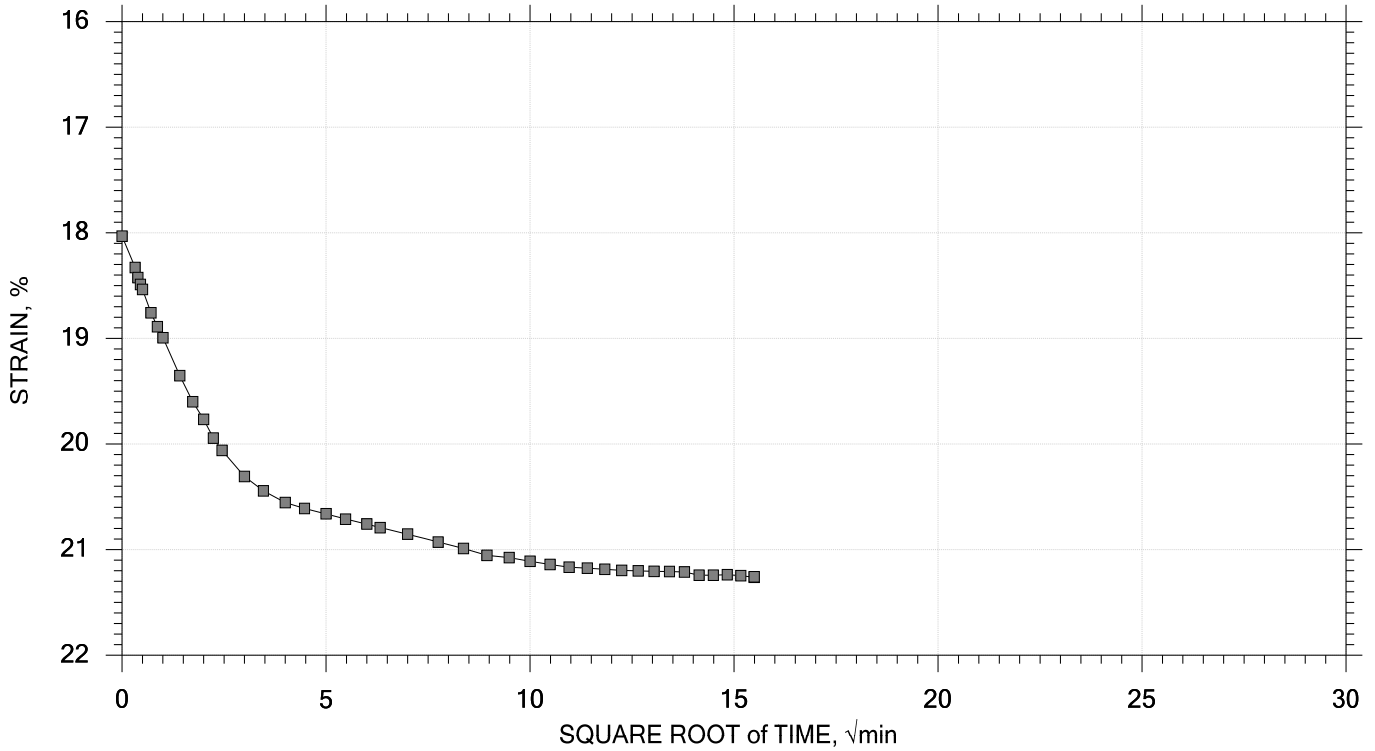
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 14 of 21

Stress: 8 tsf





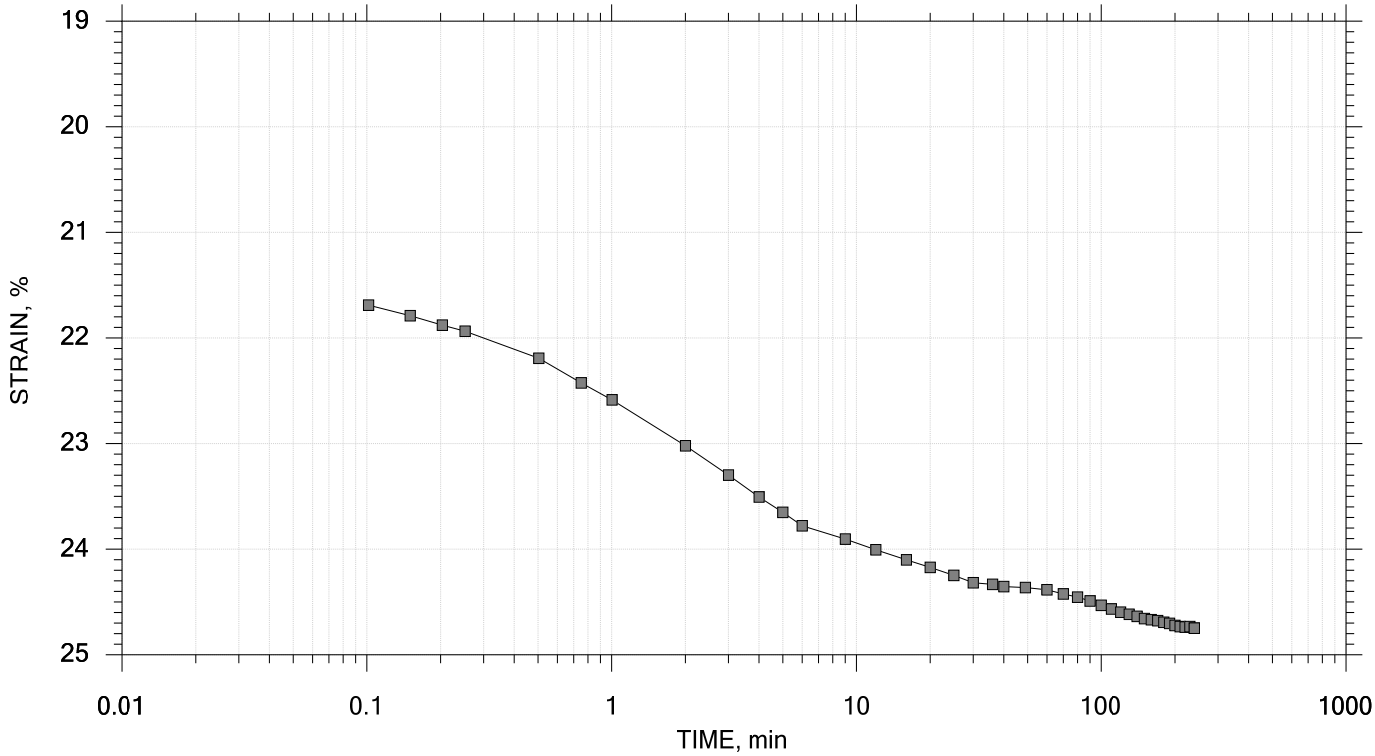
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	Boring No.: BB-MAR-104	Tested By: md	Checked By: njh
	Sample No.: U-1	Test Date: 05/30/17	Test No.: IP-1
	Depth: 20-21.4 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System O, Swell Pressure = 0.0679 tsf		

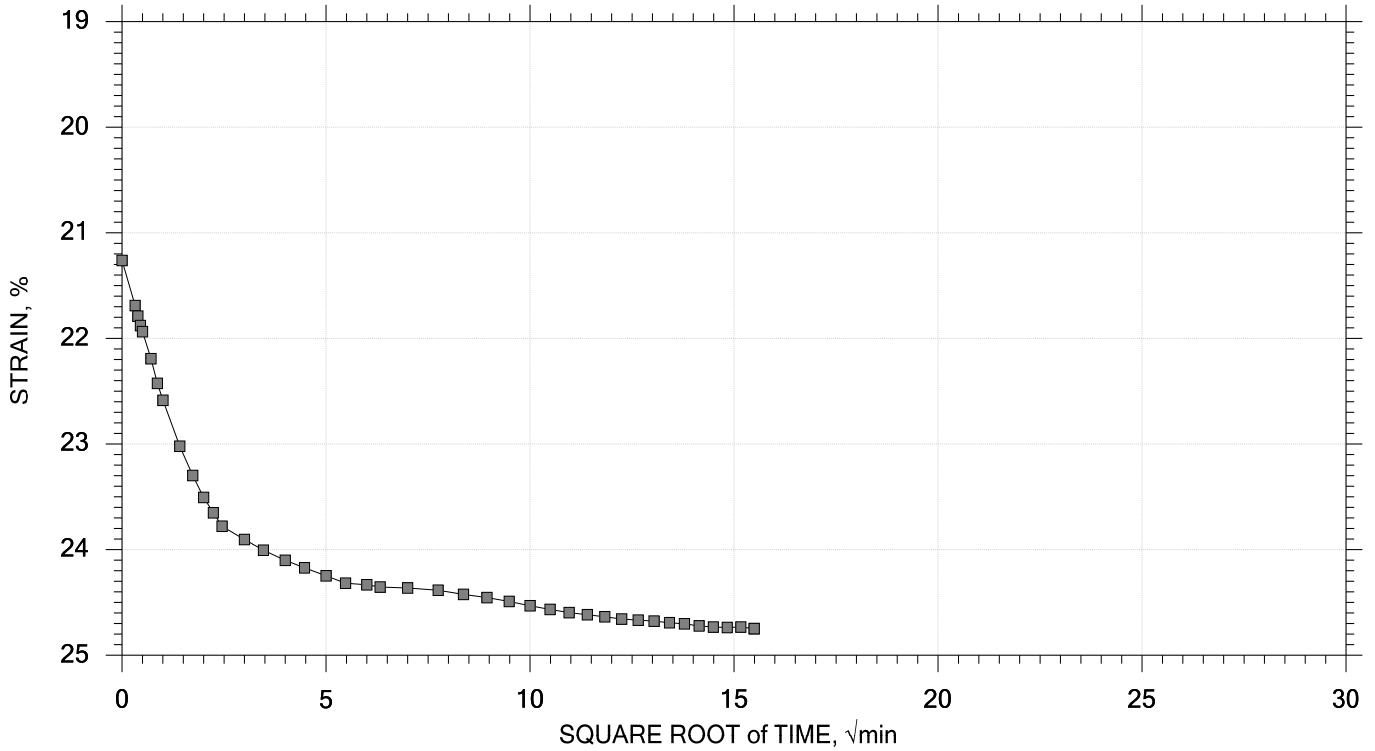
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 15 of 21

Stress: 16 tsf





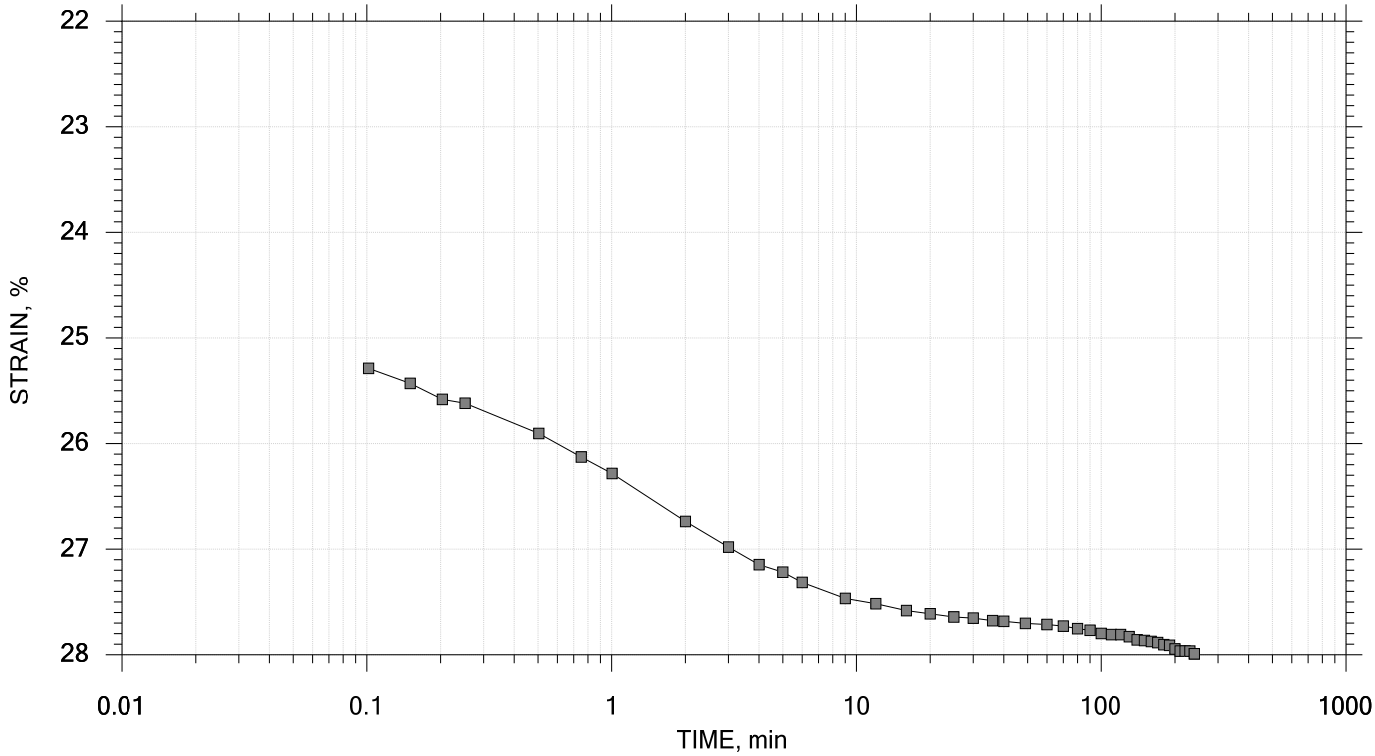
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	Boring No.: BB-MAR-104	Tested By: md	Checked By: njh
	Sample No.: U-1	Test Date: 05/30/17	Test No.: IP-1
	Depth: 20-21.4 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System O, Swell Pressure = 0.0679 tsf		

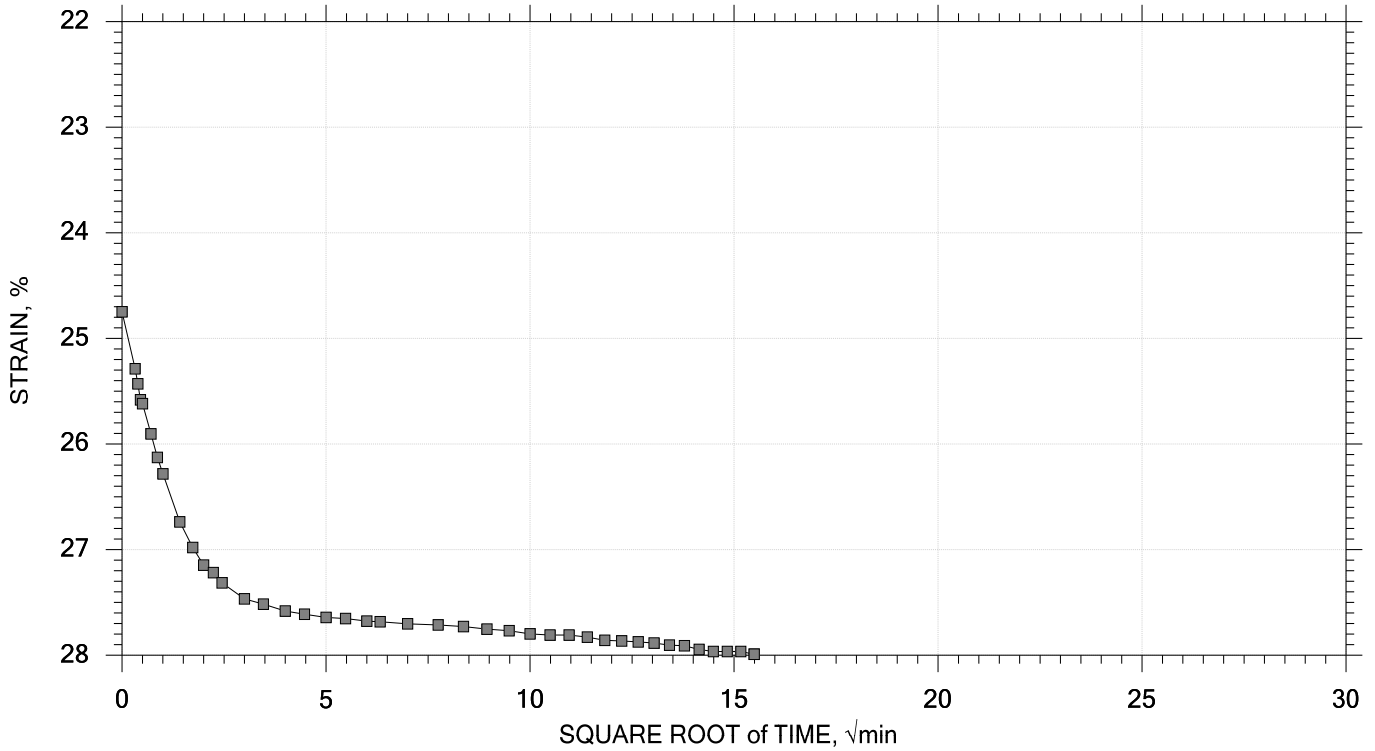
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 16 of 21

Stress: 32 tsf





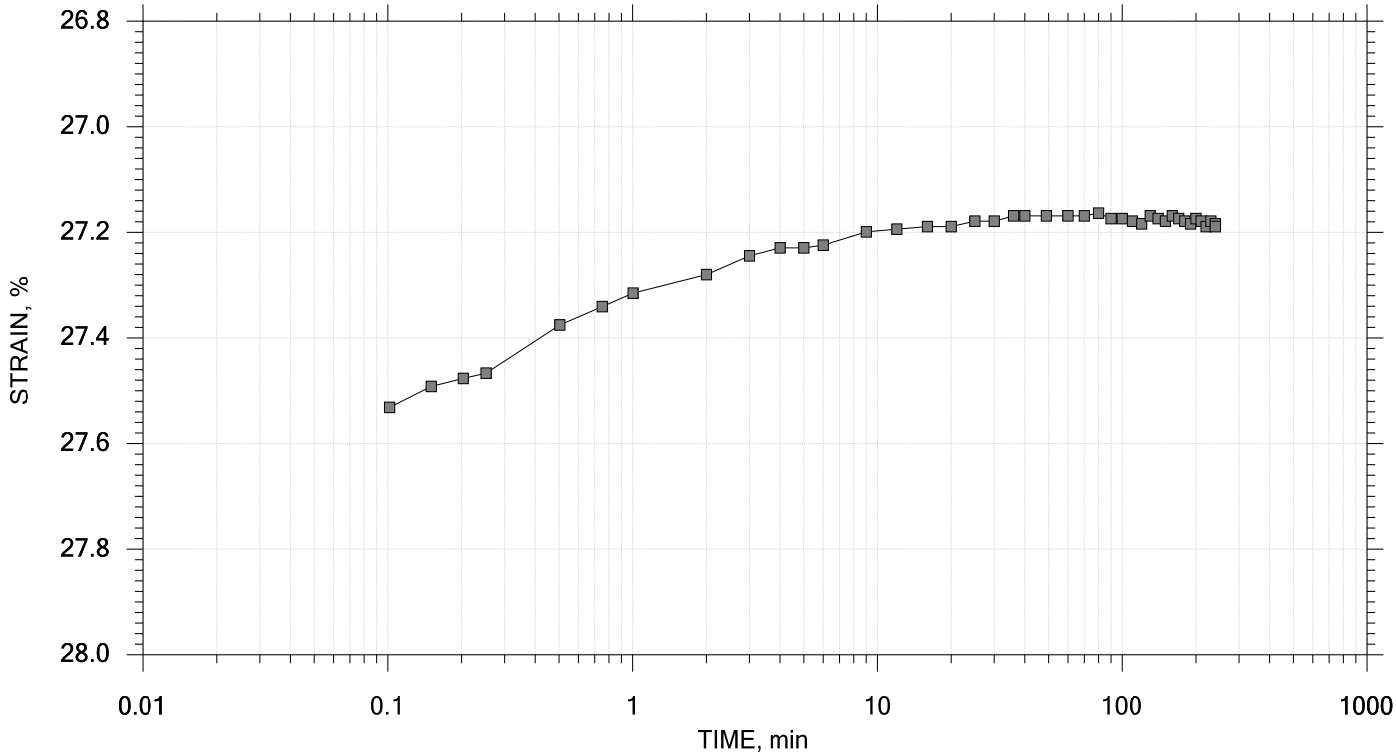
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	Boring No.: BB-MAR-104	Tested By: md	Checked By: njh
	Sample No.: U-1	Test Date: 05/30/17	Test No.: IP-1
	Depth: 20-21.4 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System O, Swell Pressure = 0.0679 tsf		

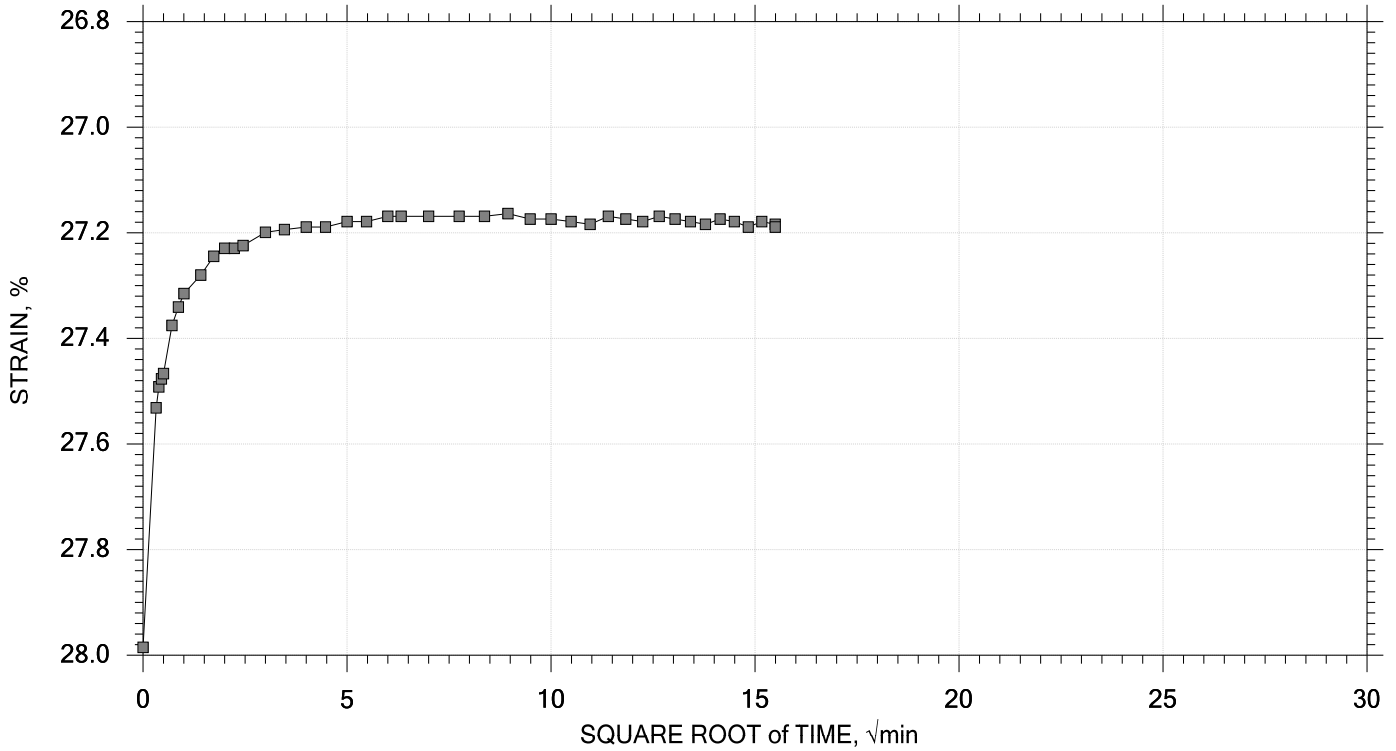
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 17 of 21

Stress: 8 tsf





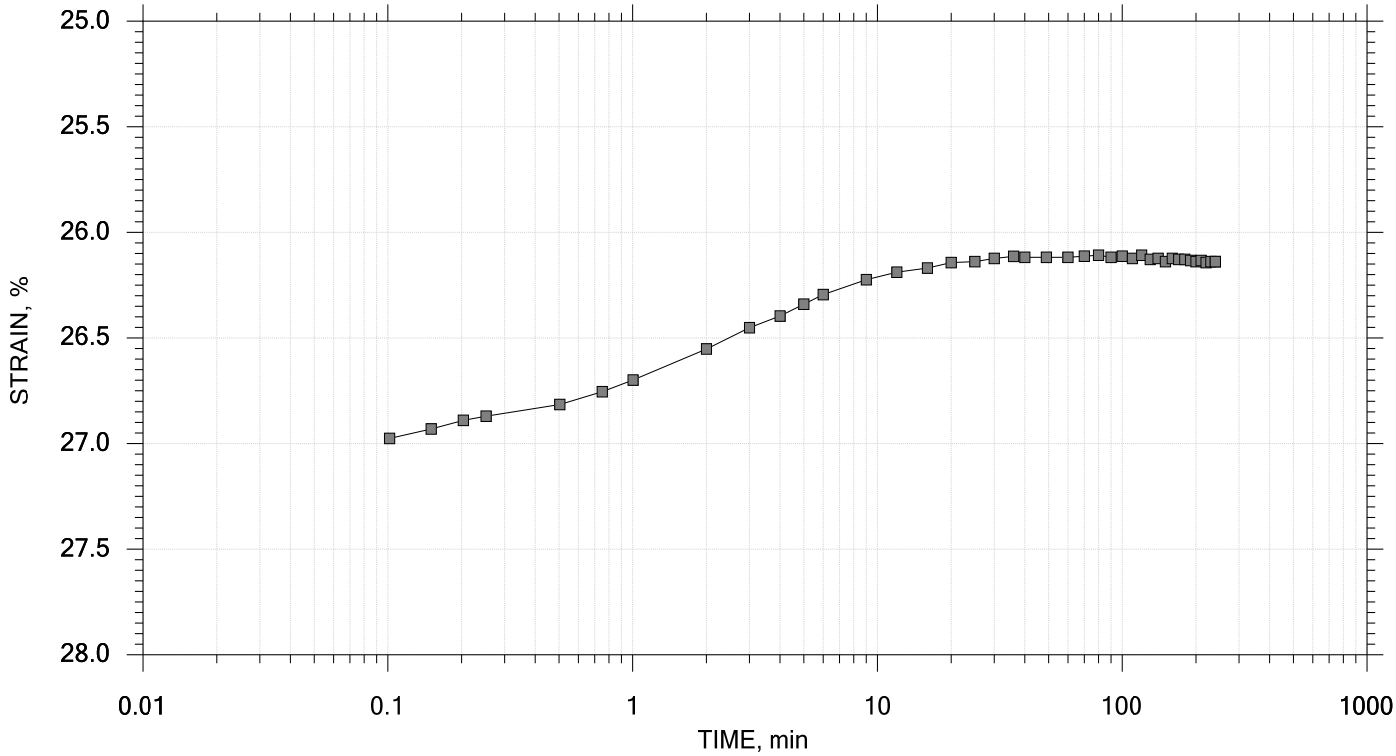
GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-104	Tested By: md	Checked By: njh
	Sample No.: U-1	Test Date: 05/30/17	Test No.: IP-1
	Depth: 20-21.4 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System O, Swell Pressure = 0.0679 tsf		

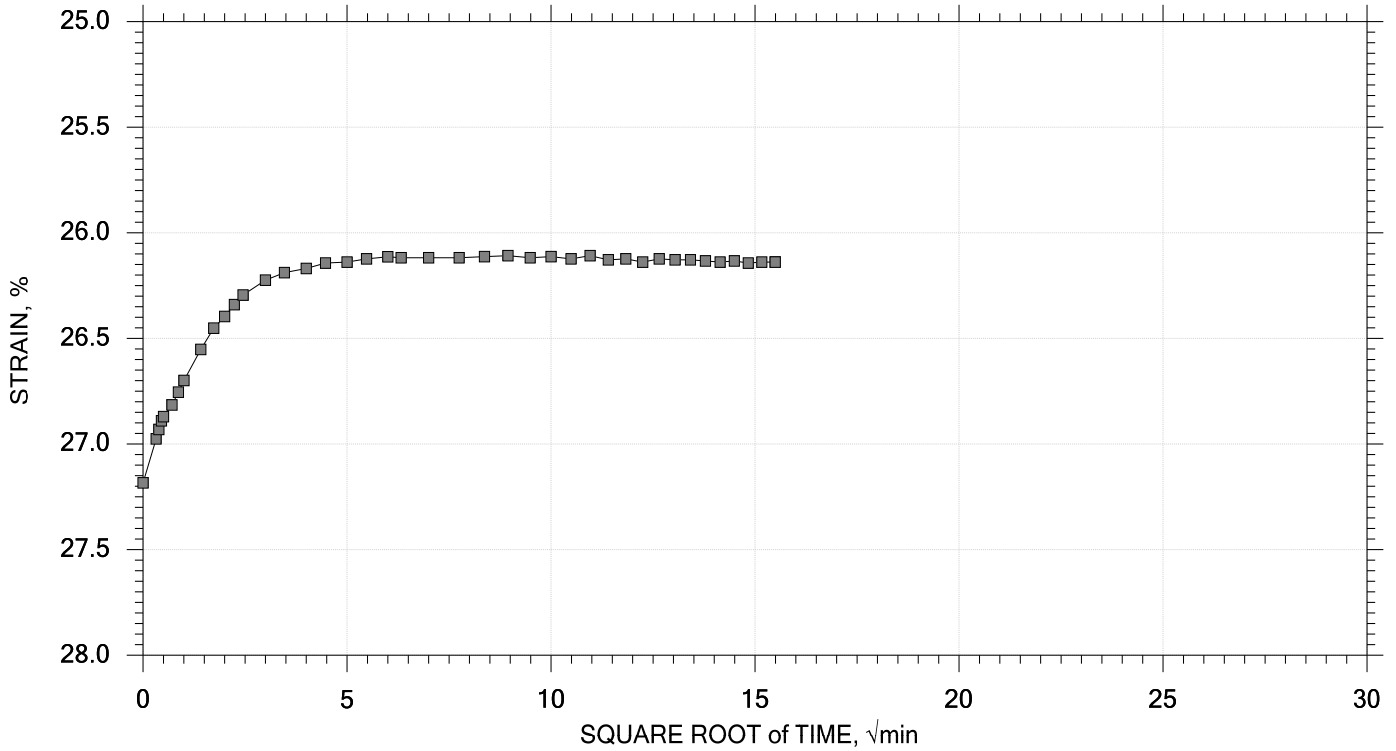
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 18 of 21

Stress: 2 tsf





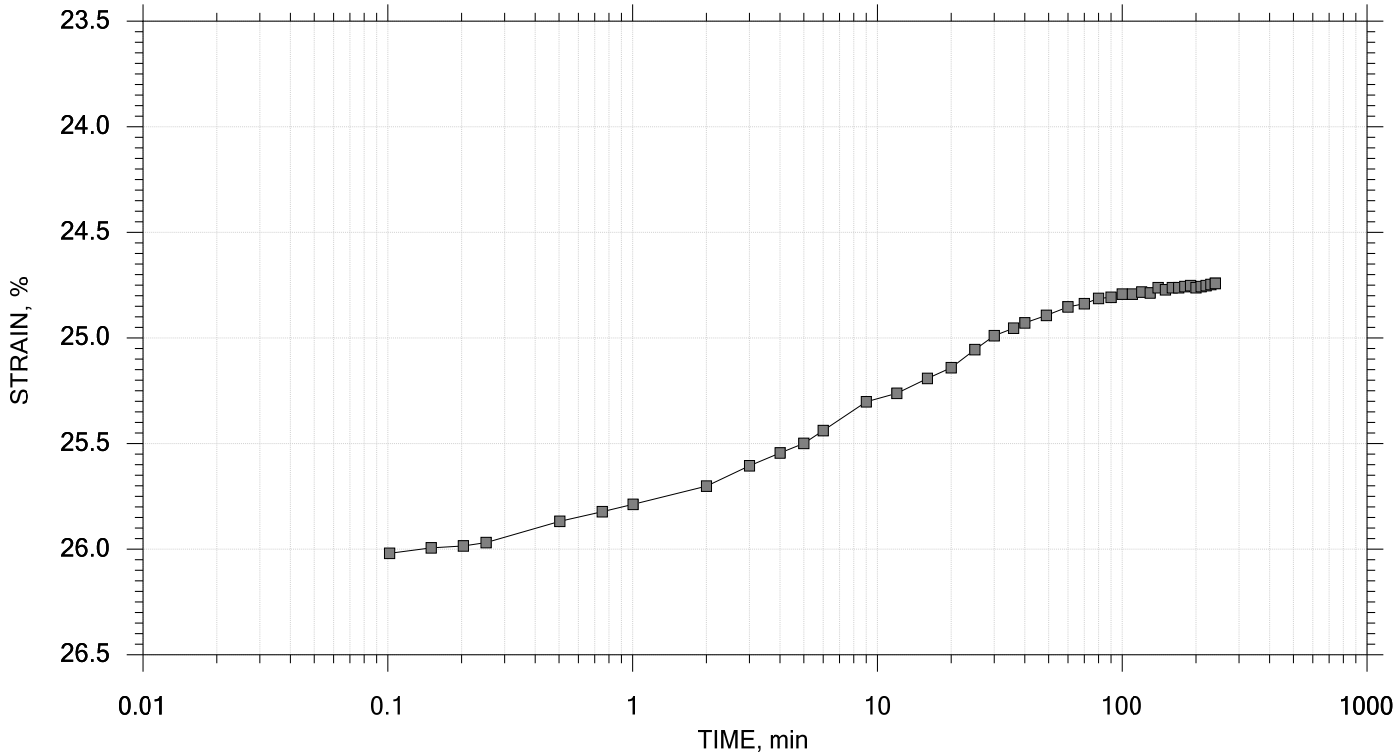
GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-104	Tested By: md	Checked By: njh
	Sample No.: U-1	Test Date: 05/30/17	Test No.: IP-1
	Depth: 20-21.4 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System O, Swell Pressure = 0.0679 tsf		

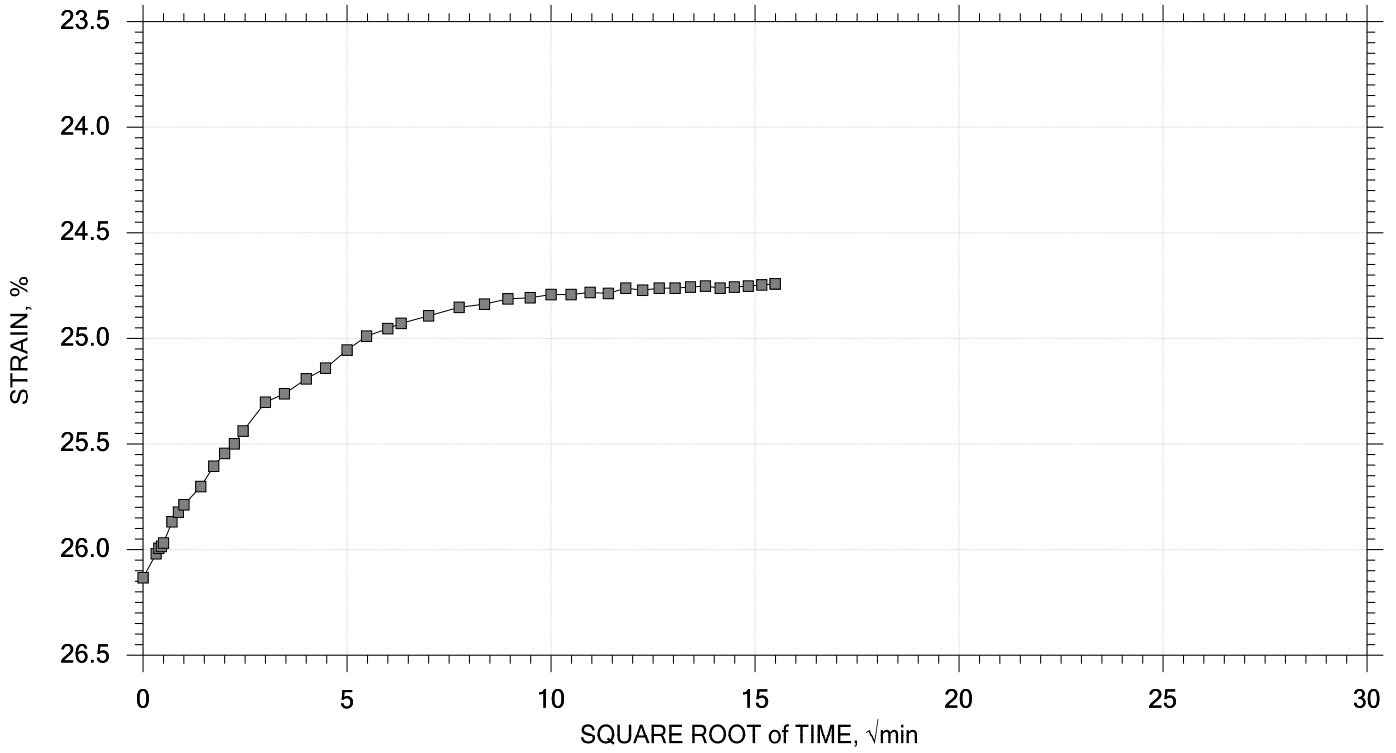
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 19 of 21

Stress: 0.5 tsf





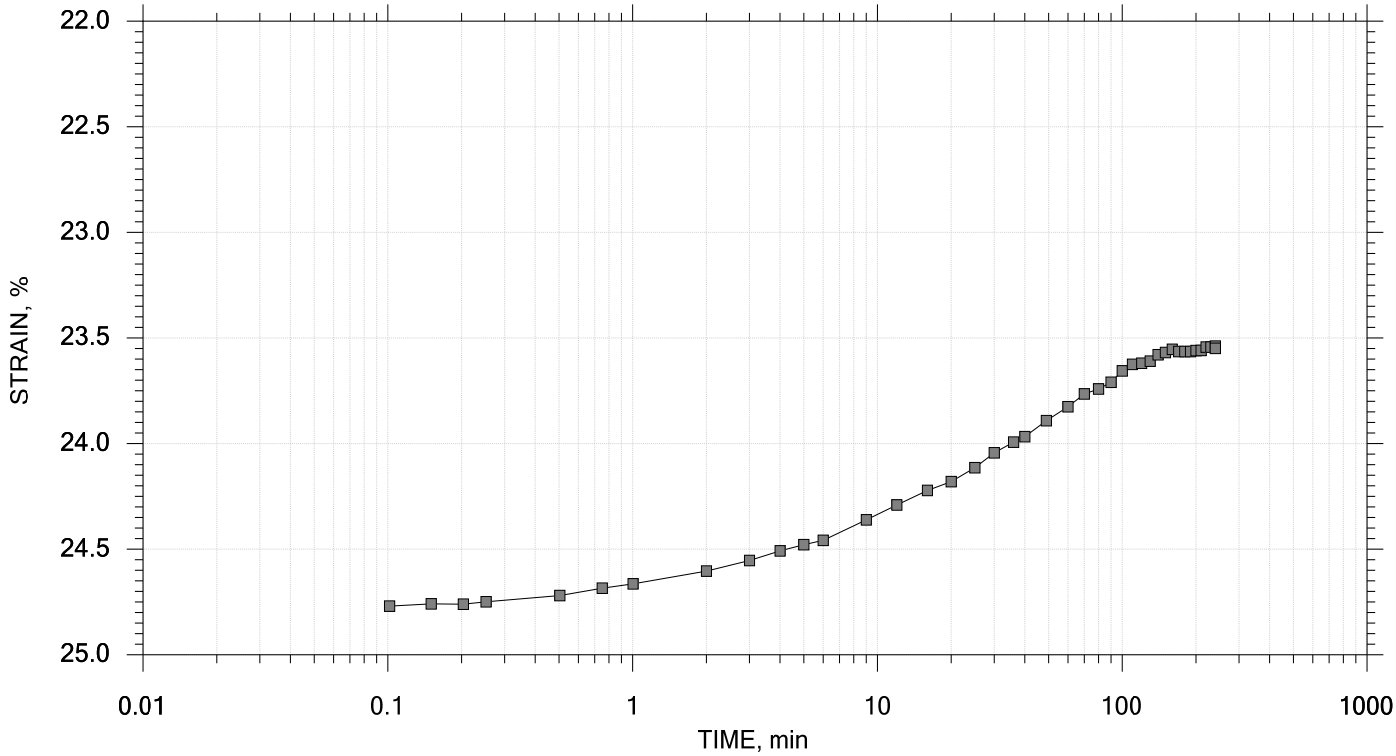
GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-104	Tested By: md	Checked By: njh
	Sample No.: U-1	Test Date: 05/30/17	Test No.: IP-1
	Depth: 20-21.4 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System O, Swell Pressure = 0.0679 tsf		

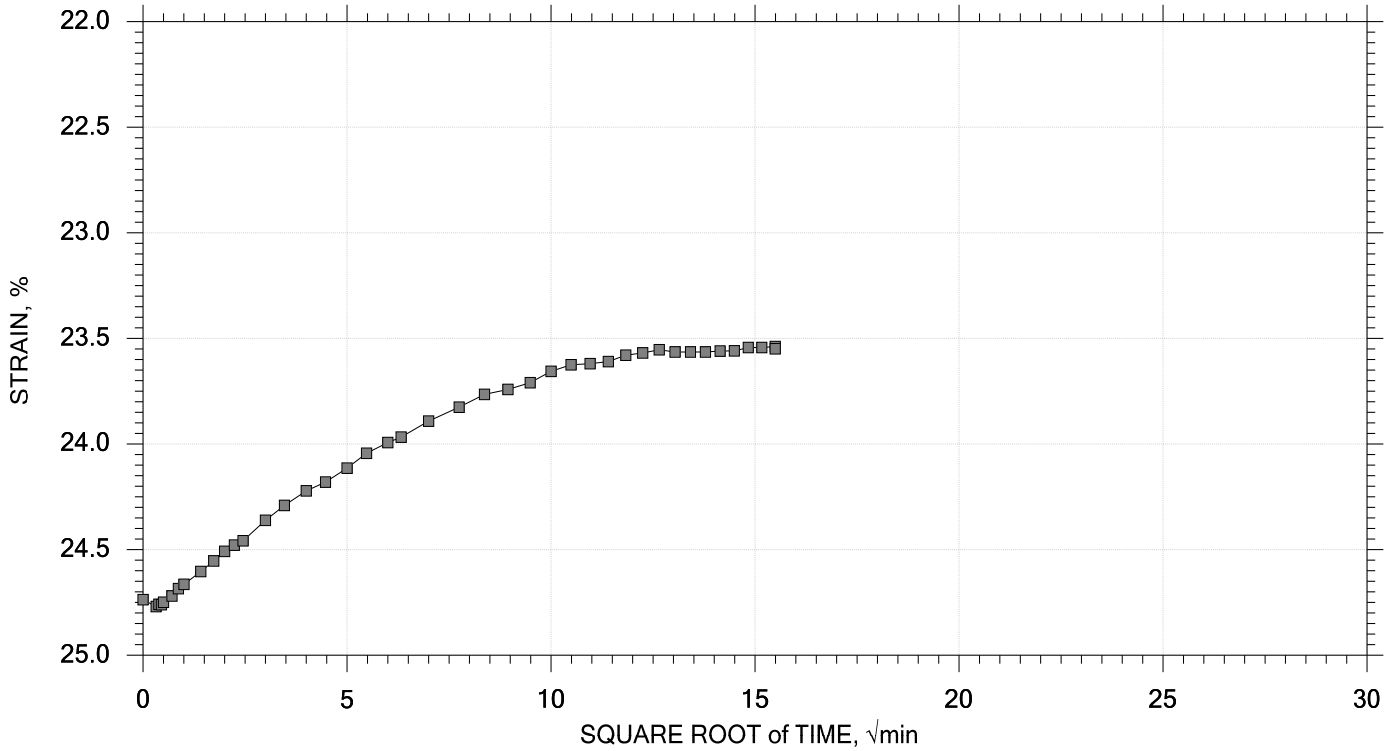
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 20 of 21

Stress: 0.125 tsf





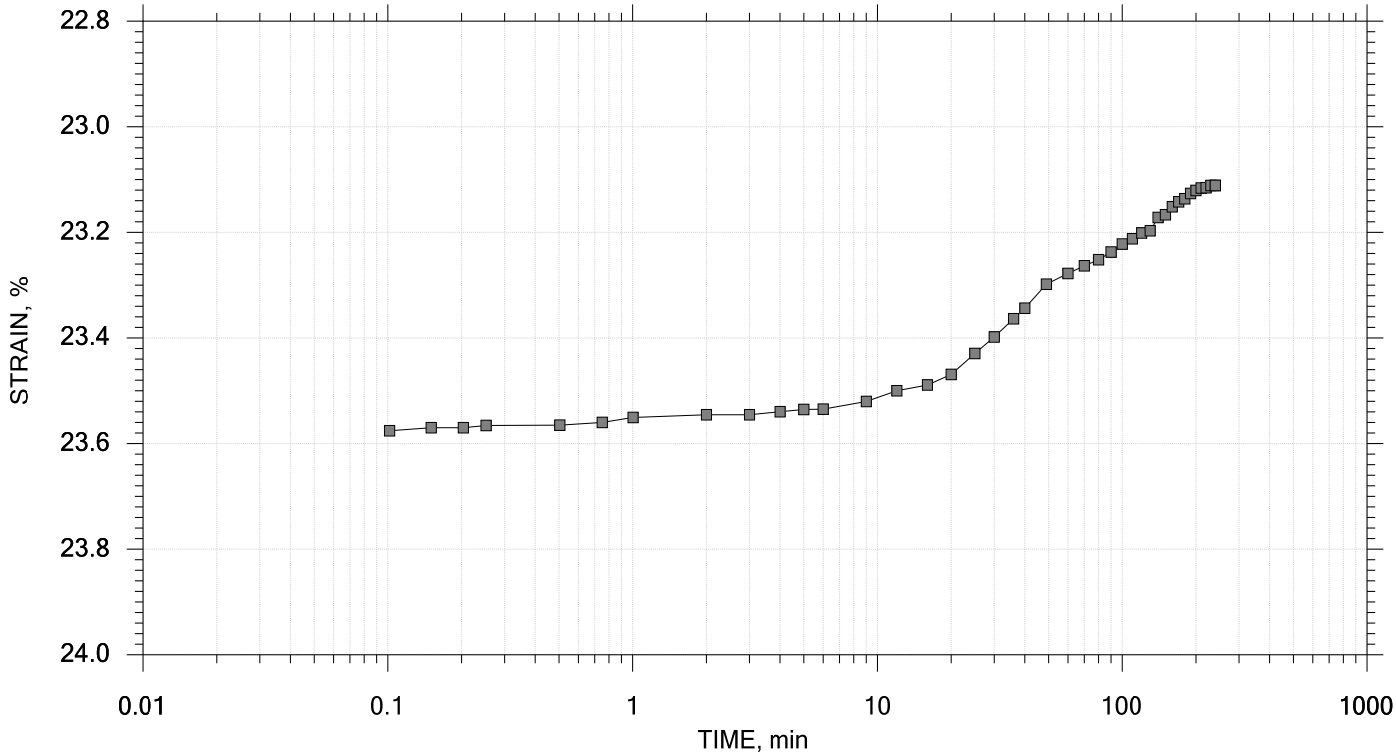
GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-104	Tested By: md	Checked By: njh
	Sample No.: U-1	Test Date: 05/30/17	Test No.: IP-1
	Depth: 20-21.4 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System O, Swell Pressure = 0.0679 tsf		

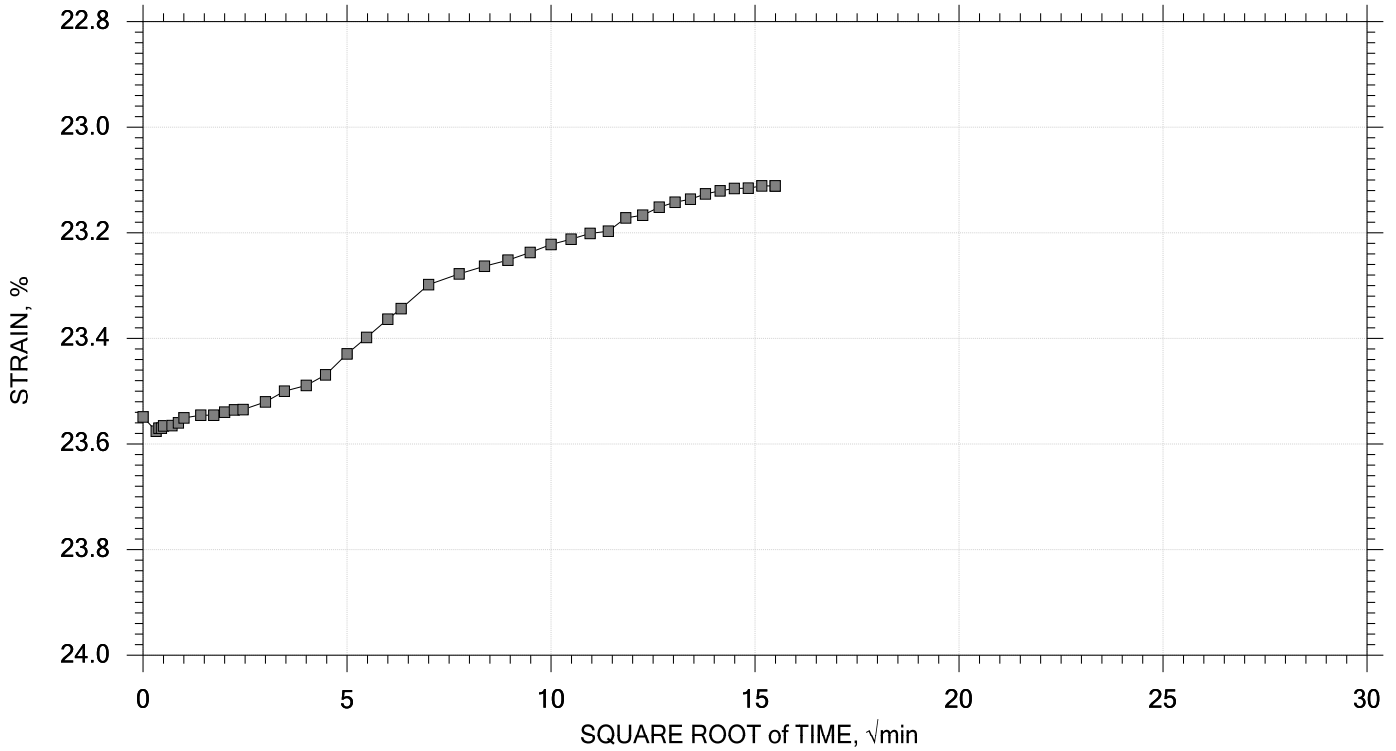
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 21 of 21

Stress: 0.0625 tsf

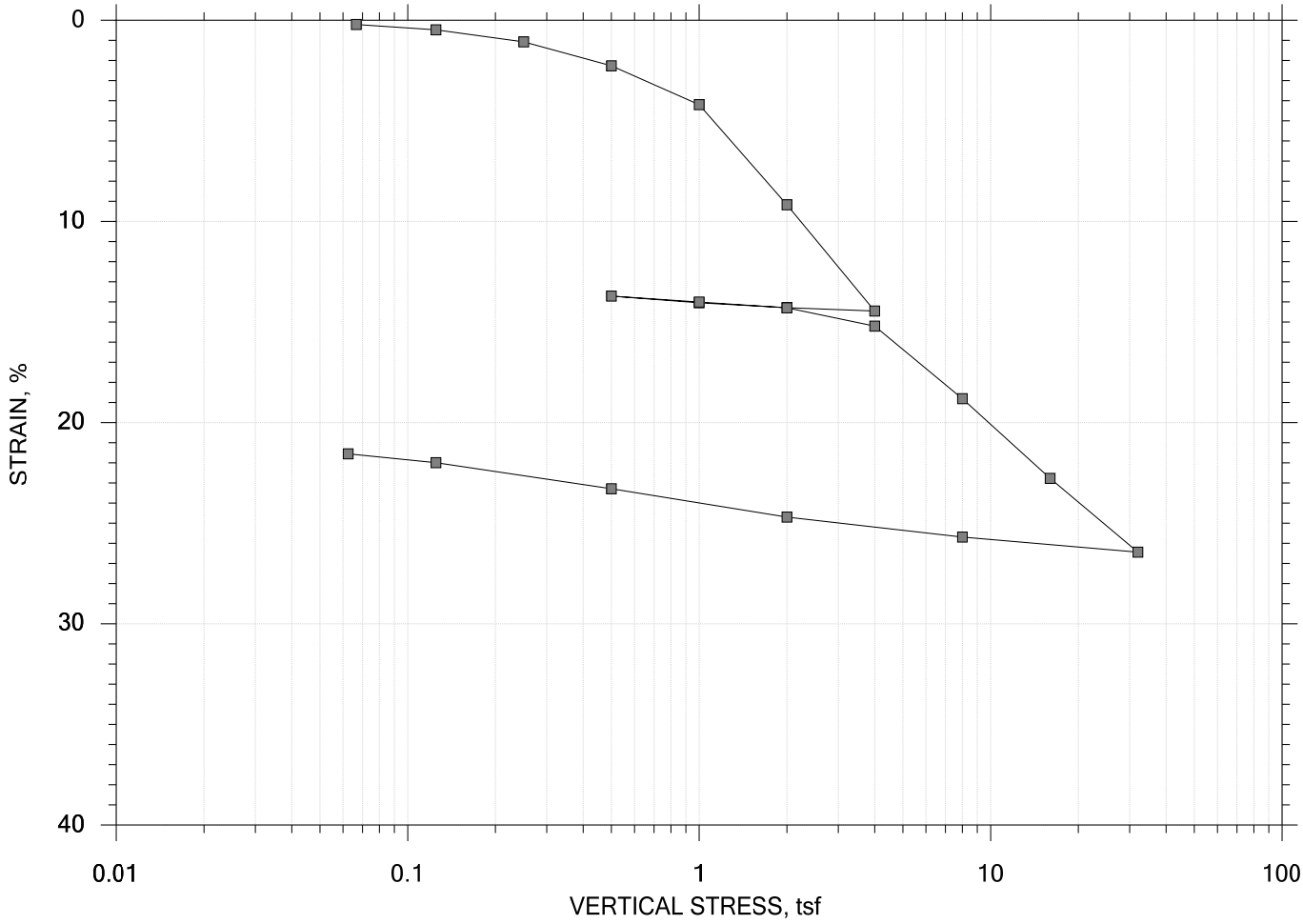


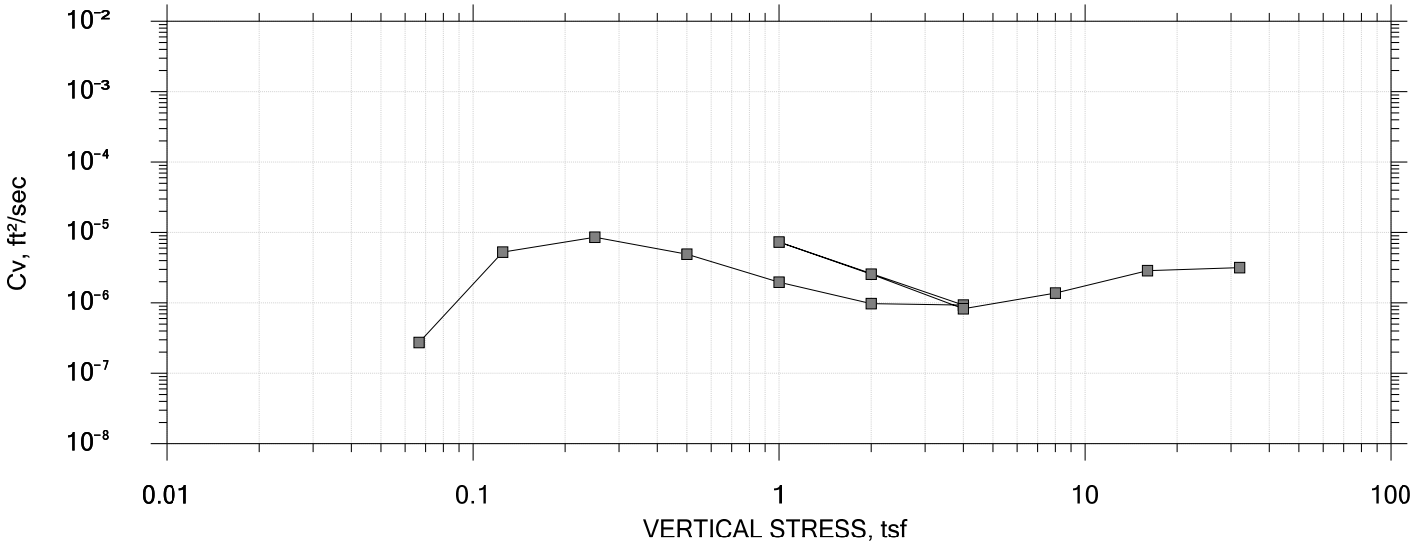


GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-104	Tested By: md	Checked By: njh
	Sample No.: U-1	Test Date: 05/30/17	Test No.: IP-1
	Depth: 20-21.4 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System O, Swell Pressure = 0.0679 tsf		

One-Dimensional Consolidation by ASTM D2435 - Method B

SUMMARY REPORT

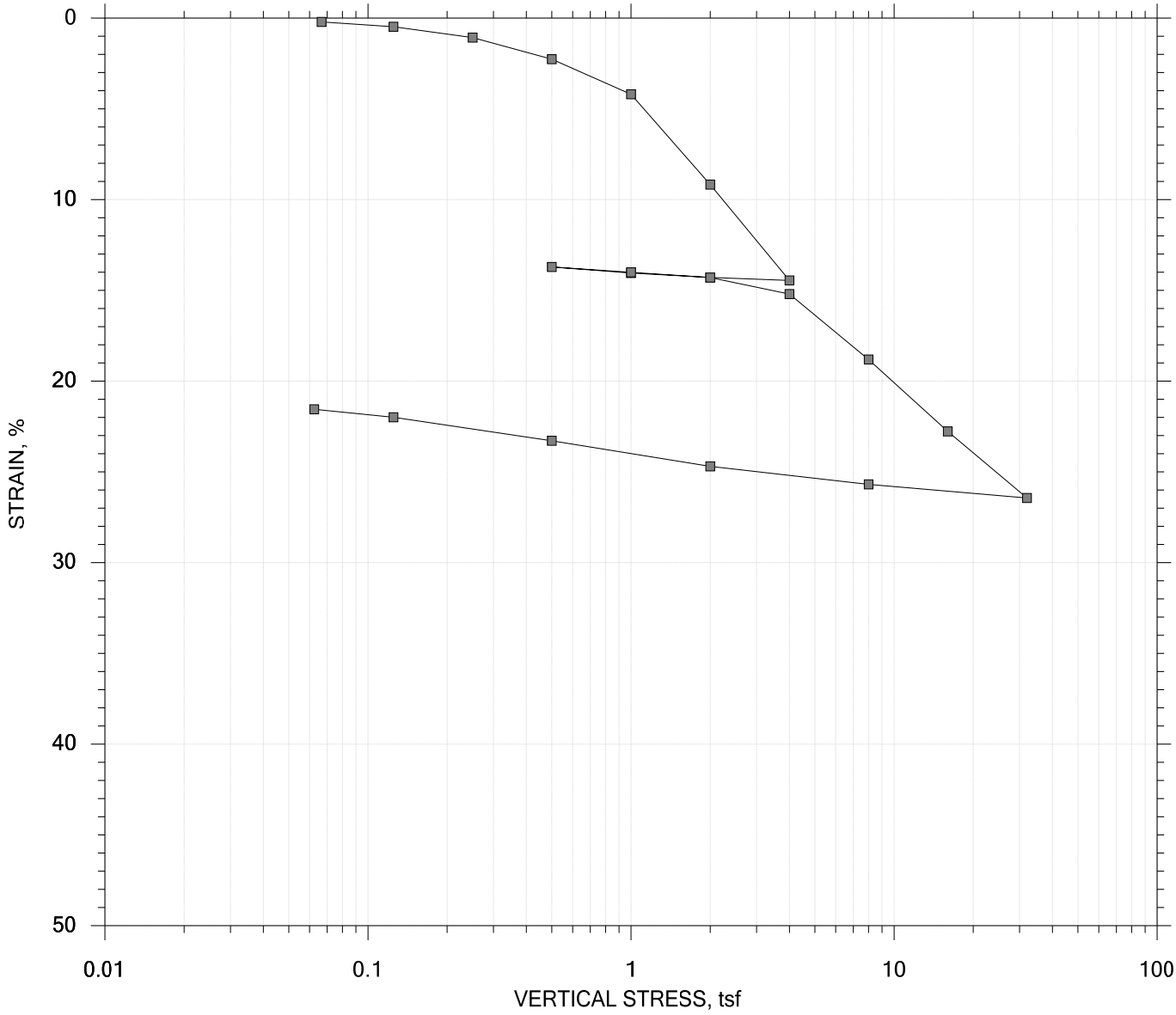




GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-202	Tested By: md	Checked By: njh
	Sample No.: U1	Test Date: 10/16/17	Test No.: IP-2
	Depth: 35-37 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System Y, Swell Pressure = 0.0666 tsf		
	Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method B

SUMMARY REPORT



				Before Test	After Test
Current Vertical Effective Stress: ---			Water Content, %	35.20	20.04
Preconsolidation Stress: ---			Dry Unit Weight, pcf	86.016	110.28
Compression Ratio: ---			Saturation, %	97.78	100.00
Diameter: 2.5 in		Height: 1 in	Void Ratio	0.98	0.55
LL: ---	PL: ---	PI: ---	GS: 2.73		

GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-202	Tested By: md	Checked By: njh
	Sample No.: U1	Test Date: 10/16/17	Test No.: IP-2
	Depth: 35-37 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System Y, Swell Pressure = 0.0666 tsf		
	Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method B

Project: Marsh Bridge | Location: Tremont/Southwest Harbor, ME | Project No.: GTX-306503
Boring No.: BB-MAR-202 | Tested By: md | Checked By: njh
Sample No.: U1 | Test Date: 10/16/17 | Depth: 35-37 ft
Test No.: IP-2 | Sample Type: intact | Elevation: ---

Soil Description: Moist, gray silty clay
Remarks: System Y, Swell Pressure = 0.0666 tsf

Estimated Specific Gravity: 2.73 | Liquid Limit: --- | Specimen Diameter: 2.50 in
Initial Void Ratio: 0.984 | Plastic Limit: --- | Initial Height: 1.00 in
Final Void Ratio: 0.548 | Plasticity Index: --- | Final Height: 0.78 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	A-1422	RING		A-1488
Wt. Container + Wet Soil, gm	144.92	260.58	243.77	142.00
Wt. Container + Dry Soil, gm	105.71	221.56	221.56	119.67
Wt. Container, gm	8.1100	110.73	110.73	8.2200
Wt. Dry Soil, gm	97.600	110.83	110.83	111.45
Water Content, %	40.17	35.20	20.04	20.04
Void Ratio	---	0.984	0.548	---
Degree of Saturation, %	---	97.78	100.00	---
Dry Unit Weight, pcf	---	86.016	110.28	---

Note: Specific Gravity and Void Ratios are calculated assuming the degree of saturation equals 100% at the end of the test. Therefore, values may not represent actual values for the specimen.

One-Dimensional Consolidation by ASTM D2435 - Method B

Project: Marsh Bridge | Location: Tremont/Southwest Harbor, ME | Project No.: GTX-306503
Boring No.: BB-MAR-202 | Tested By: md | Checked By: njh
Sample No.: U1 | Test Date: 10/16/17 | Depth: 35-37 ft
Test No.: IP-2 | Sample Type: intact | Elevation: ---

Soil Description: Moist, gray silty clay
Remarks: System Y, Swell Pressure = 0.0666 tsf

Displacement at End of Increment

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	Sq.Rt T90 min	Cv ft²/sec	Mv 1/tsf	k ft/day
1	0.0666	0.002084	0.980	0.208	80.001	3.06e-007	3.13e-002	2.58e-005
2	0.125	0.004742	0.975	0.474	4.635	5.26e-006	4.55e-002	6.45e-004
3	0.250	0.01074	0.963	1.07	3.920	6.16e-006	4.80e-002	7.97e-004
4	0.500	0.02261	0.940	2.26	5.877	4.04e-006	4.75e-002	5.17e-004
5	1.00	0.04190	0.901	4.19	12.791	1.80e-006	3.86e-002	1.87e-004
6	2.00	0.09171	0.802	9.17	19.288	1.11e-006	4.98e-002	1.49e-004
7	4.00	0.1445	0.698	14.5	22.408	8.52e-007	2.64e-002	6.07e-005
8	2.00	0.1429	0.701	14.3	2.997	6.00e-006	8.41e-004	1.36e-005
9	1.00	0.1404	0.706	14.0	4.758	3.80e-006	2.46e-003	2.52e-005
10	0.500	0.1371	0.712	13.7	26.077	6.98e-007	6.58e-003	1.24e-005
11	1.00	0.1400	0.707	14.0	2.230	8.16e-006	5.80e-003	1.28e-004
12	2.00	0.1429	0.701	14.3	7.003	2.58e-006	2.94e-003	2.05e-005
13	4.00	0.1520	0.683	15.2	21.287	8.38e-007	4.55e-003	1.03e-005
14	8.00	0.1881	0.611	18.8	12.767	1.32e-006	9.01e-003	3.22e-005
15	16.0	0.2277	0.532	22.8	4.974	3.09e-006	4.95e-003	4.14e-005
16	32.0	0.2644	0.460	26.4	3.425	4.07e-006	2.29e-003	2.52e-005
17	8.00	0.2569	0.475	25.7	2.944	4.56e-006	3.13e-004	3.84e-006
18	2.00	0.2469	0.494	24.7	6.274	2.19e-006	1.66e-003	9.78e-006
19	0.500	0.2329	0.522	23.3	24.245	5.85e-007	9.38e-003	1.48e-005
20	0.125	0.2199	0.548	22.0	70.683	2.08e-007	3.46e-002	1.94e-005
21	0.0625	0.2155	0.557	21.5	150.002	1.00e-007	7.04e-002	1.90e-005

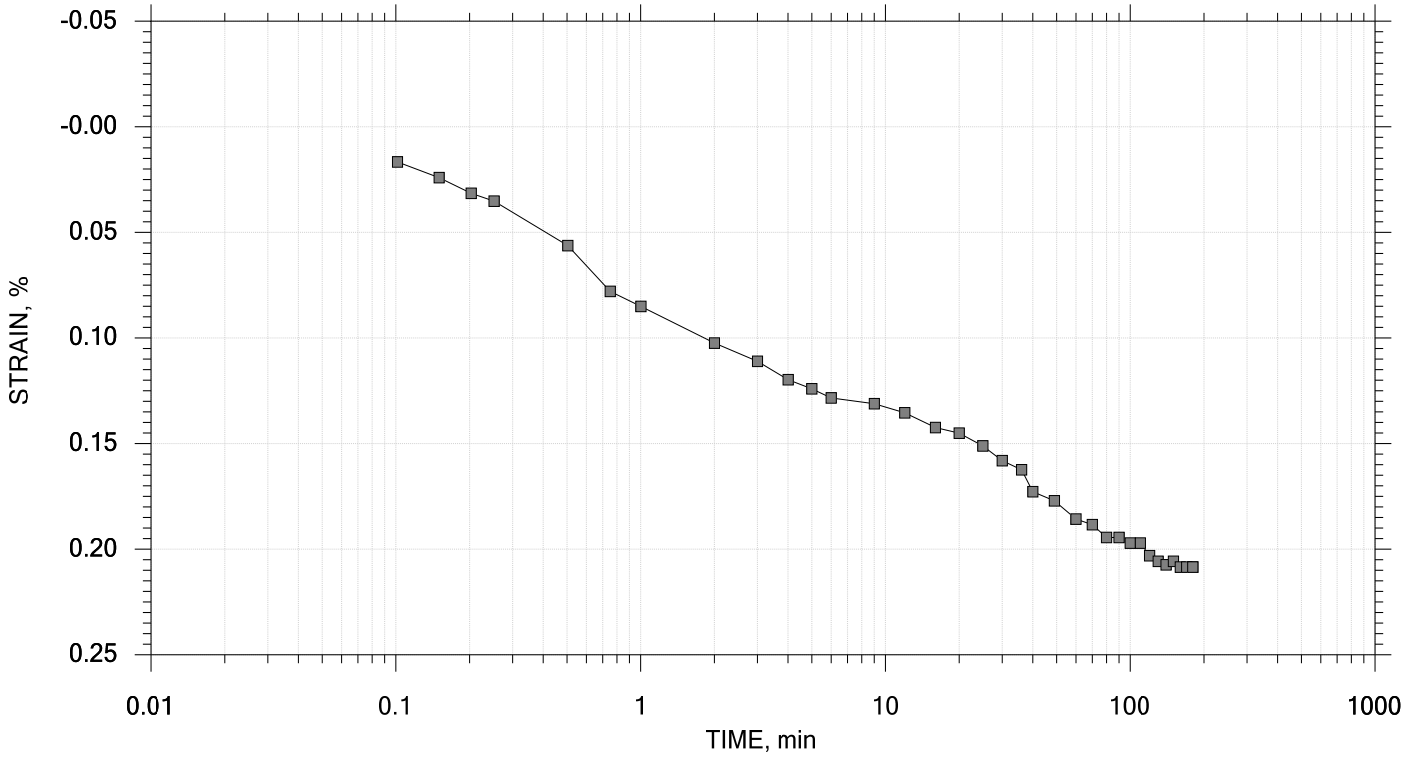
	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	Log T50 min	Cv ft²/sec	Mv 1/tsf	k ft/day	Ca %
1	0.0666	0.002084	0.980	0.208	0.000	0.00e+000	3.13e-002	0.00e+000	0.00e+000
2	0.125	0.004742	0.975	0.474	1.354	4.18e-006	4.55e-002	5.13e-004	0.00e+000
3	0.250	0.01074	0.963	1.07	0.616	9.11e-006	4.80e-002	1.18e-003	0.00e+000
4	0.500	0.02261	0.940	2.26	0.000	0.00e+000	4.75e-002	0.00e+000	0.00e+000
5	1.00	0.04190	0.901	4.19	2.537	2.10e-006	3.86e-002	2.19e-004	0.00e+000
6	2.00	0.09171	0.802	9.17	5.432	9.14e-007	4.98e-002	1.23e-004	0.00e+000
7	4.00	0.1445	0.698	14.5	4.487	9.88e-007	2.64e-002	7.04e-005	0.00e+000
8	2.00	0.1429	0.701	14.3	0.000	0.00e+000	8.41e-004	0.00e+000	0.00e+000
9	1.00	0.1404	0.706	14.0	0.000	0.00e+000	2.46e-003	0.00e+000	0.00e+000
10	0.500	0.1371	0.712	13.7	0.000	0.00e+000	6.58e-003	0.00e+000	0.00e+000
11	1.00	0.1400	0.707	14.0	0.000	0.00e+000	5.80e-003	0.00e+000	0.00e+000
12	2.00	0.1429	0.701	14.3	0.000	0.00e+000	2.94e-003	0.00e+000	0.00e+000
13	4.00	0.1520	0.683	15.2	0.000	0.00e+000	4.55e-003	0.00e+000	0.00e+000
14	8.00	0.1881	0.611	18.8	2.717	1.45e-006	9.01e-003	3.51e-005	0.00e+000
15	16.0	0.2277	0.532	22.8	1.372	2.61e-006	4.95e-003	3.48e-005	0.00e+000
16	32.0	0.2644	0.460	26.4	1.252	2.59e-006	2.29e-003	1.60e-005	0.00e+000
17	8.00	0.2569	0.475	25.7	0.000	0.00e+000	3.13e-004	0.00e+000	0.00e+000
18	2.00	0.2469	0.494	24.7	1.911	1.67e-006	1.66e-003	7.46e-006	0.00e+000
19	0.500	0.2329	0.522	23.3	6.802	4.84e-007	9.38e-003	1.23e-005	0.00e+000
20	0.125	0.2199	0.548	22.0	0.000	0.00e+000	3.46e-002	0.00e+000	0.00e+000
21	0.0625	0.2155	0.557	21.5	21.376	1.63e-007	7.04e-002	3.10e-005	0.00e+000

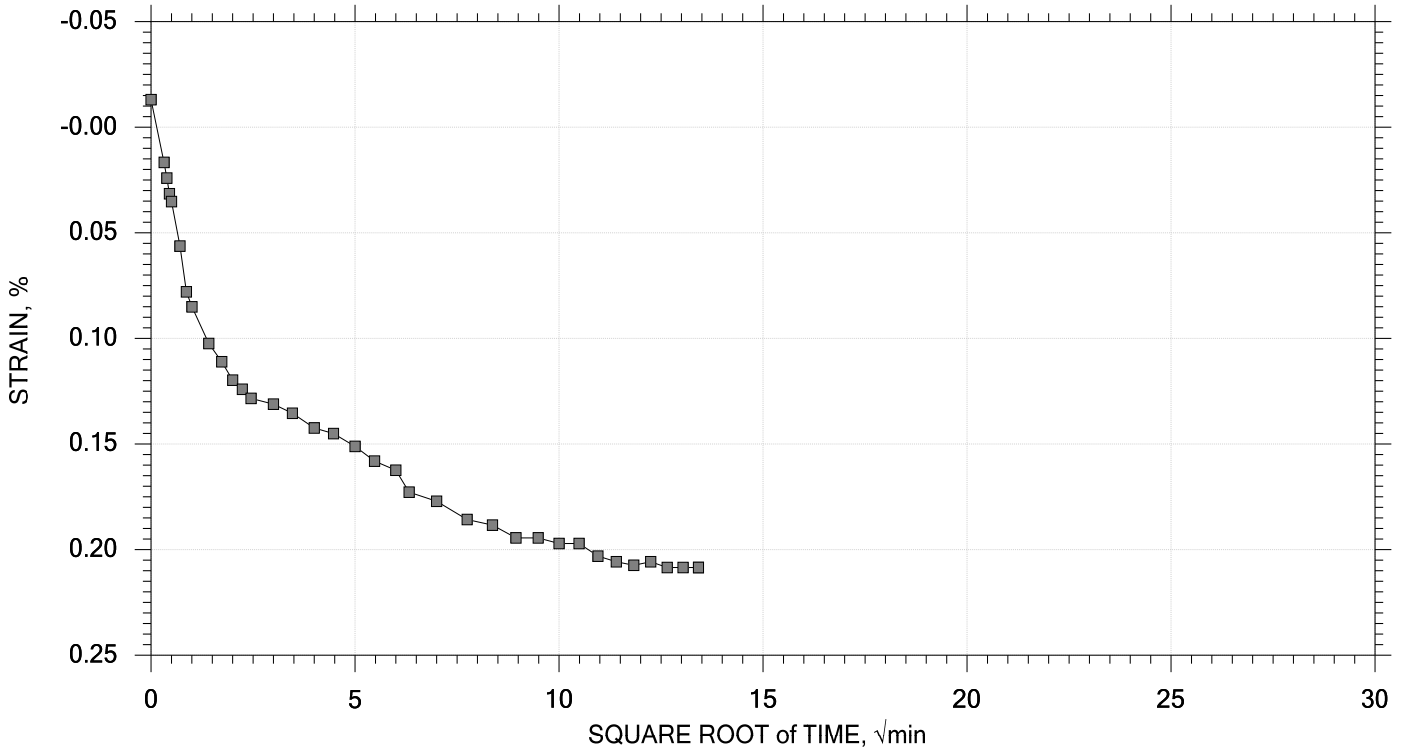
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Volume Step 1 of 21

Stress: 0.066578 tsf





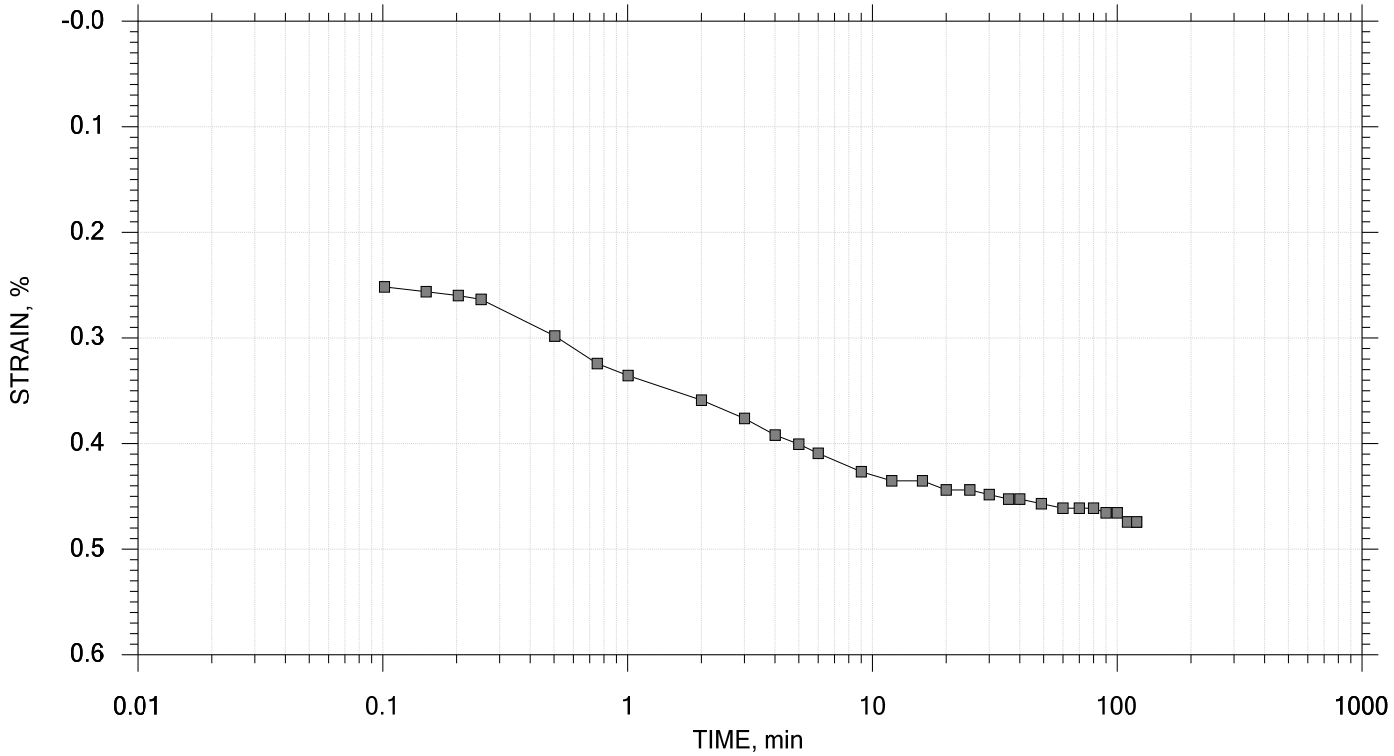
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	Boring No.: BB-MAR-202	Tested By: md	Checked By: njh
	Sample No.: U1	Test Date: 10/16/17	Test No.: IP-2
	Depth: 35-37 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System Y, Swell Pressure = 0.0666 tsf		

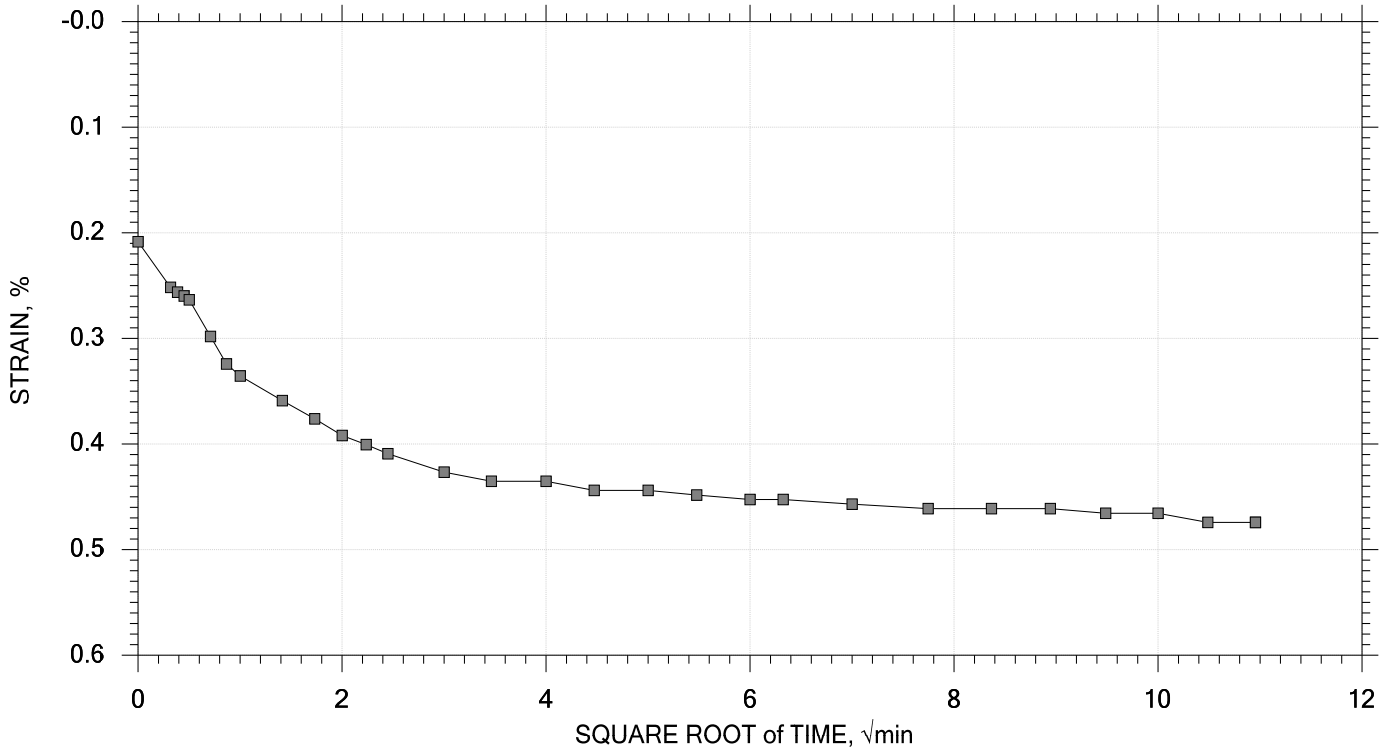
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 2 of 21

Stress: 0.125 tsf





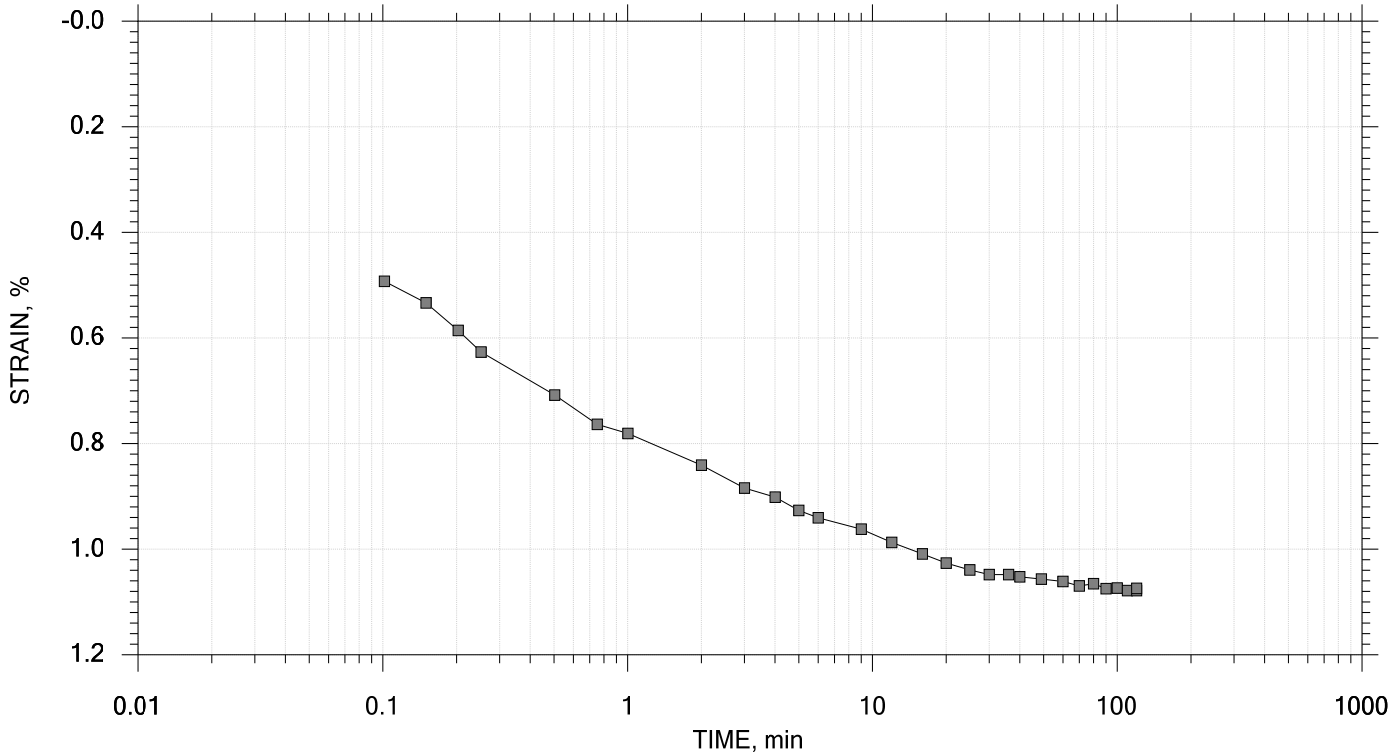
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	Boring No.: BB-MAR-202	Tested By: md	Checked By: njh
	Sample No.: U1	Test Date: 10/16/17	Test No.: IP-2
	Depth: 35-37 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System Y, Swell Pressure = 0.0666 tsf		

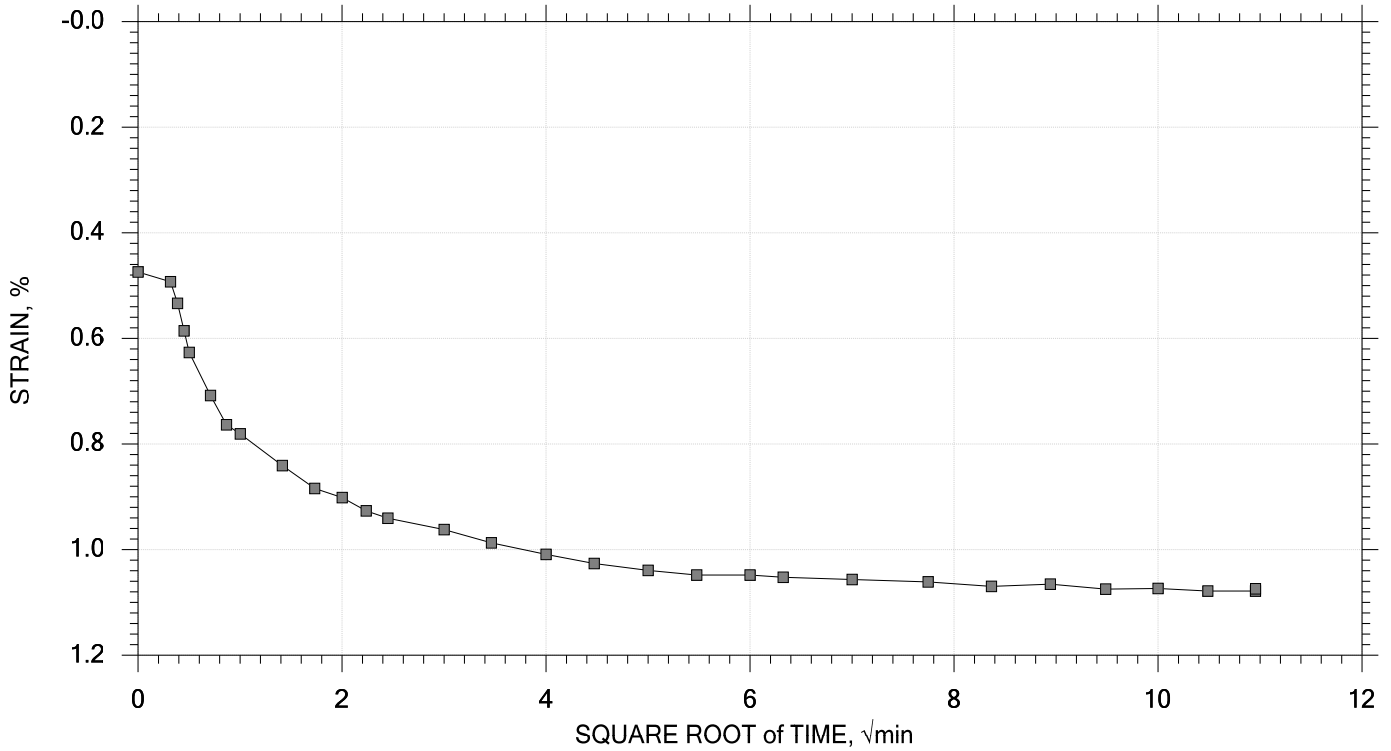
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 3 of 21

Stress: 0.25 tsf





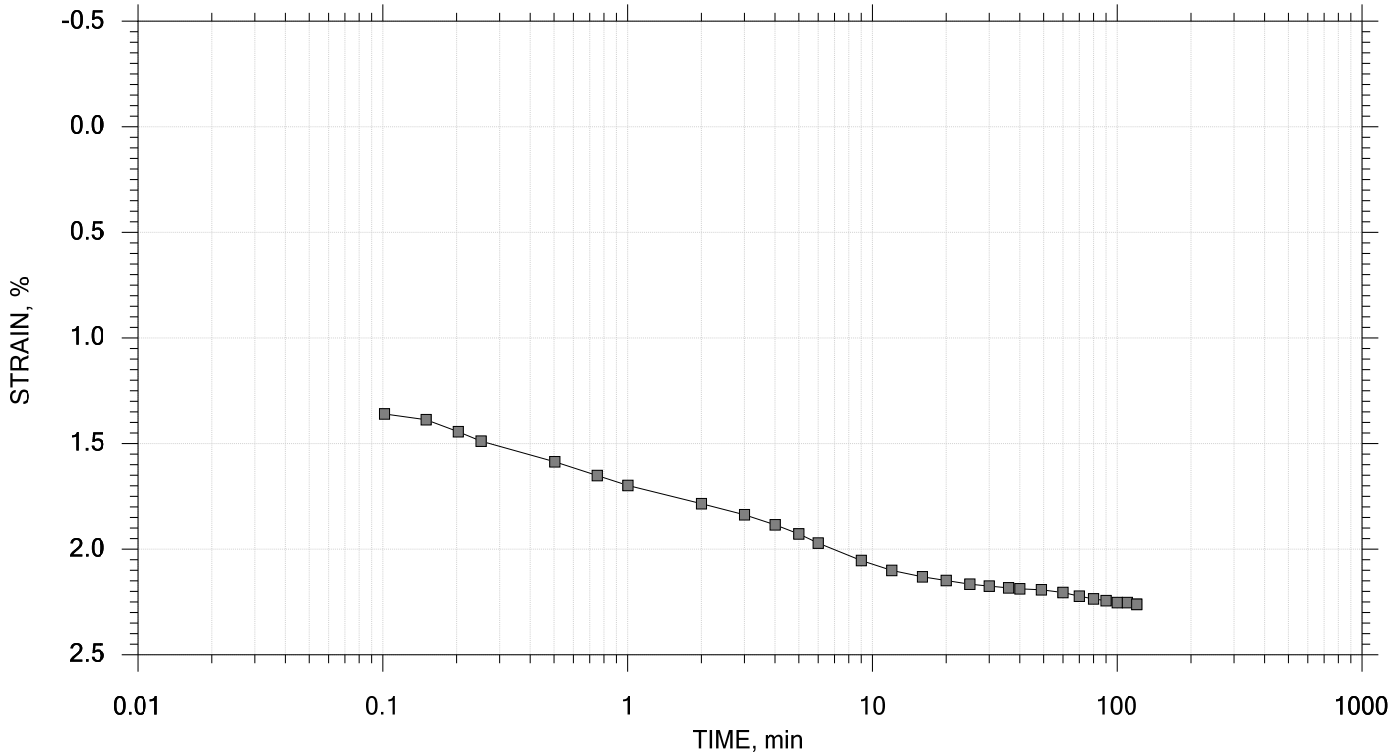
GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-202	Tested By: md	Checked By: njh
	Sample No.: U1	Test Date: 10/16/17	Test No.: IP-2
	Depth: 35-37 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System Y, Swell Pressure = 0.0666 tsf		

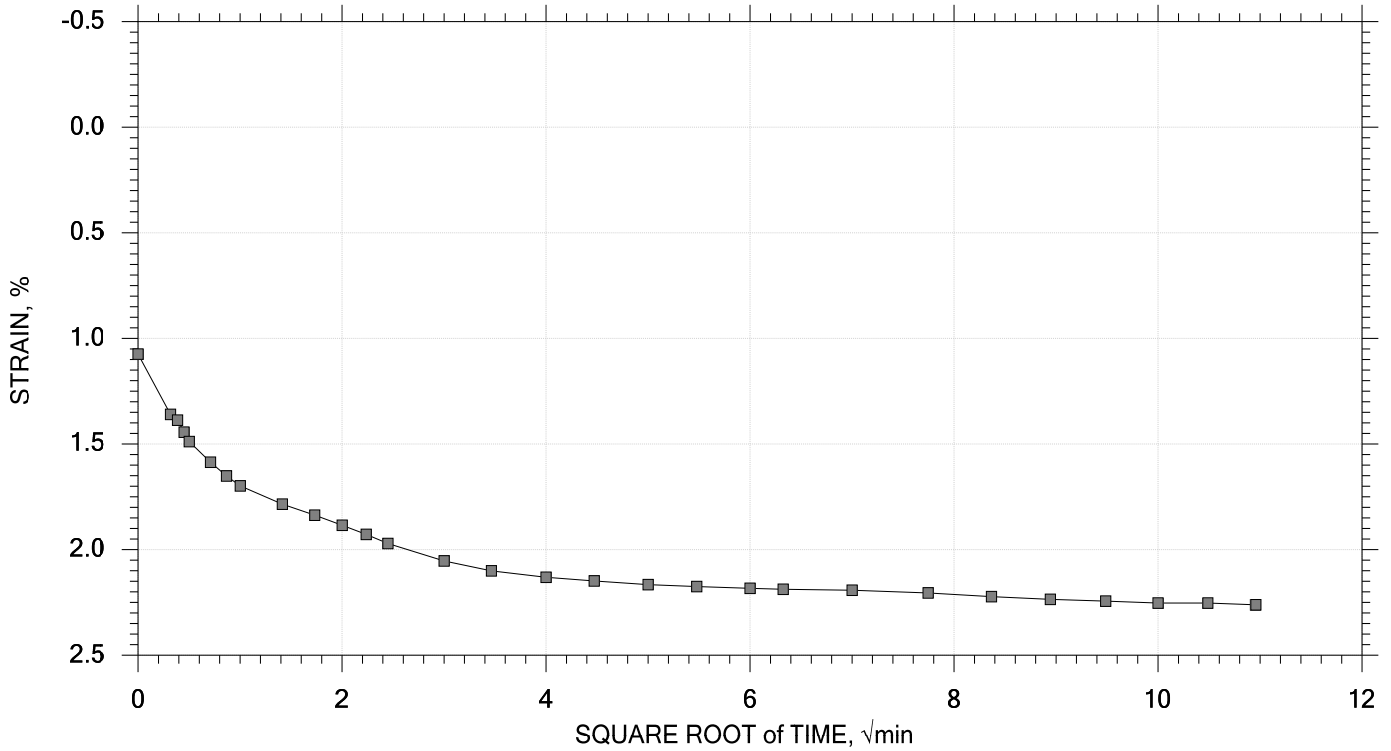
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 4 of 21

Stress: 0.5 tsf





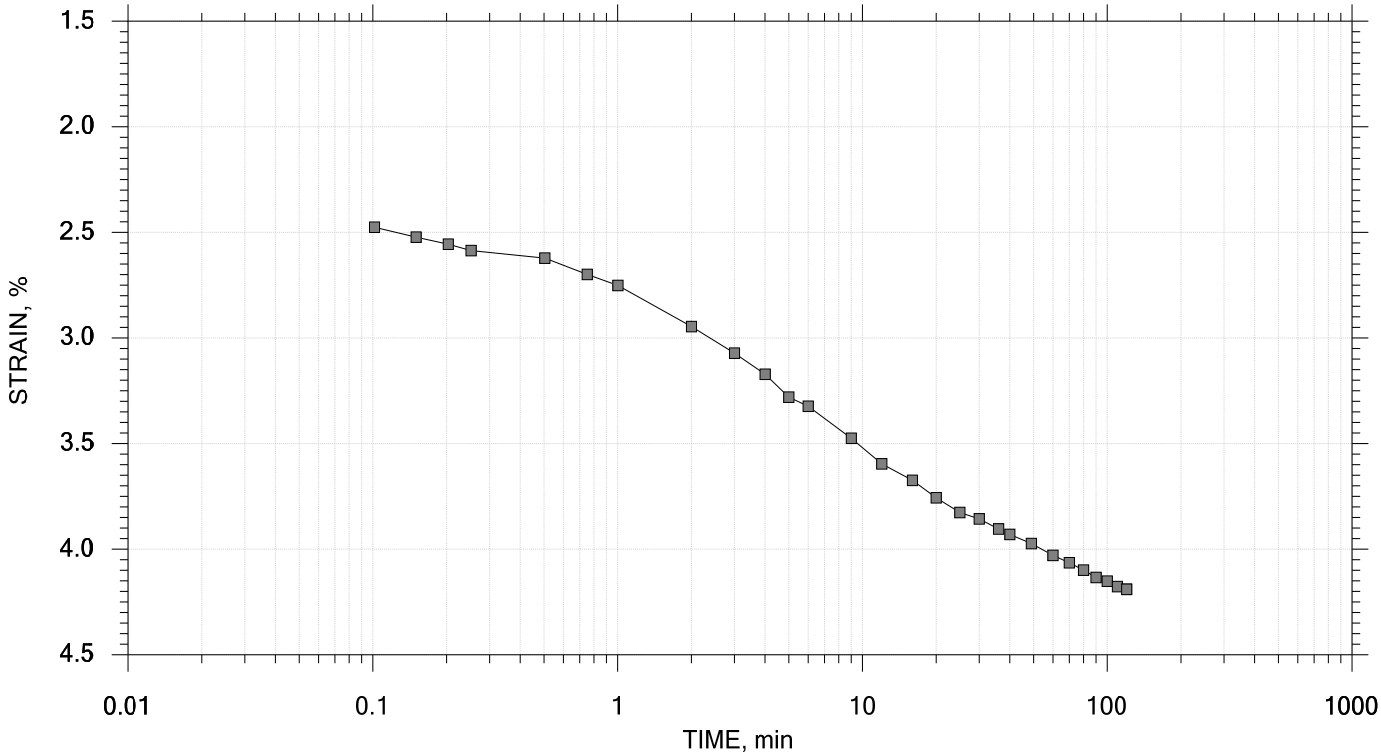
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	Boring No.: BB-MAR-202	Tested By: md	Checked By: njh
	Sample No.: U1	Test Date: 10/16/17	Test No.: IP-2
	Depth: 35-37 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System Y, Swell Pressure = 0.0666 tsf		

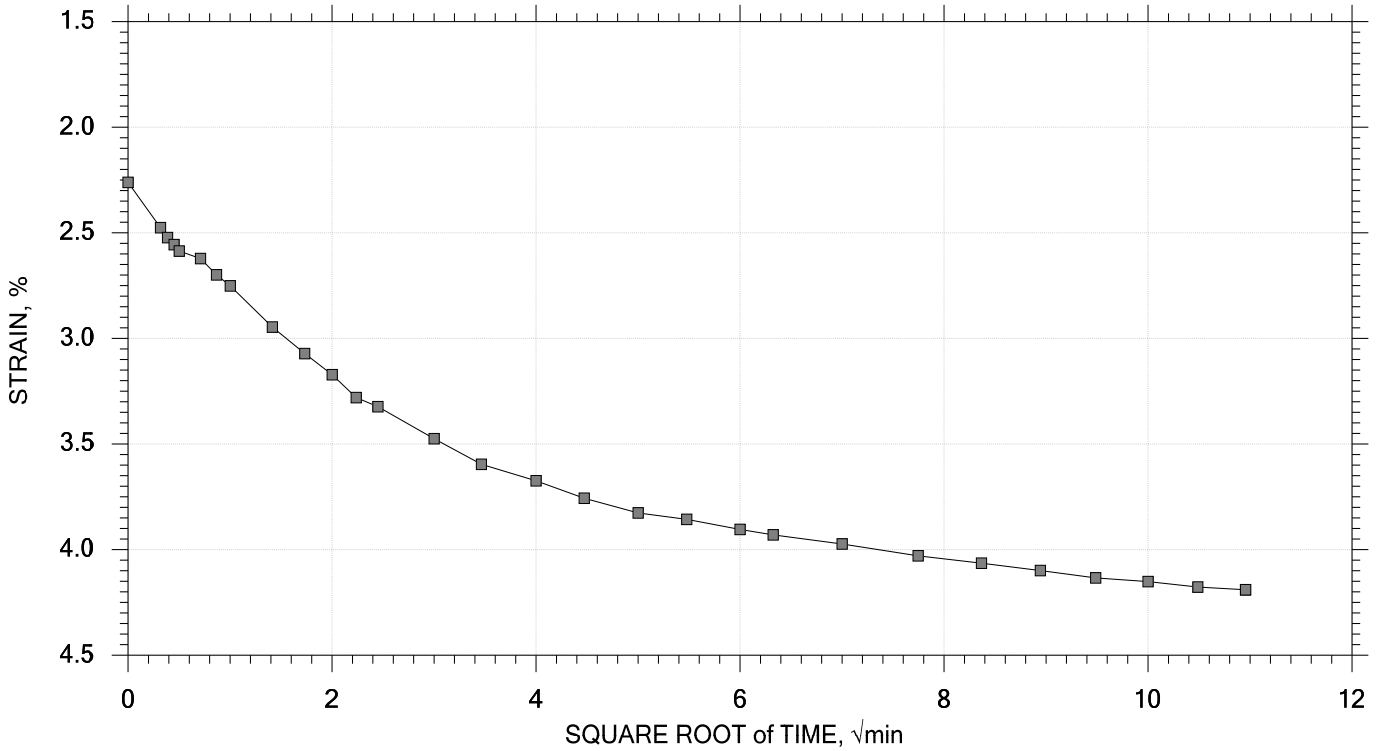
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 5 of 21

Stress: 1 tsf





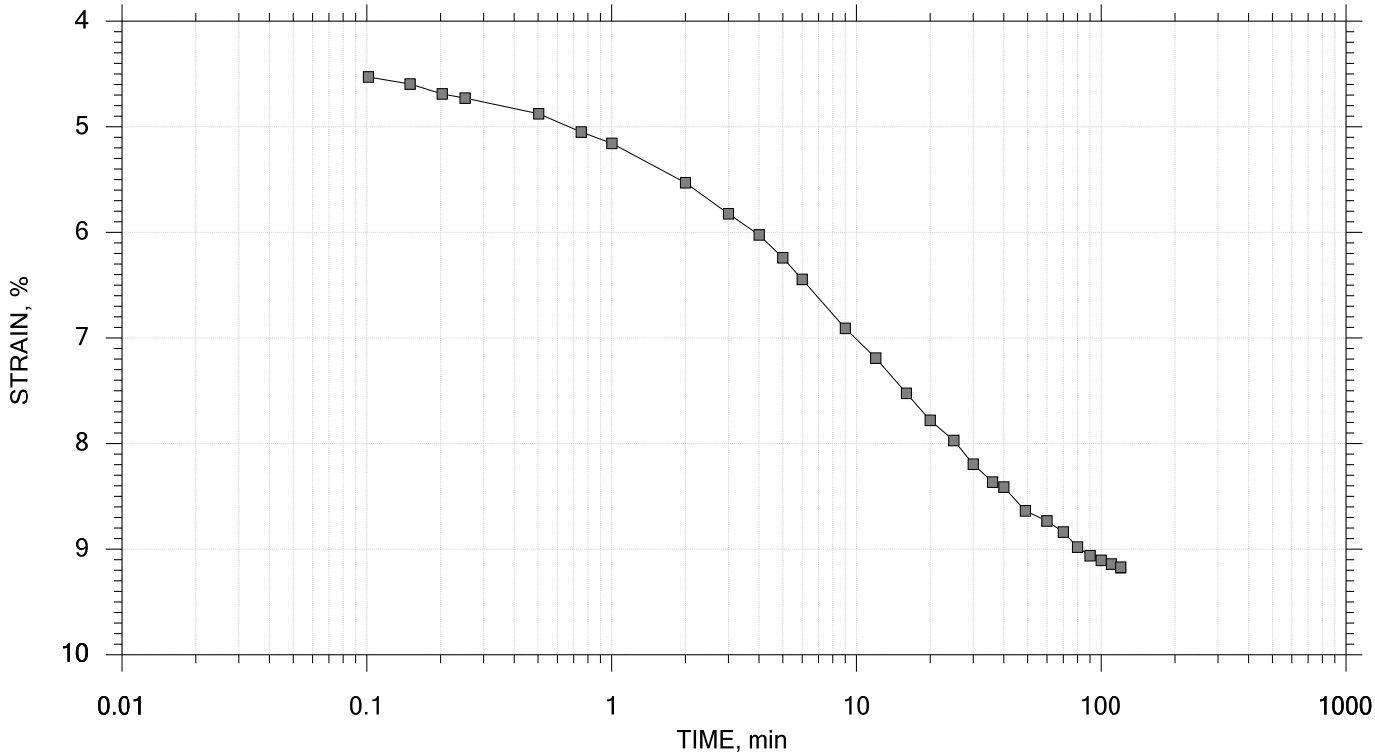
GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-202	Tested By: md	Checked By: njh
	Sample No.: U1	Test Date: 10/16/17	Test No.: IP-2
	Depth: 35-37 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System Y, Swell Pressure = 0.0666 tsf		

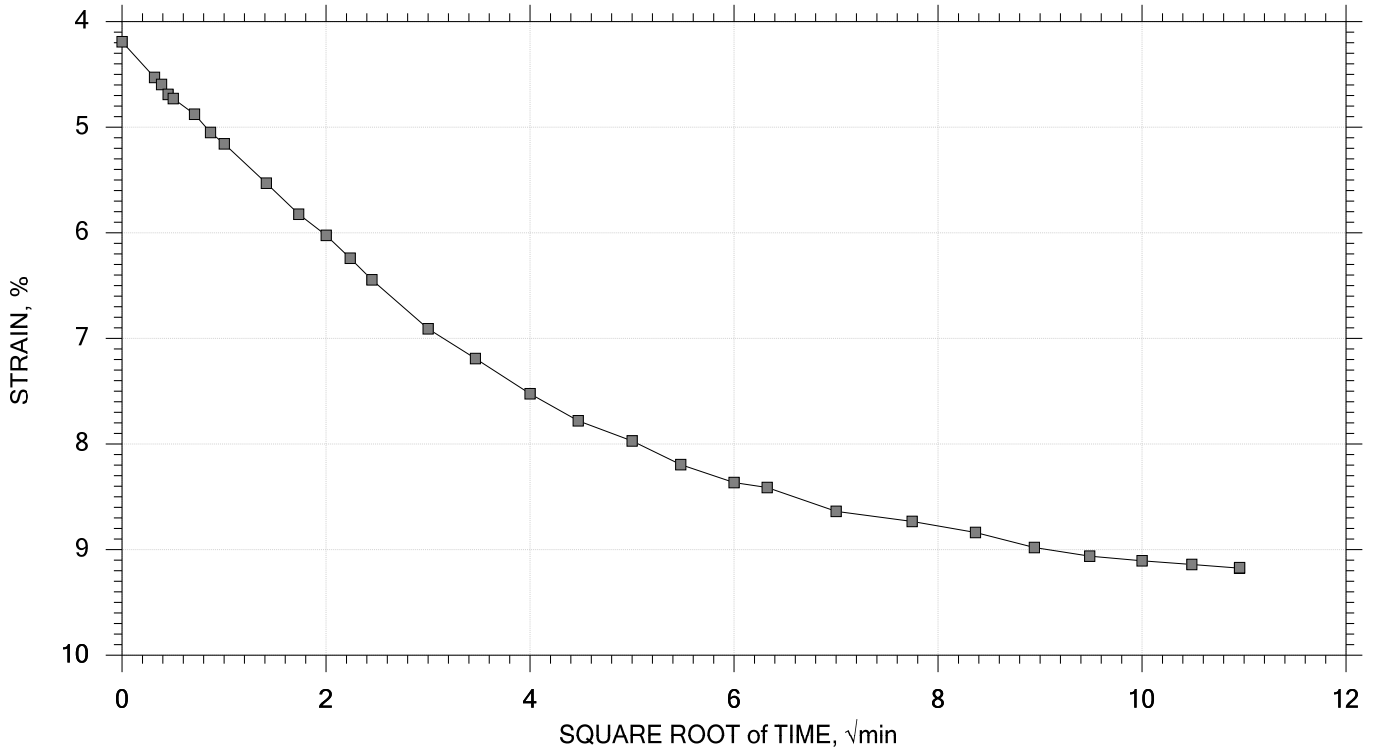
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 6 of 21

Stress: 2 tsf





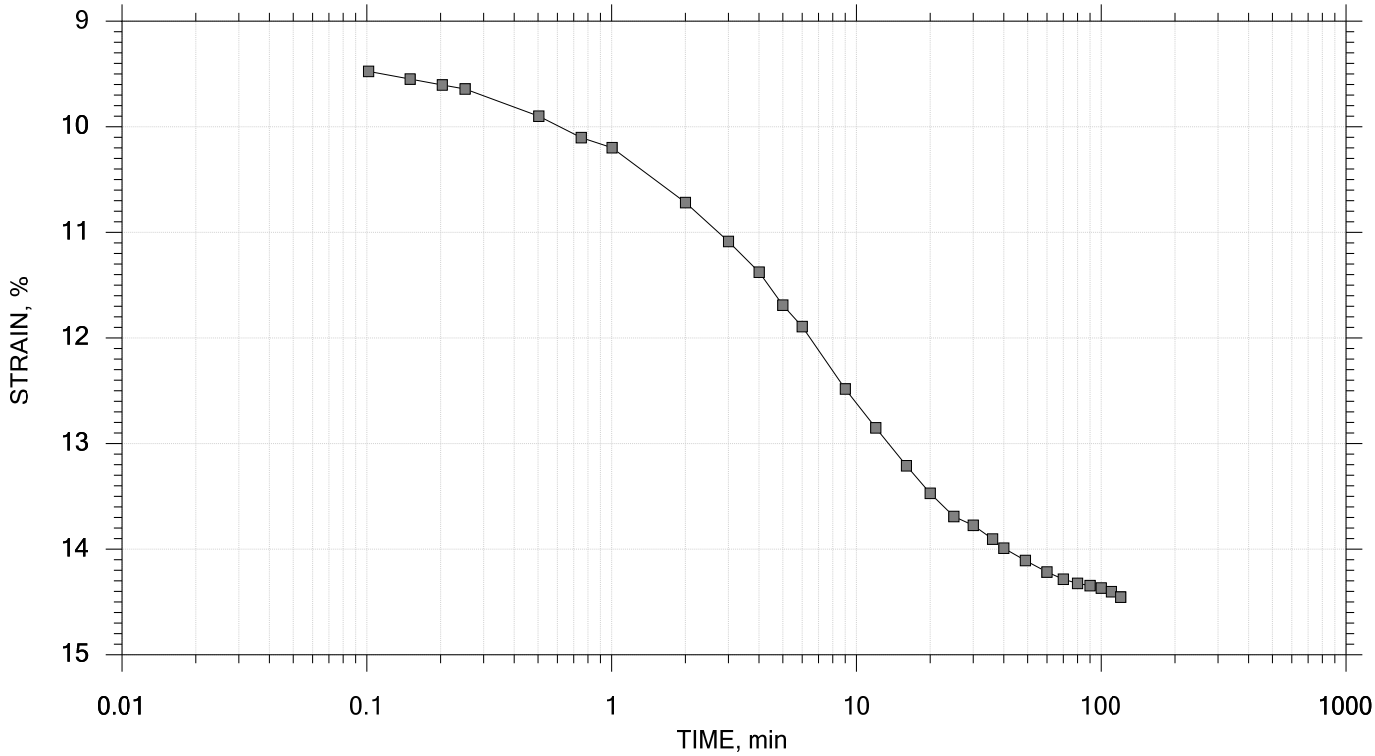
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	Boring No.: BB-MAR-202	Tested By: md	Checked By: njh
	Sample No.: U1	Test Date: 10/16/17	Test No.: IP-2
	Depth: 35-37 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System Y, Swell Pressure = 0.0666 tsf		

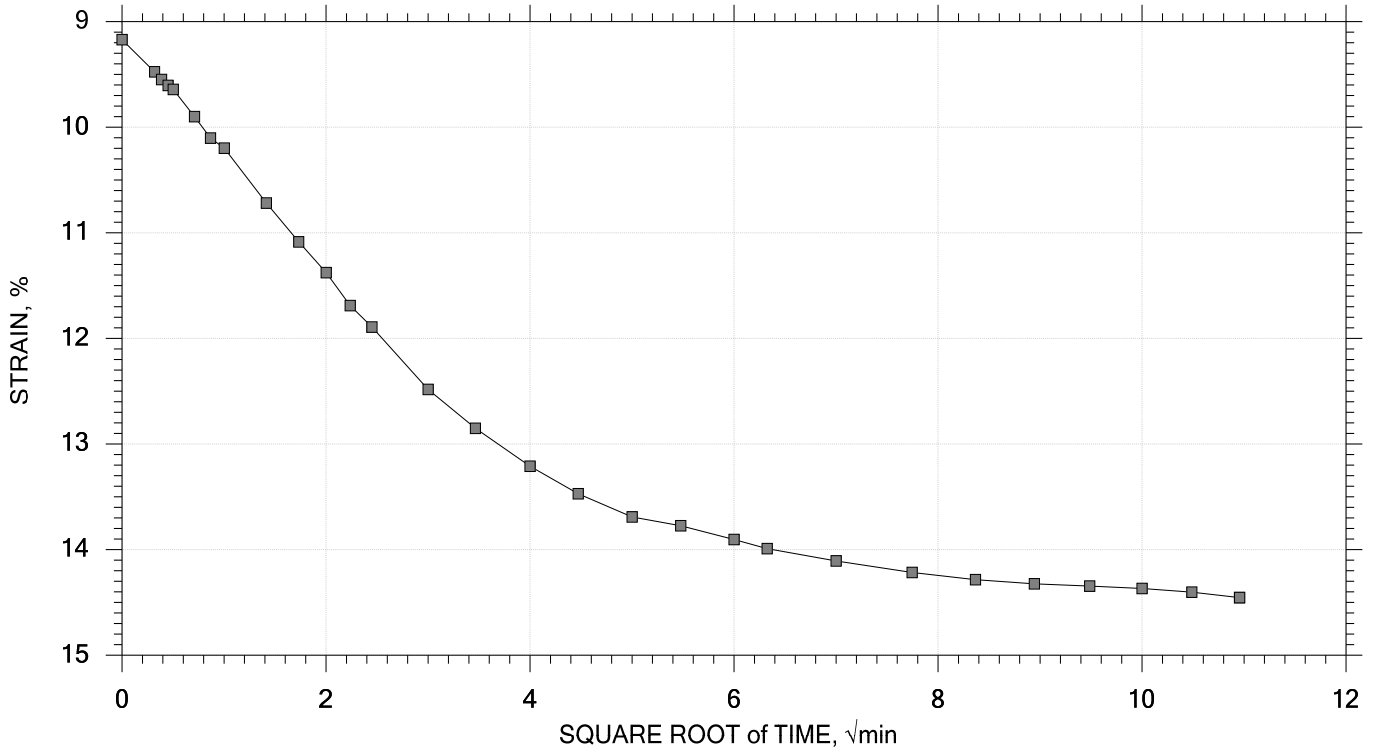
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 7 of 21

Stress: 4 tsf





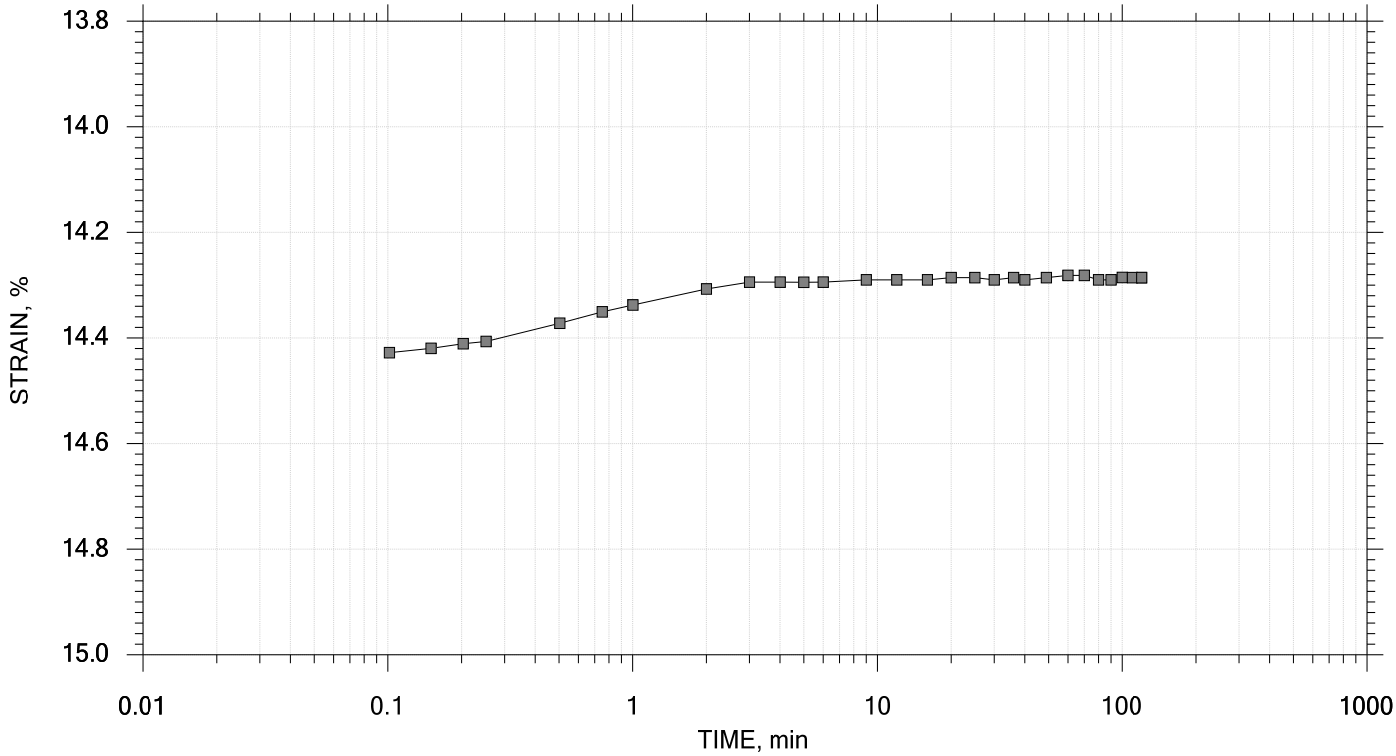
GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-202	Tested By: md	Checked By: njh
	Sample No.: U1	Test Date: 10/16/17	Test No.: IP-2
	Depth: 35-37 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System Y, Swell Pressure = 0.0666 tsf		

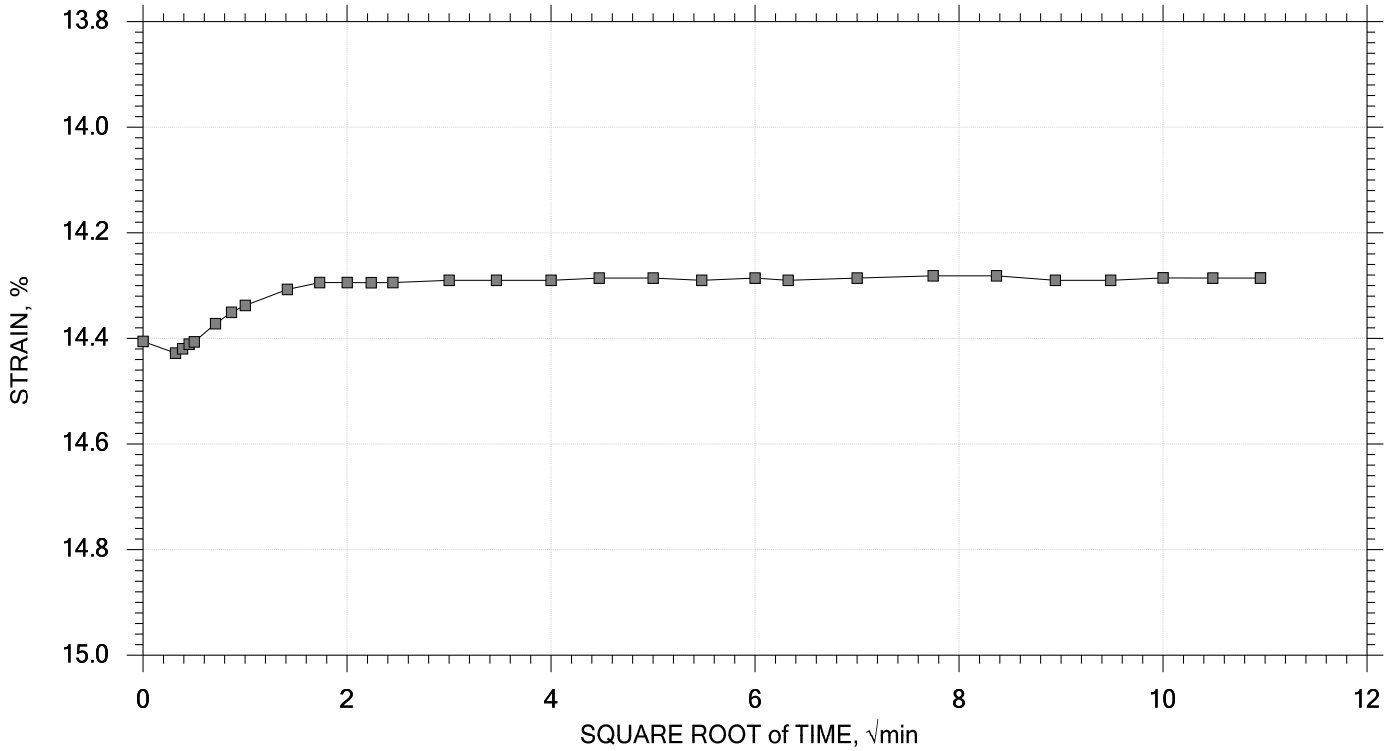
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 8 of 21

Stress: 2 tsf





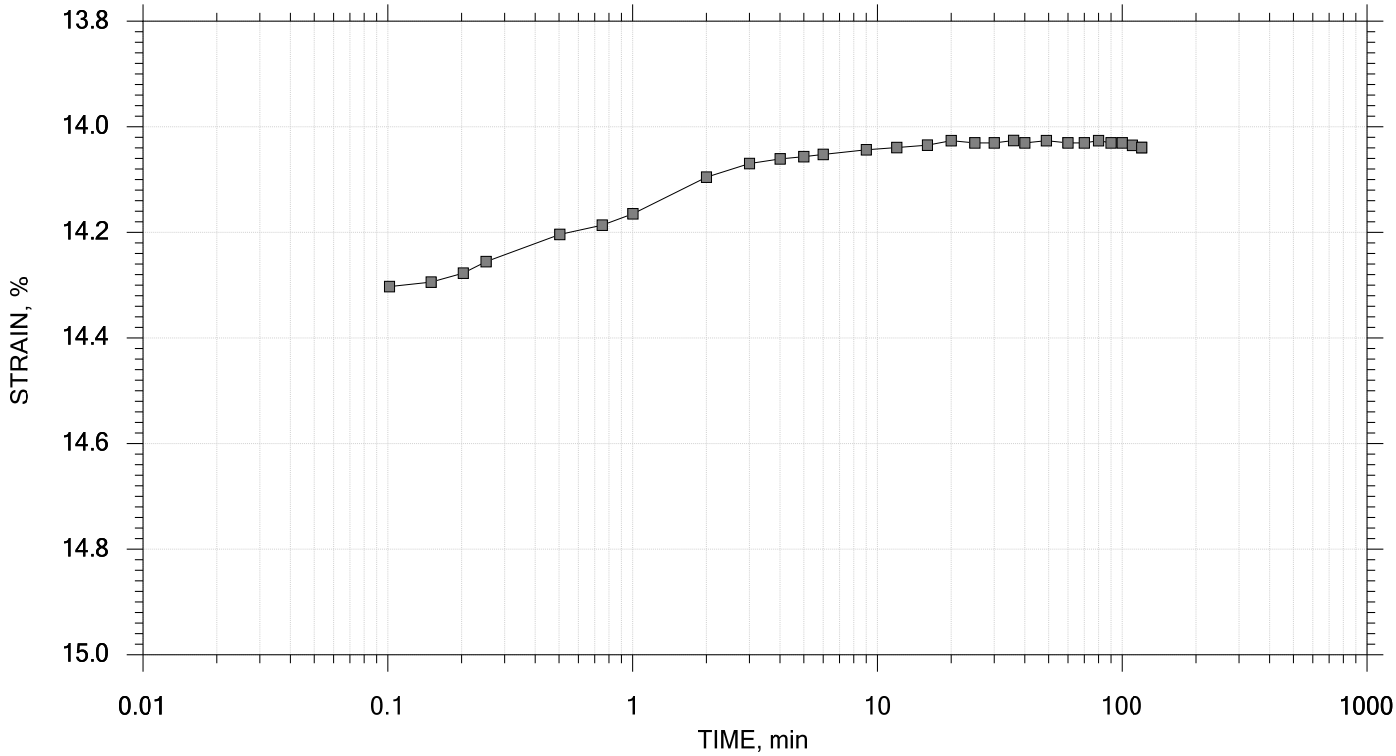
GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-202	Tested By: md	Checked By: njh
	Sample No.: U1	Test Date: 10/16/17	Test No.: IP-2
	Depth: 35-37 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System Y, Swell Pressure = 0.0666 tsf		

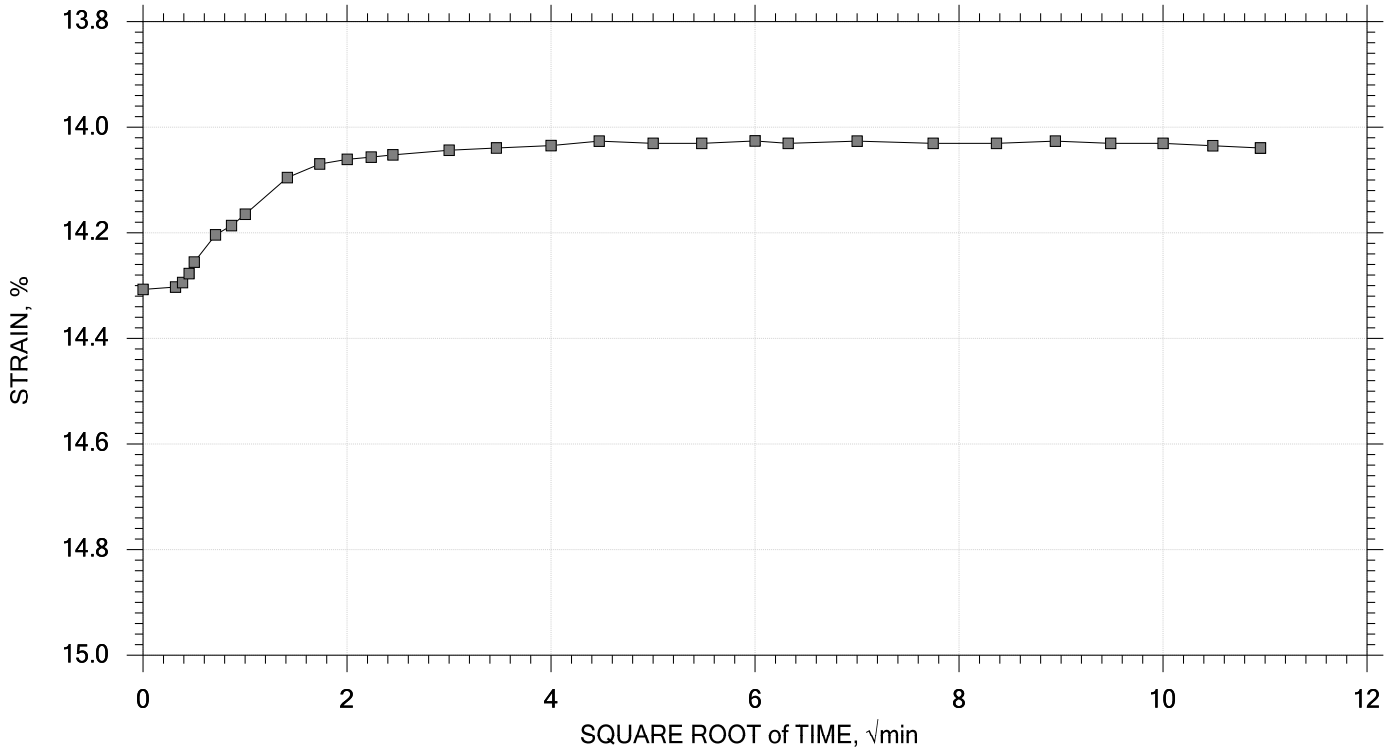
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 9 of 21

Stress: 1 tsf





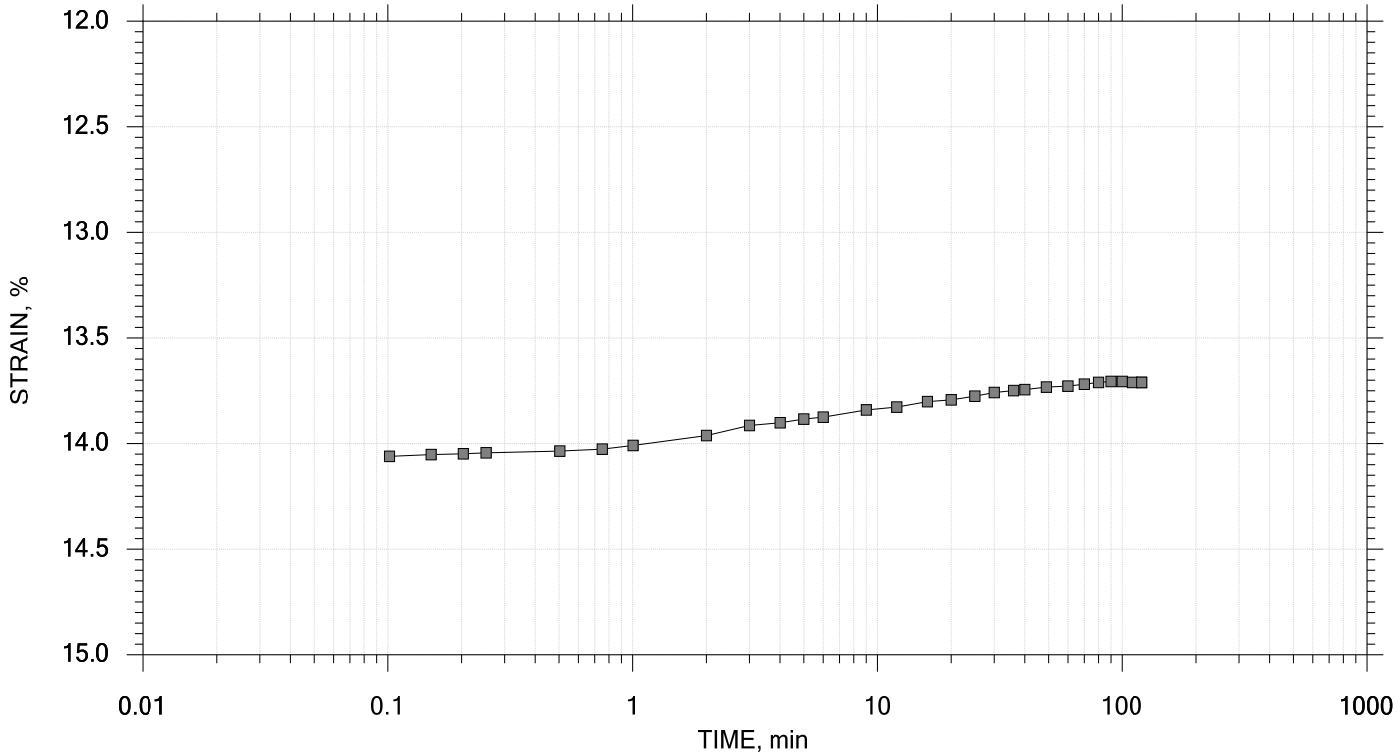
GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-202	Tested By: md	Checked By: njh
	Sample No.: U1	Test Date: 10/16/17	Test No.: IP-2
	Depth: 35-37 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System Y, Swell Pressure = 0.0666 tsf		

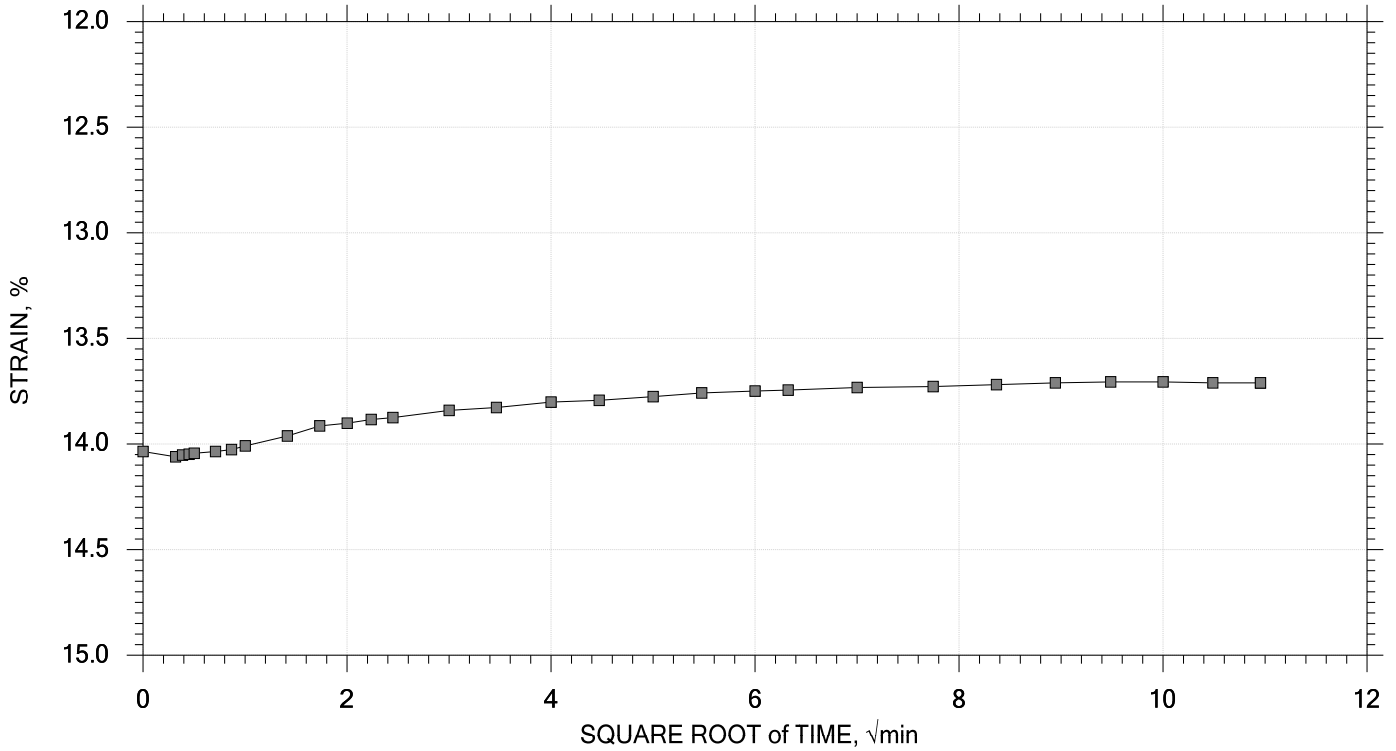
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 10 of 21

Stress: 0.5 tsf





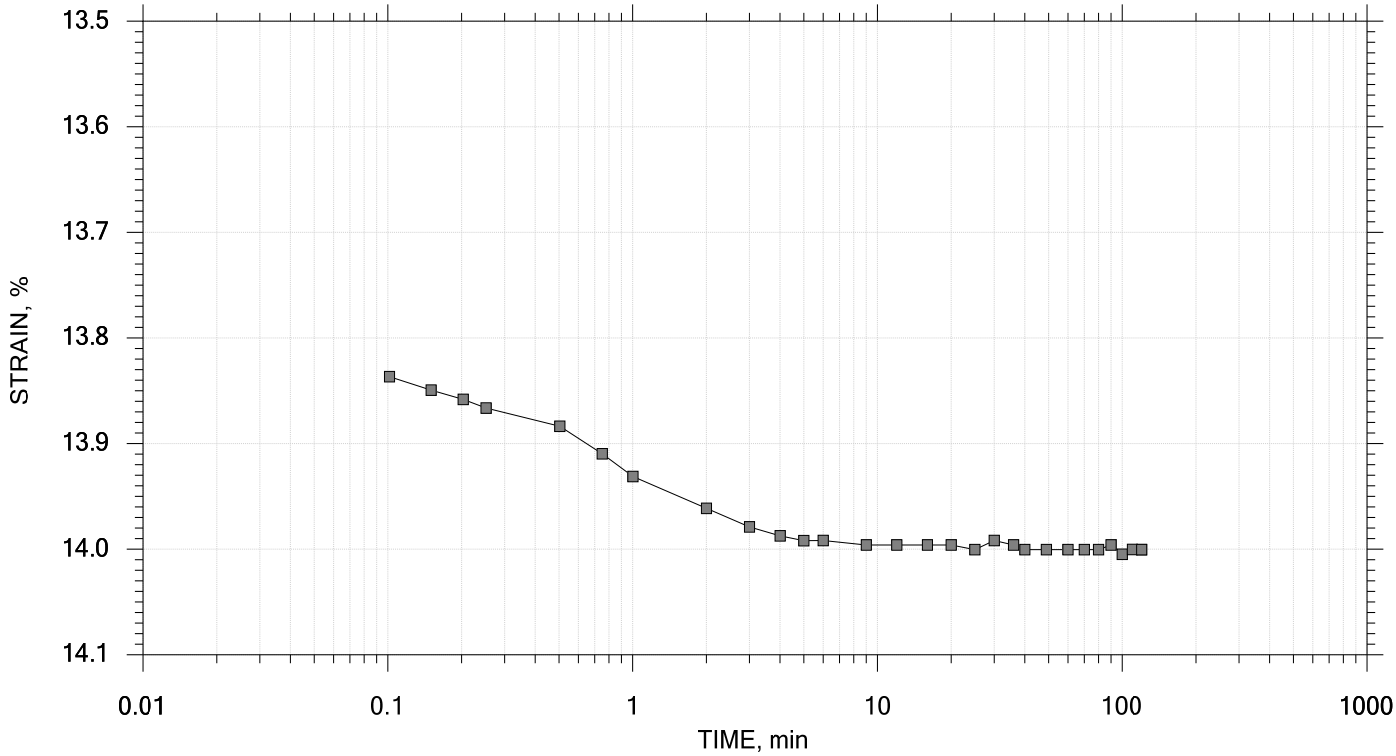
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	Boring No.: BB-MAR-202	Tested By: md	Checked By: njh
	Sample No.: U1	Test Date: 10/16/17	Test No.: IP-2
	Depth: 35-37 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System Y, Swell Pressure = 0.0666 tsf		

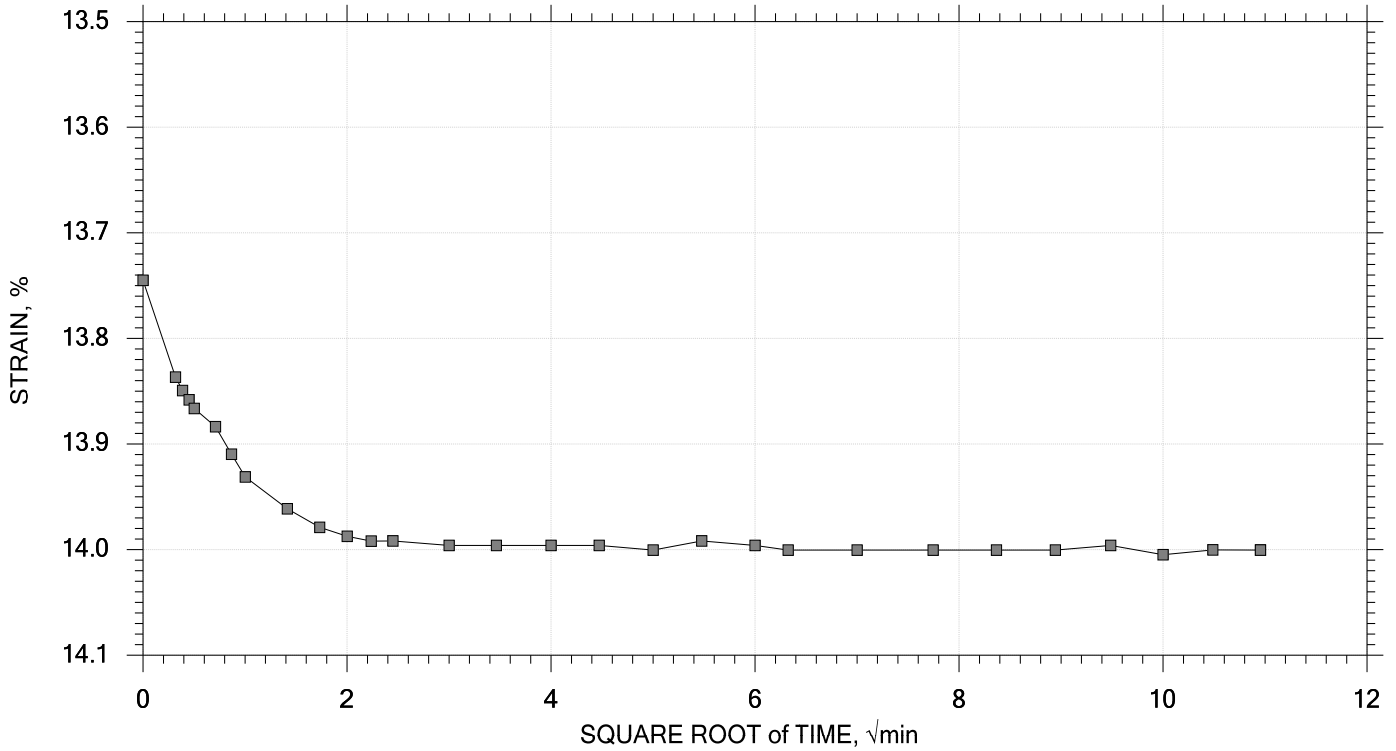
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 11 of 21

Stress: 1 tsf





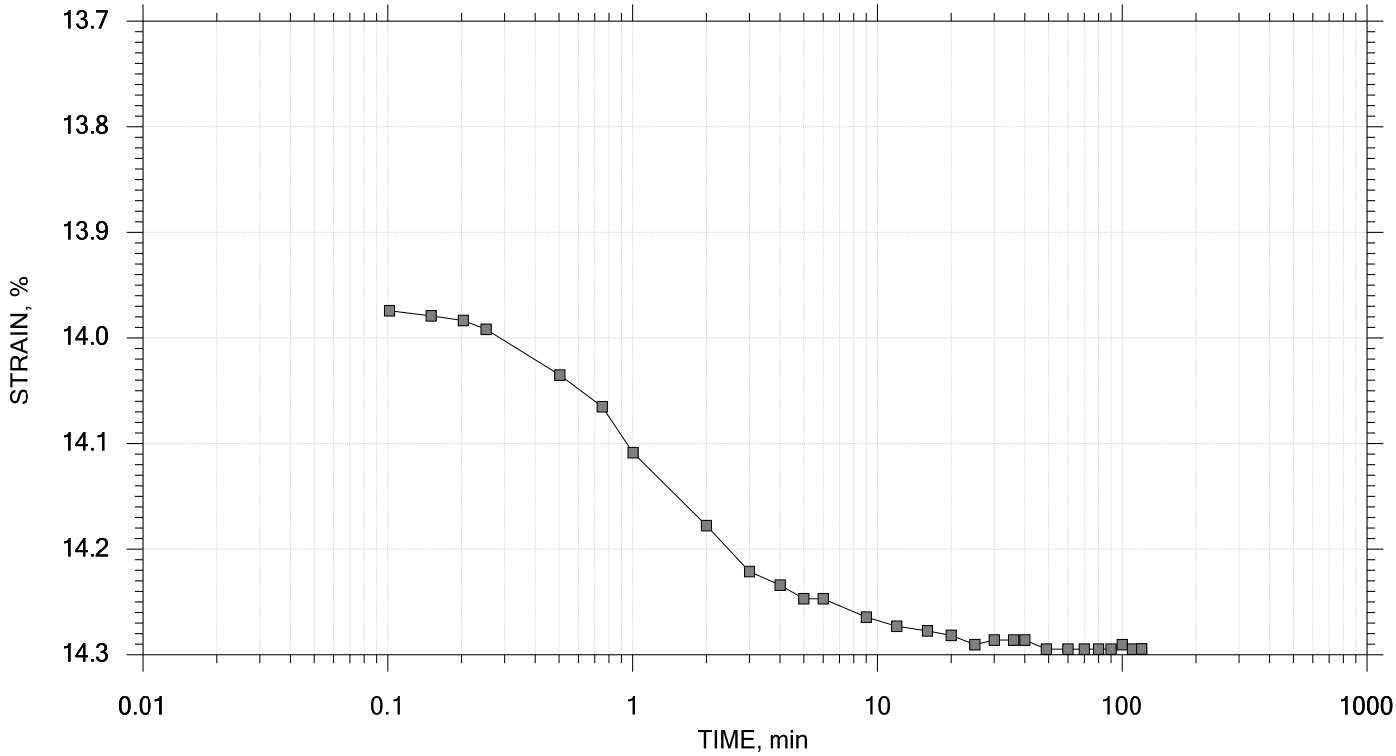
GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-202	Tested By: md	Checked By: njh
	Sample No.: U1	Test Date: 10/16/17	Test No.: IP-2
	Depth: 35-37 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System Y, Swell Pressure = 0.0666 tsf		

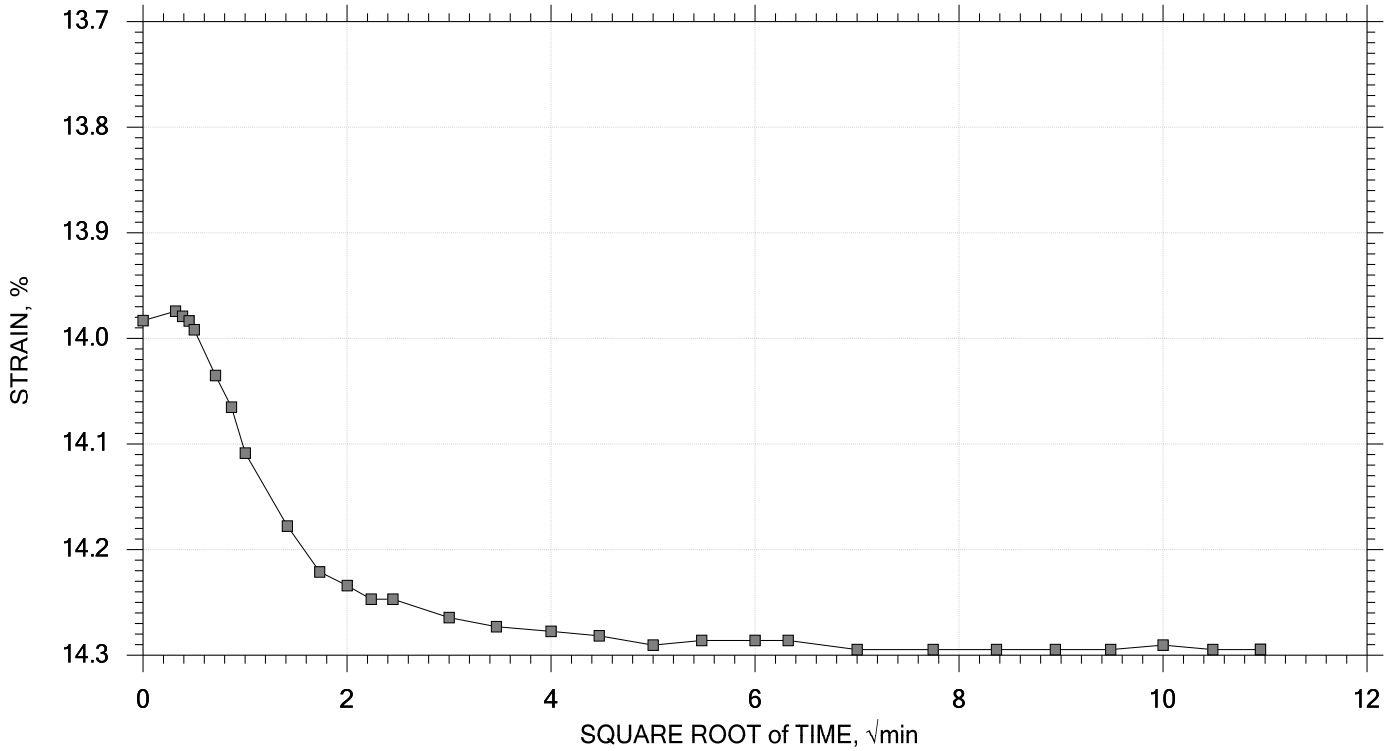
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 12 of 21

Stress: 2 tsf





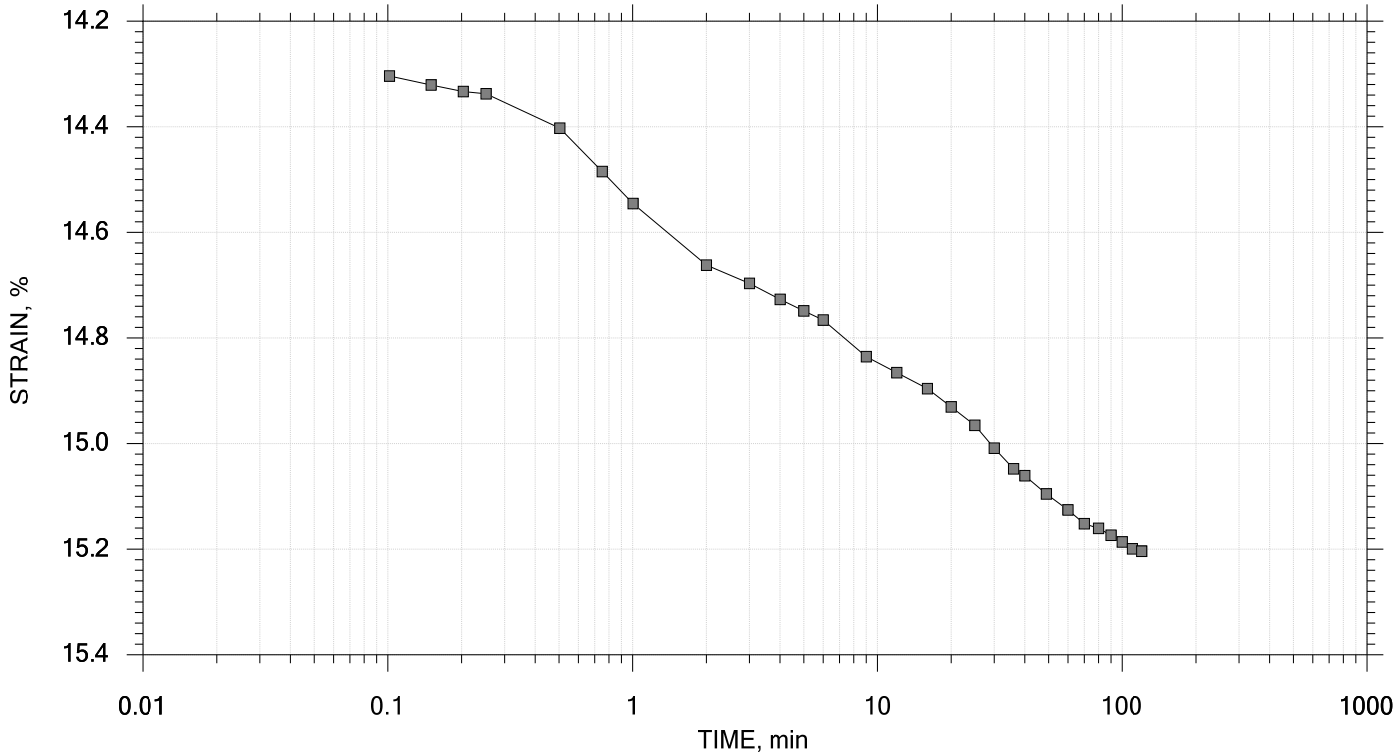
GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-202	Tested By: md	Checked By: njh
	Sample No.: U1	Test Date: 10/16/17	Test No.: IP-2
	Depth: 35-37 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System Y, Swell Pressure = 0.0666 tsf		

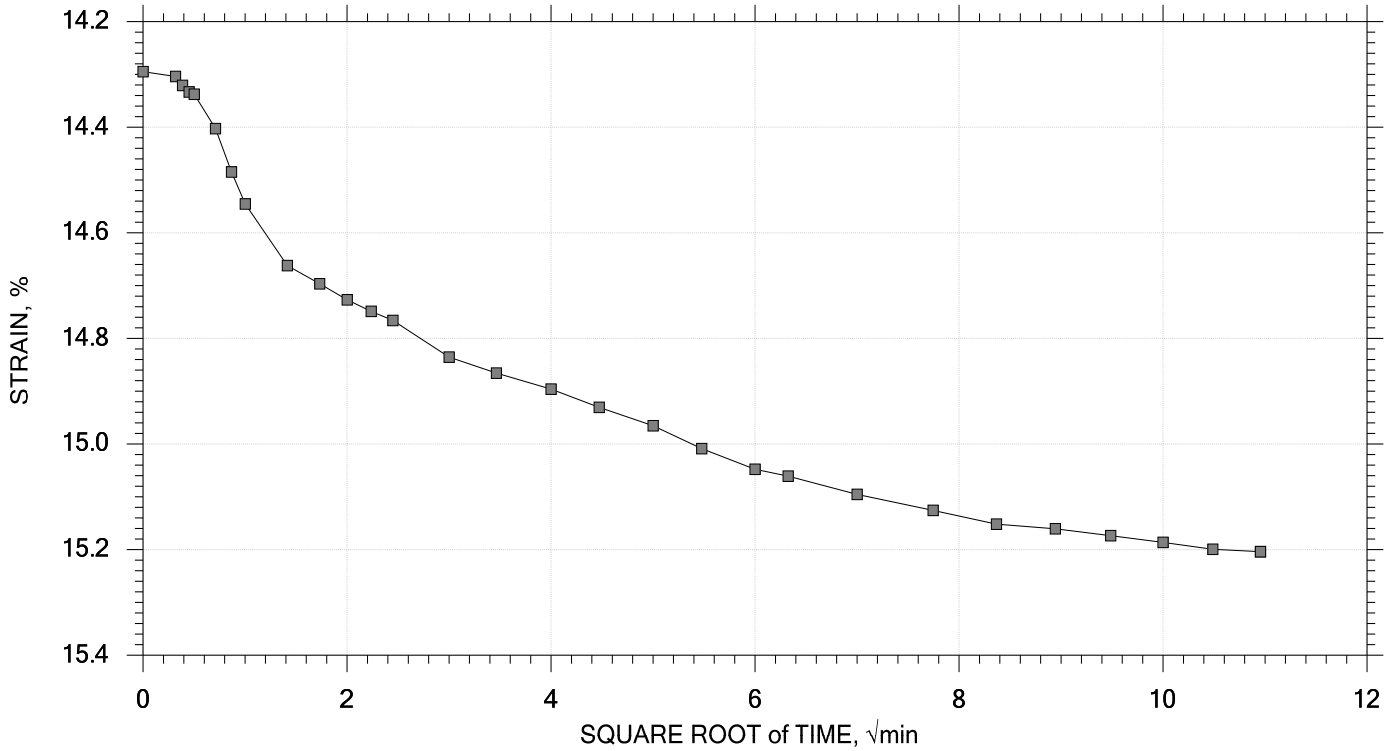
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 13 of 21

Stress: 4 tsf





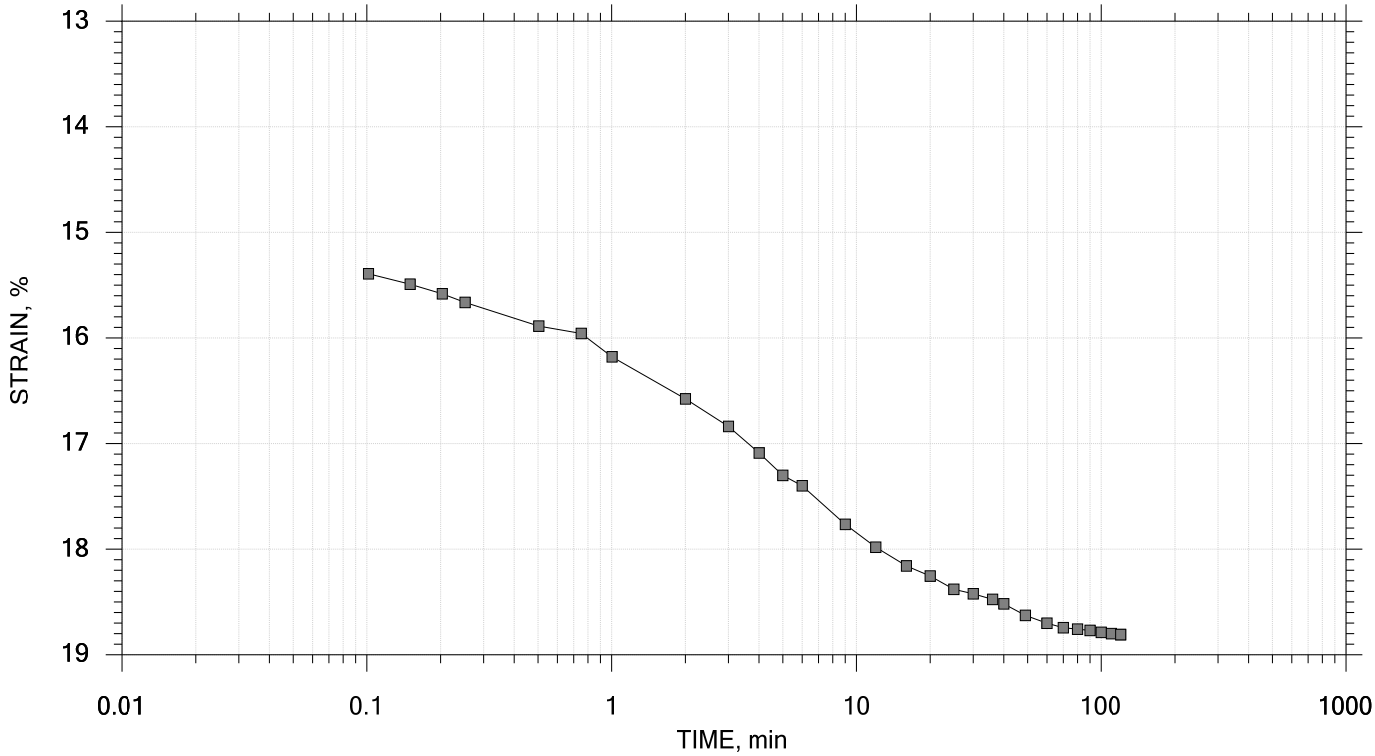
GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-202	Tested By: md	Checked By: njh
	Sample No.: U1	Test Date: 10/16/17	Test No.: IP-2
	Depth: 35-37 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System Y, Swell Pressure = 0.0666 tsf		

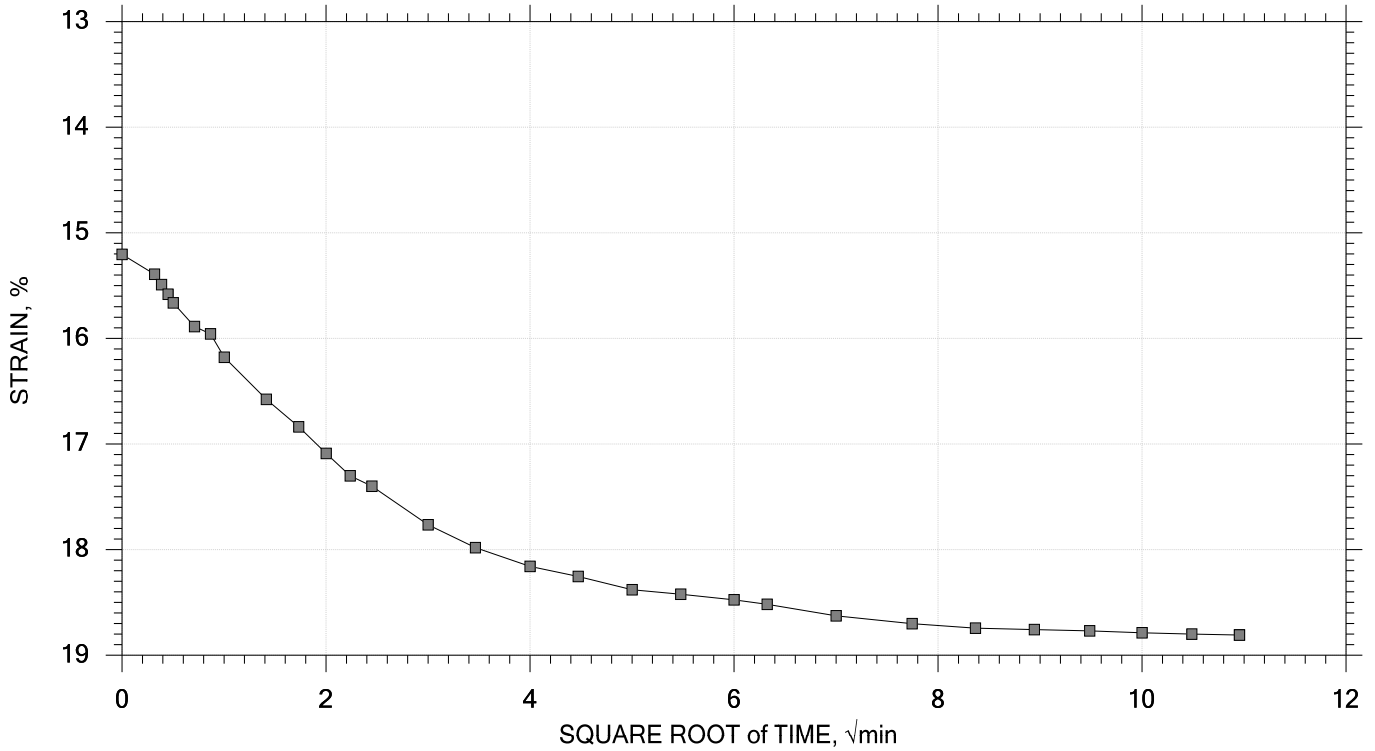
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 14 of 21

Stress: 8 tsf





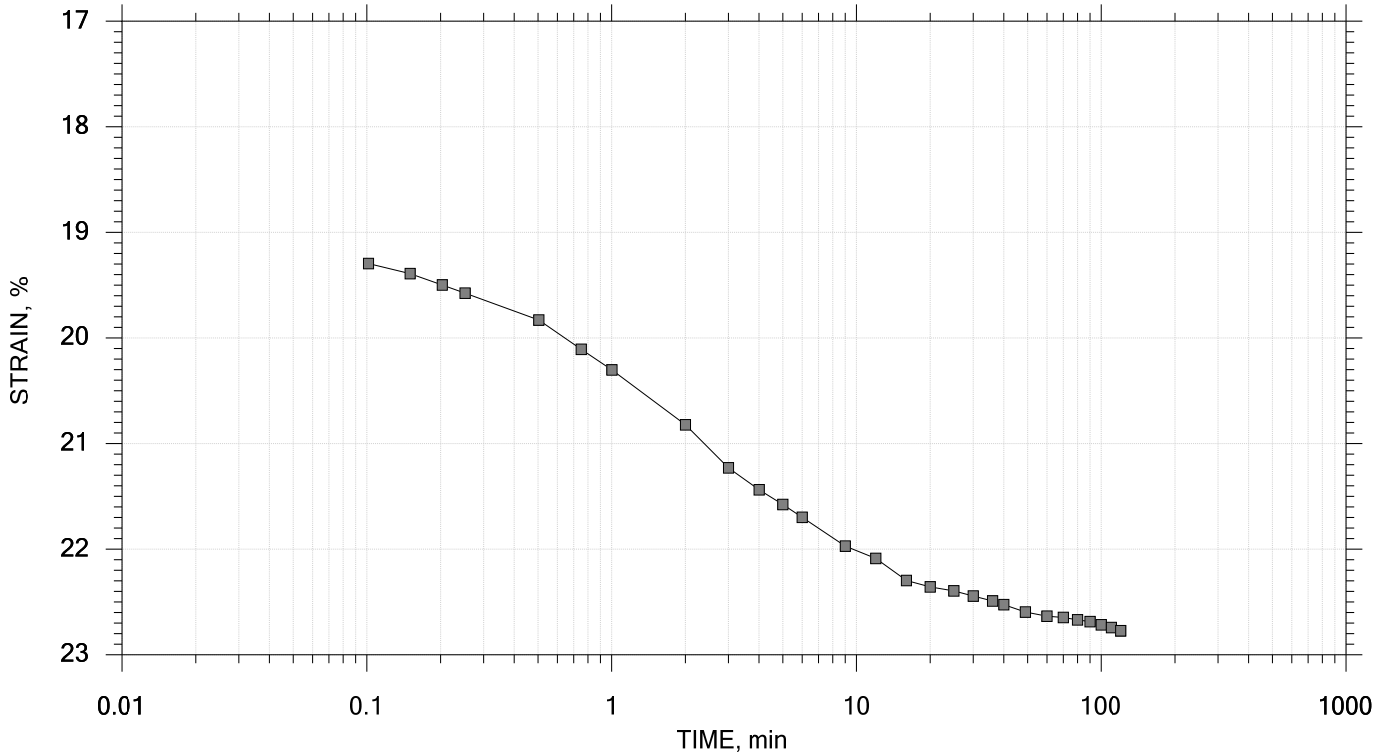
GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-202	Tested By: md	Checked By: njh
	Sample No.: U1	Test Date: 10/16/17	Test No.: IP-2
	Depth: 35-37 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System Y, Swell Pressure = 0.0666 tsf		

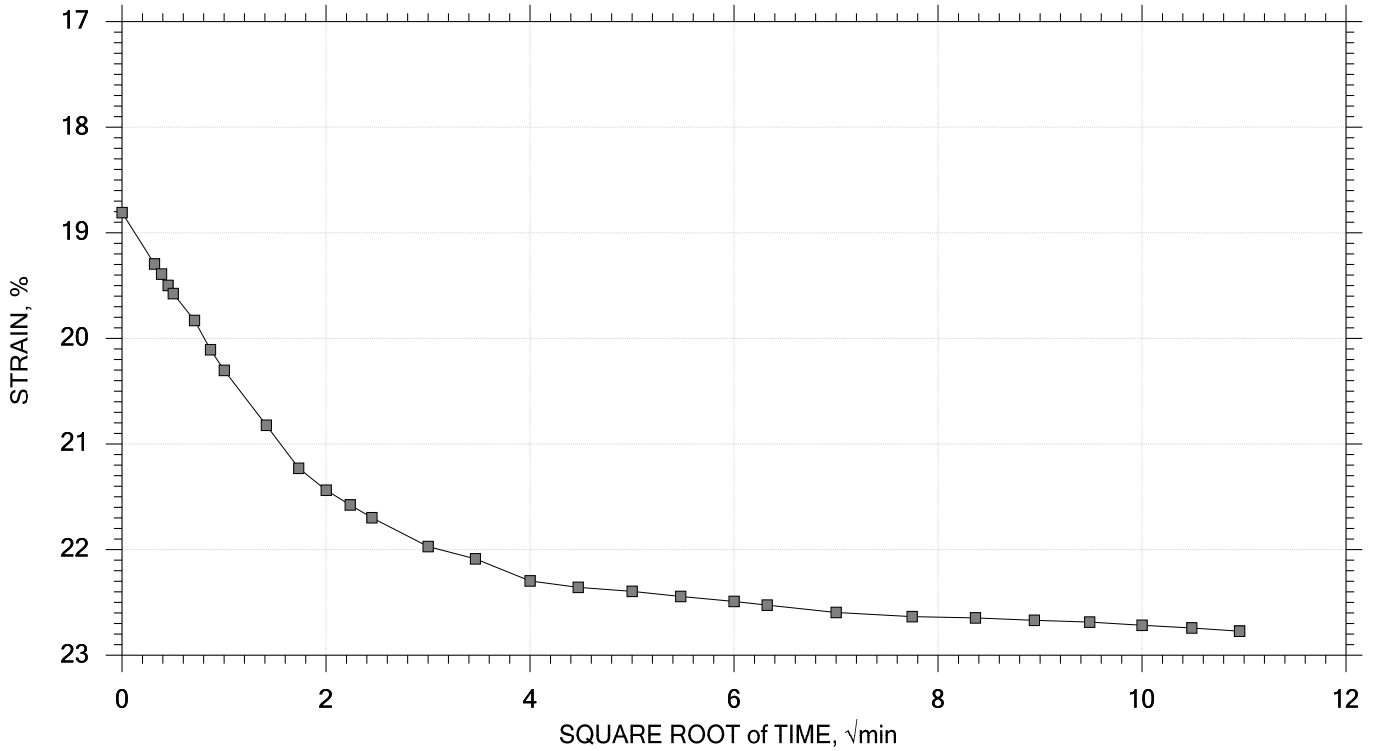
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 15 of 21

Stress: 16 tsf





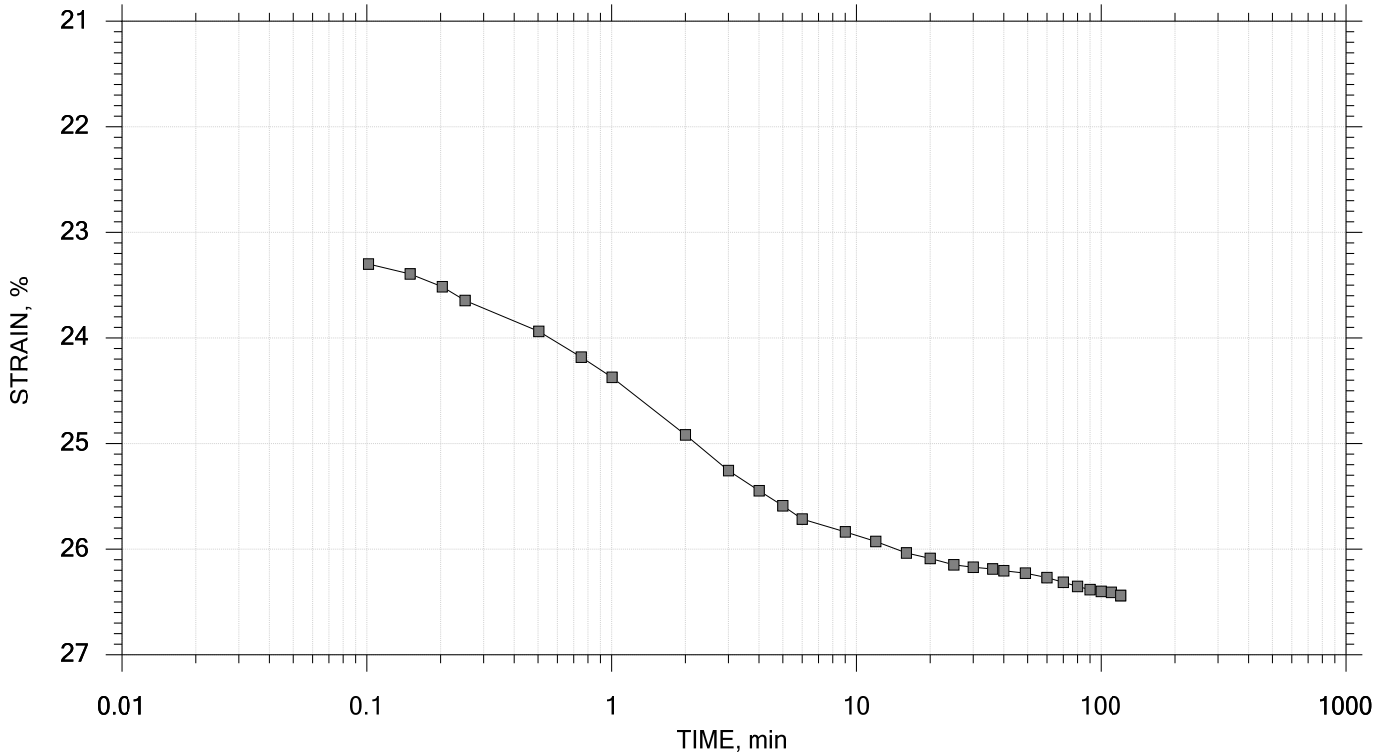
GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-202	Tested By: md	Checked By: njh
	Sample No.: U1	Test Date: 10/16/17	Test No.: IP-2
	Depth: 35-37 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System Y, Swell Pressure = 0.0666 tsf		

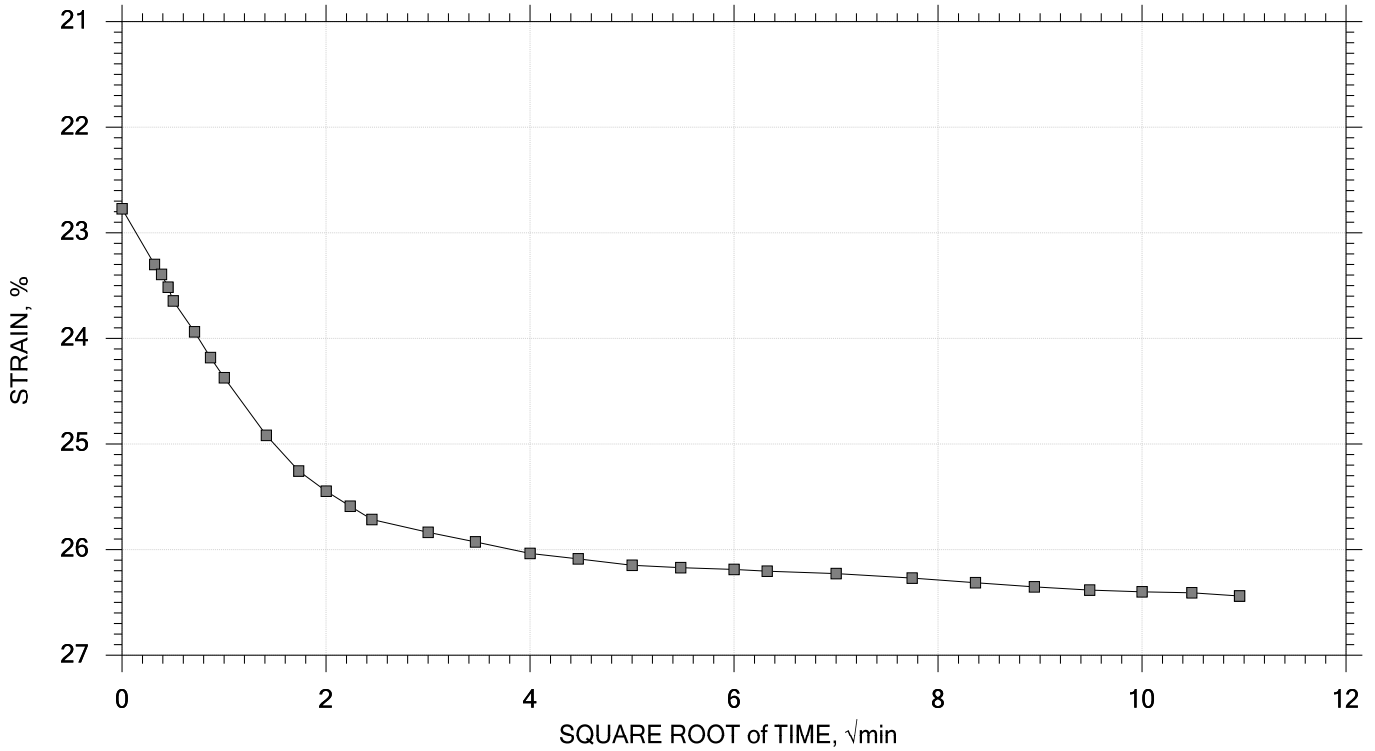
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 16 of 21

Stress: 32 tsf





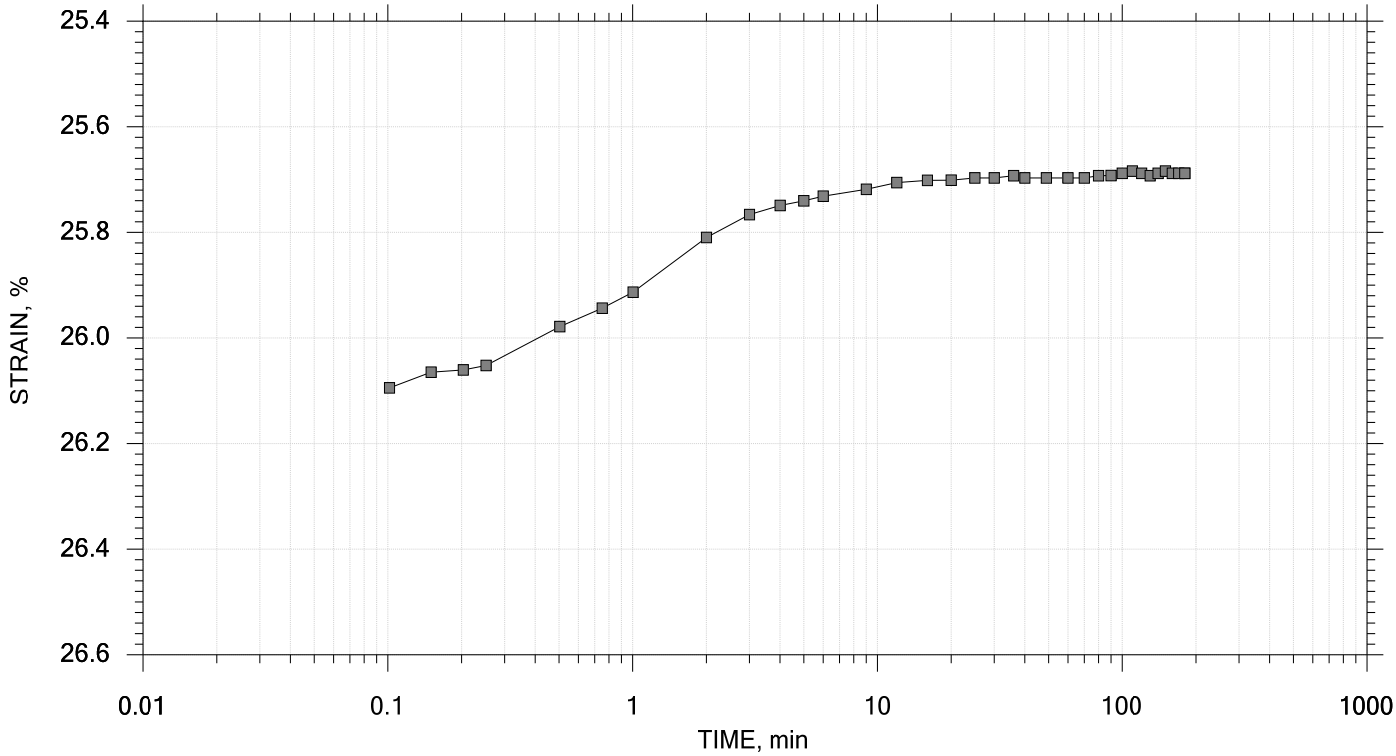
GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-202	Tested By: md	Checked By: njh
	Sample No.: U1	Test Date: 10/16/17	Test No.: IP-2
	Depth: 35-37 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System Y, Swell Pressure = 0.0666 tsf		

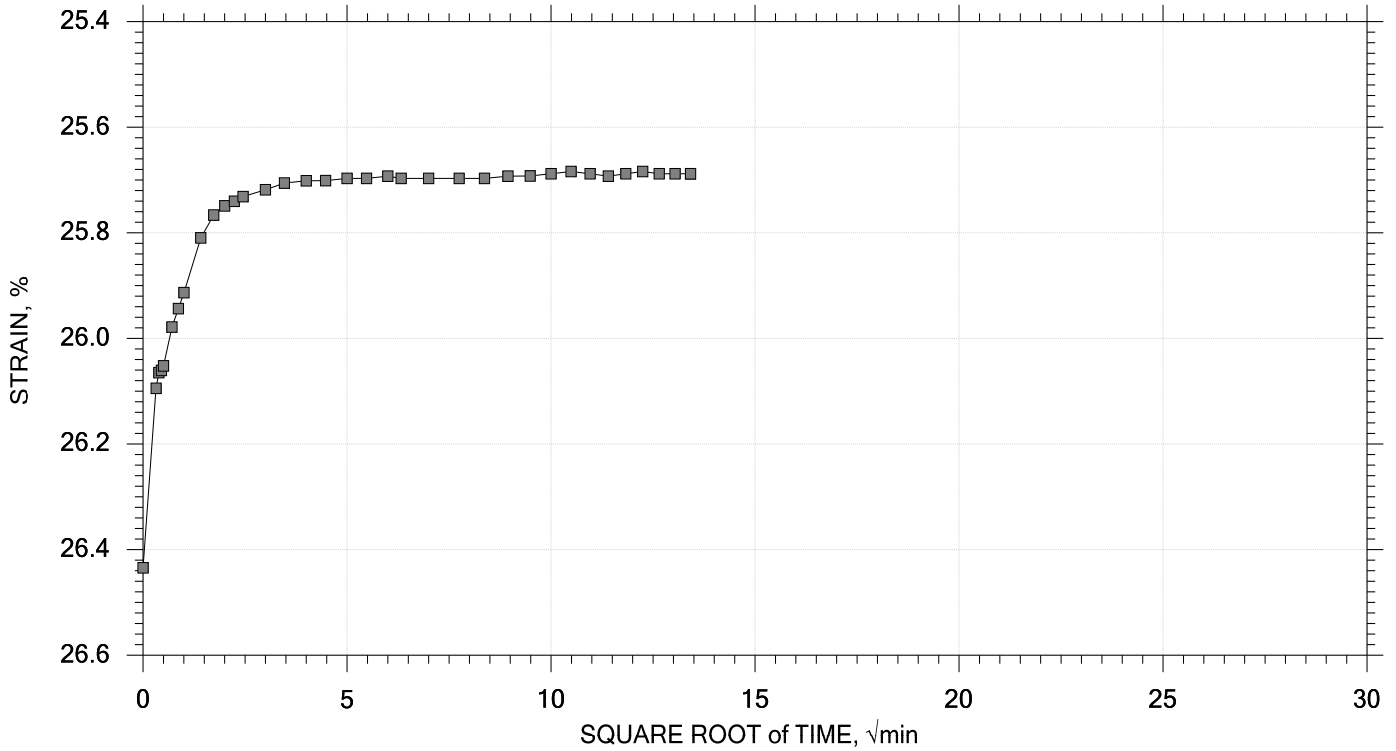
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 17 of 21

Stress: 8 tsf





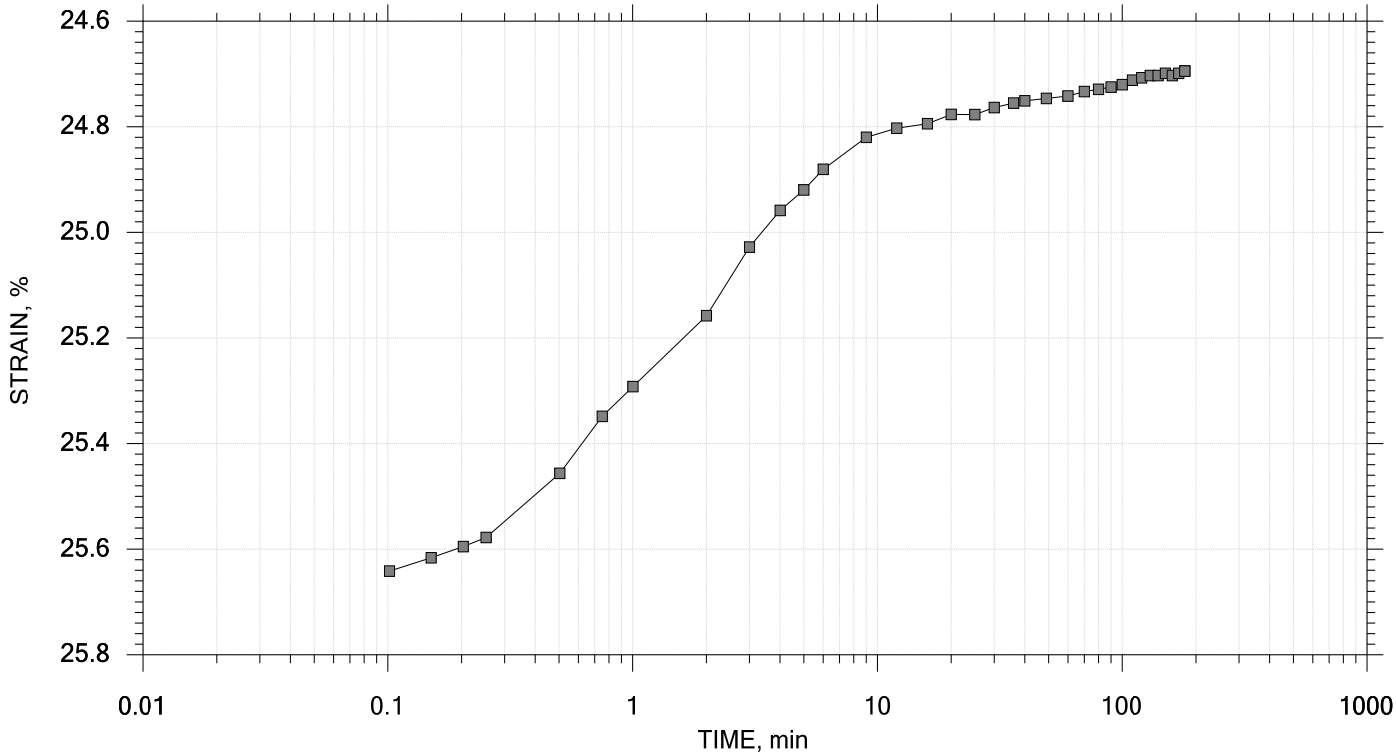
GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-202	Tested By: md	Checked By: njh
	Sample No.: U1	Test Date: 10/16/17	Test No.: IP-2
	Depth: 35-37 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System Y, Swell Pressure = 0.0666 tsf		

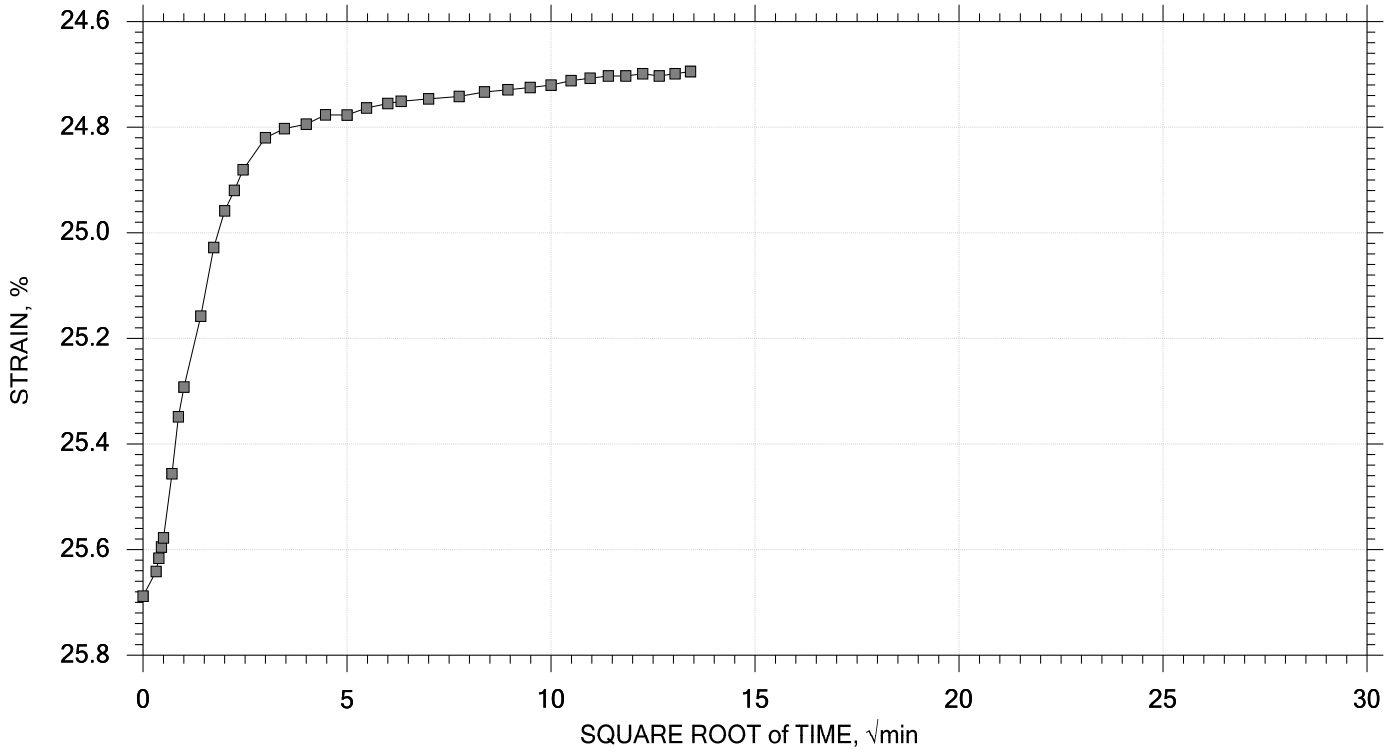
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 18 of 21

Stress: 2 tsf





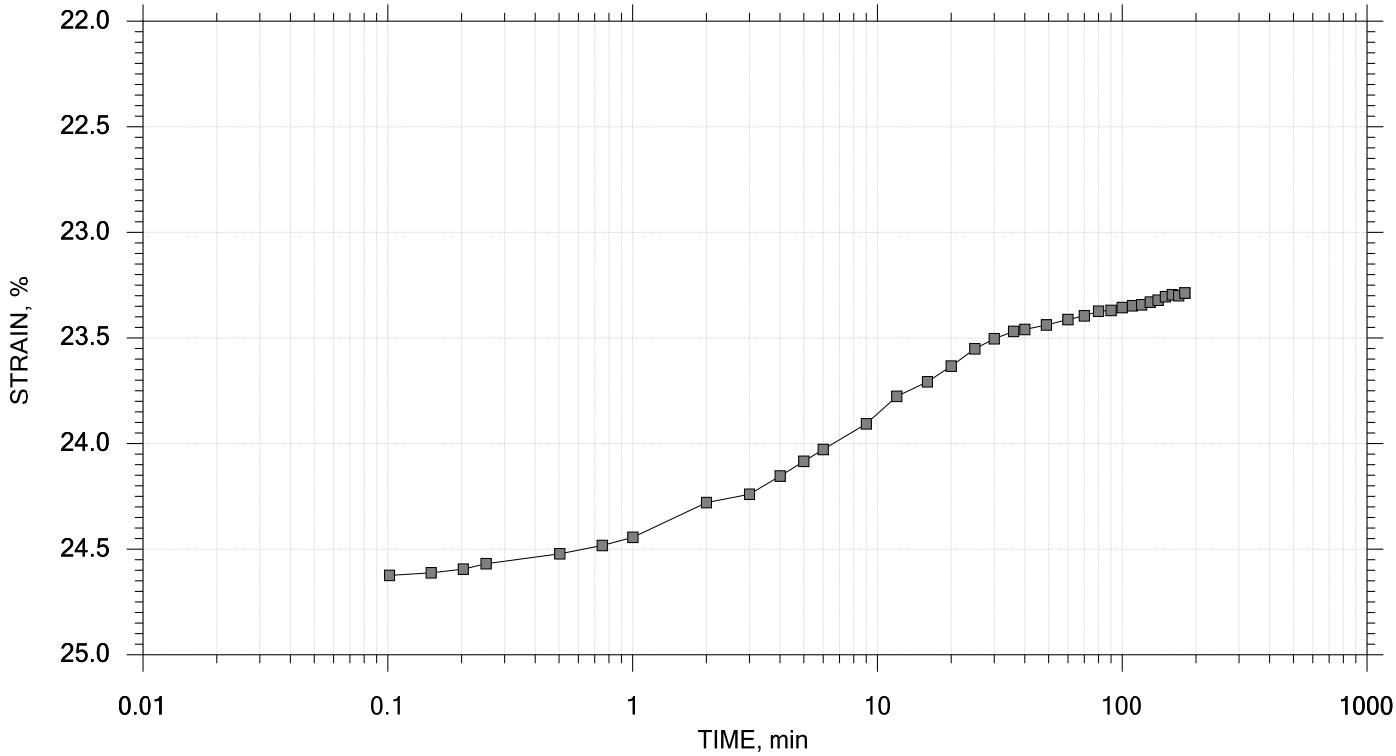
GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-202	Tested By: md	Checked By: njh
	Sample No.: U1	Test Date: 10/16/17	Test No.: IP-2
	Depth: 35-37 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System Y, Swell Pressure = 0.0666 tsf		

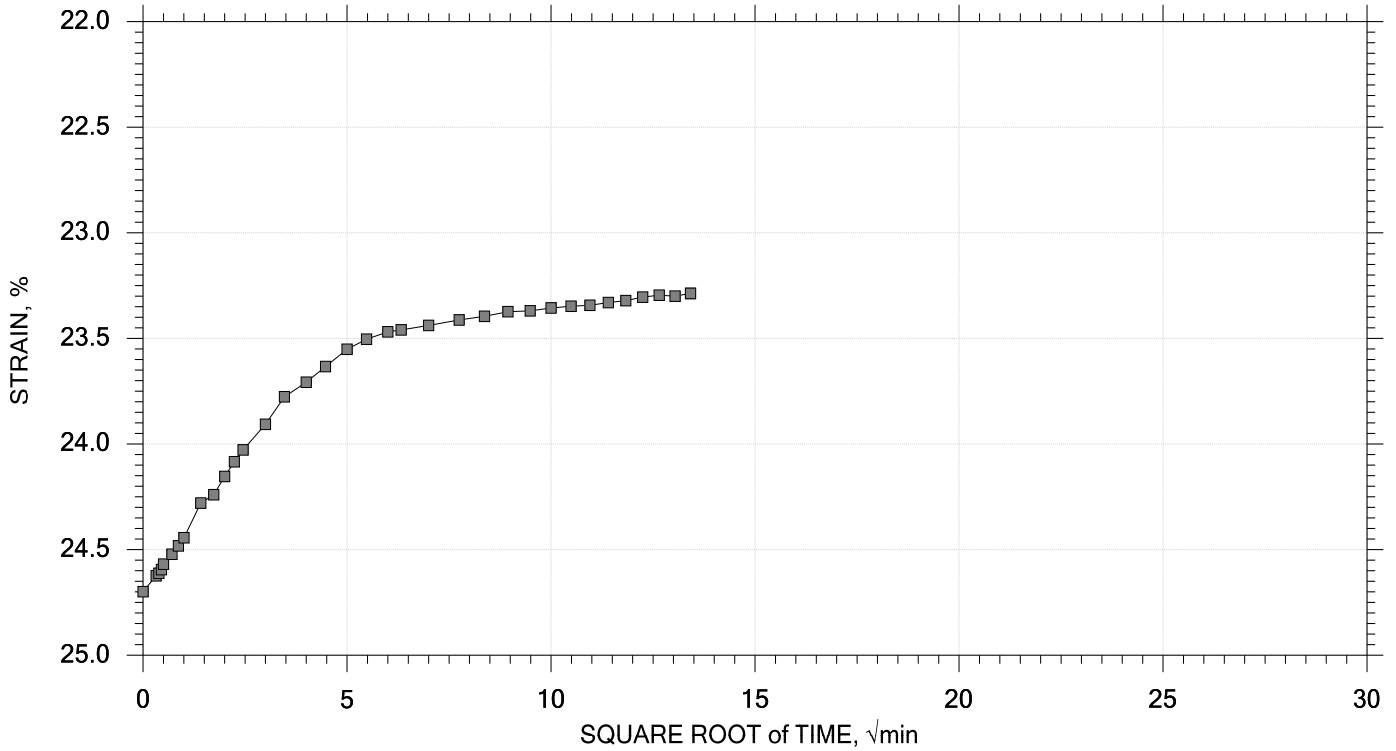
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 19 of 21

Stress: 0.5 tsf





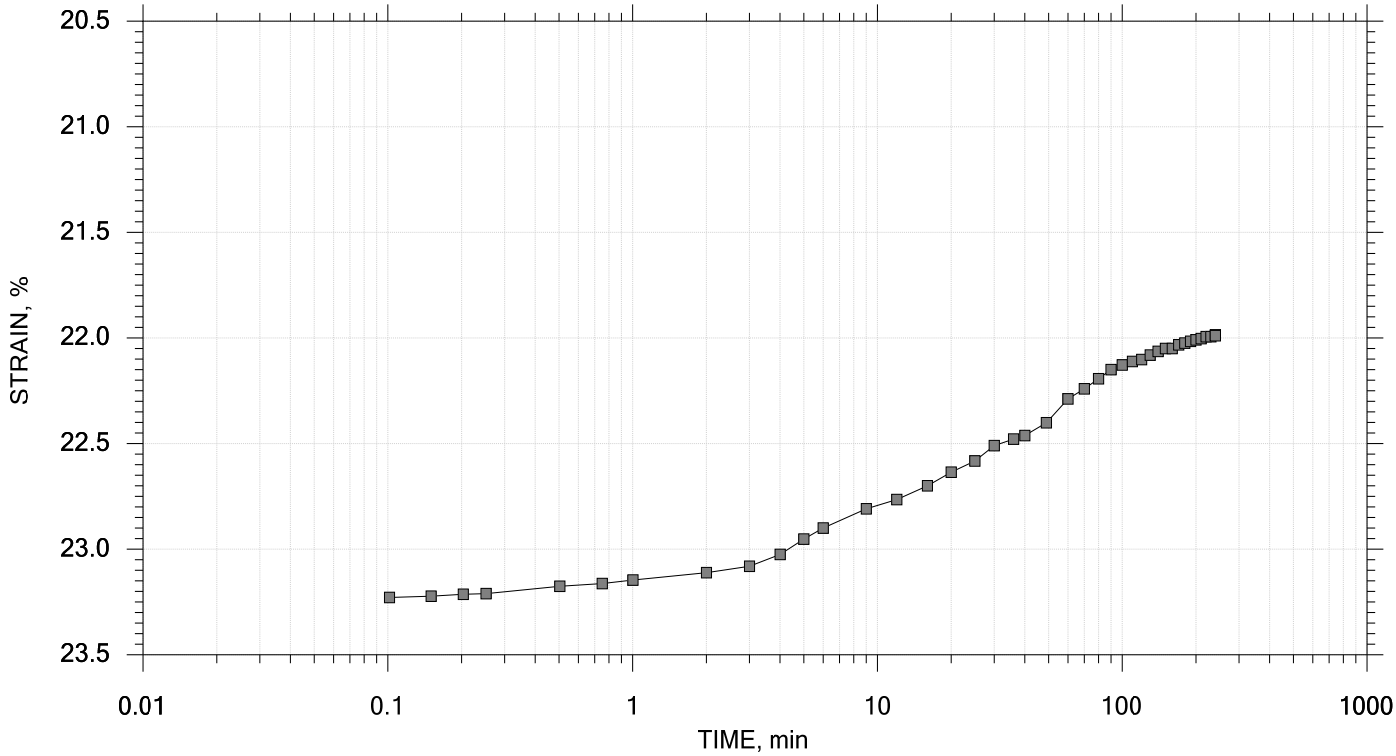
GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-202	Tested By: md	Checked By: njh
	Sample No.: U1	Test Date: 10/16/17	Test No.: IP-2
	Depth: 35-37 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System Y, Swell Pressure = 0.0666 tsf		

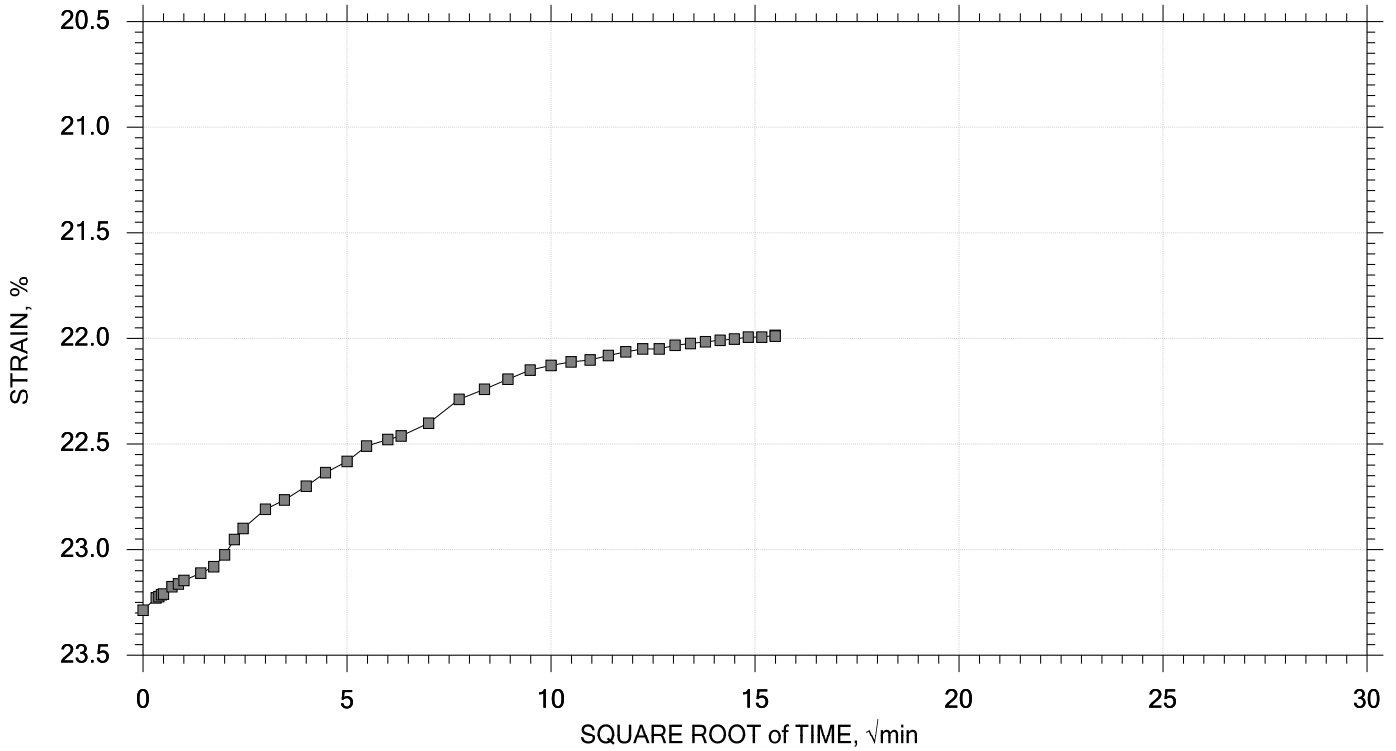
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 20 of 21

Stress: 0.125 tsf





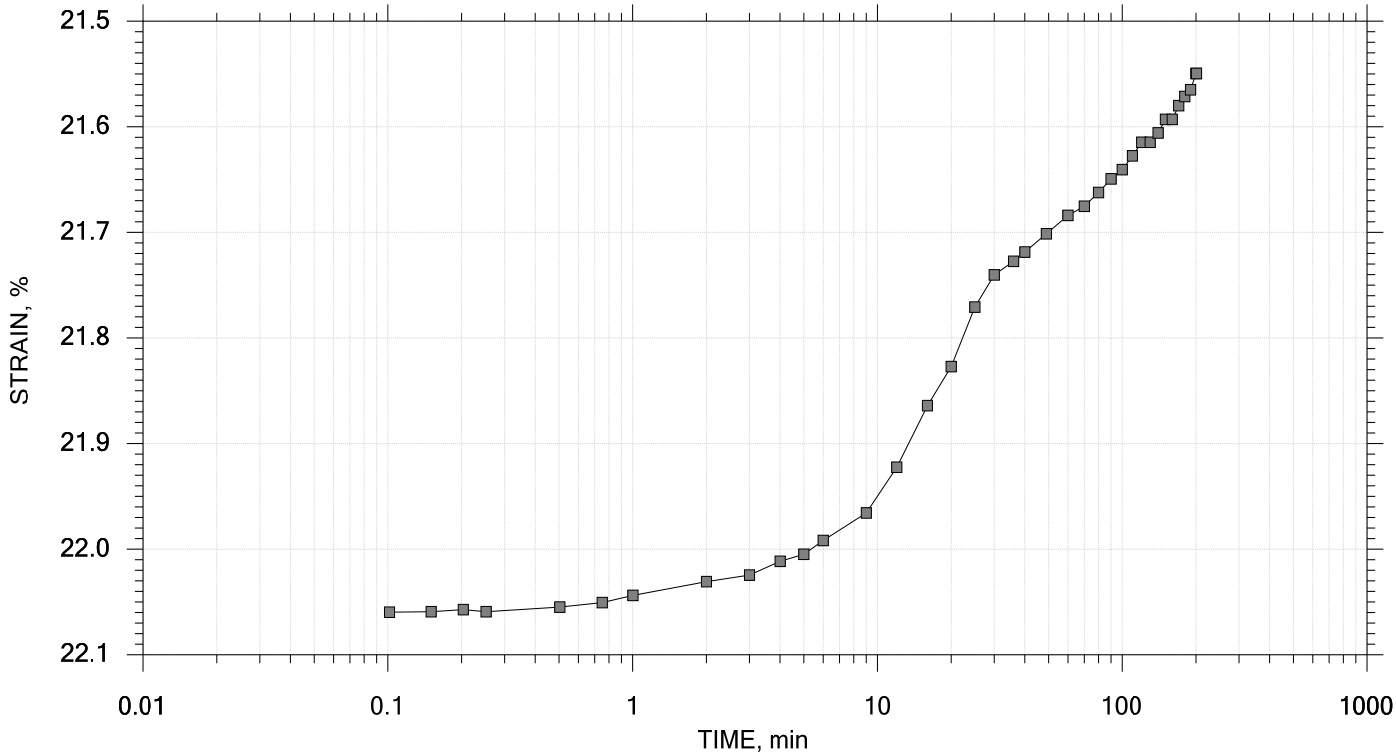
GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-202	Tested By: md	Checked By: njh
	Sample No.: U1	Test Date: 10/16/17	Test No.: IP-2
	Depth: 35-37 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System Y, Swell Pressure = 0.0666 tsf		

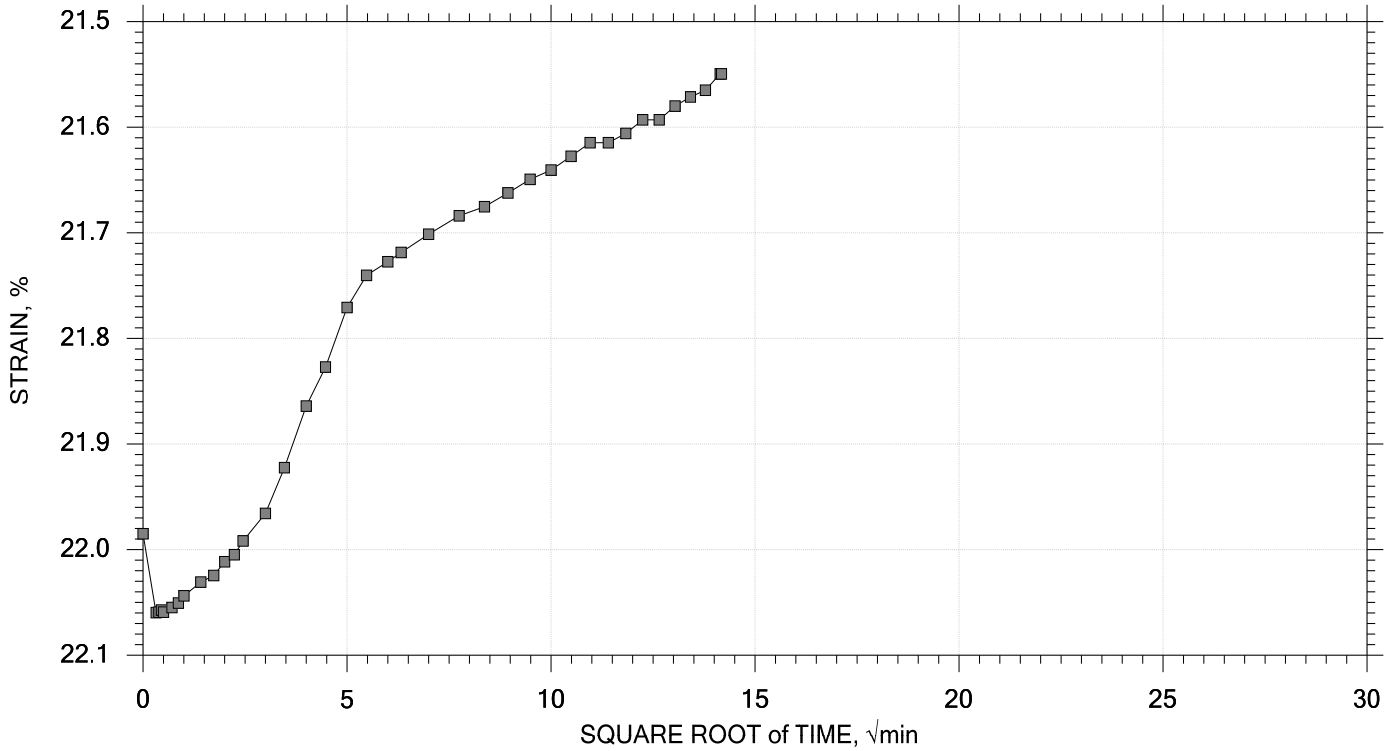
One-Dimensional Consolidation by ASTM D2435 - Method B

TIME CURVES

Constant Load Step 21 of 21

Stress: 0.0625 tsf

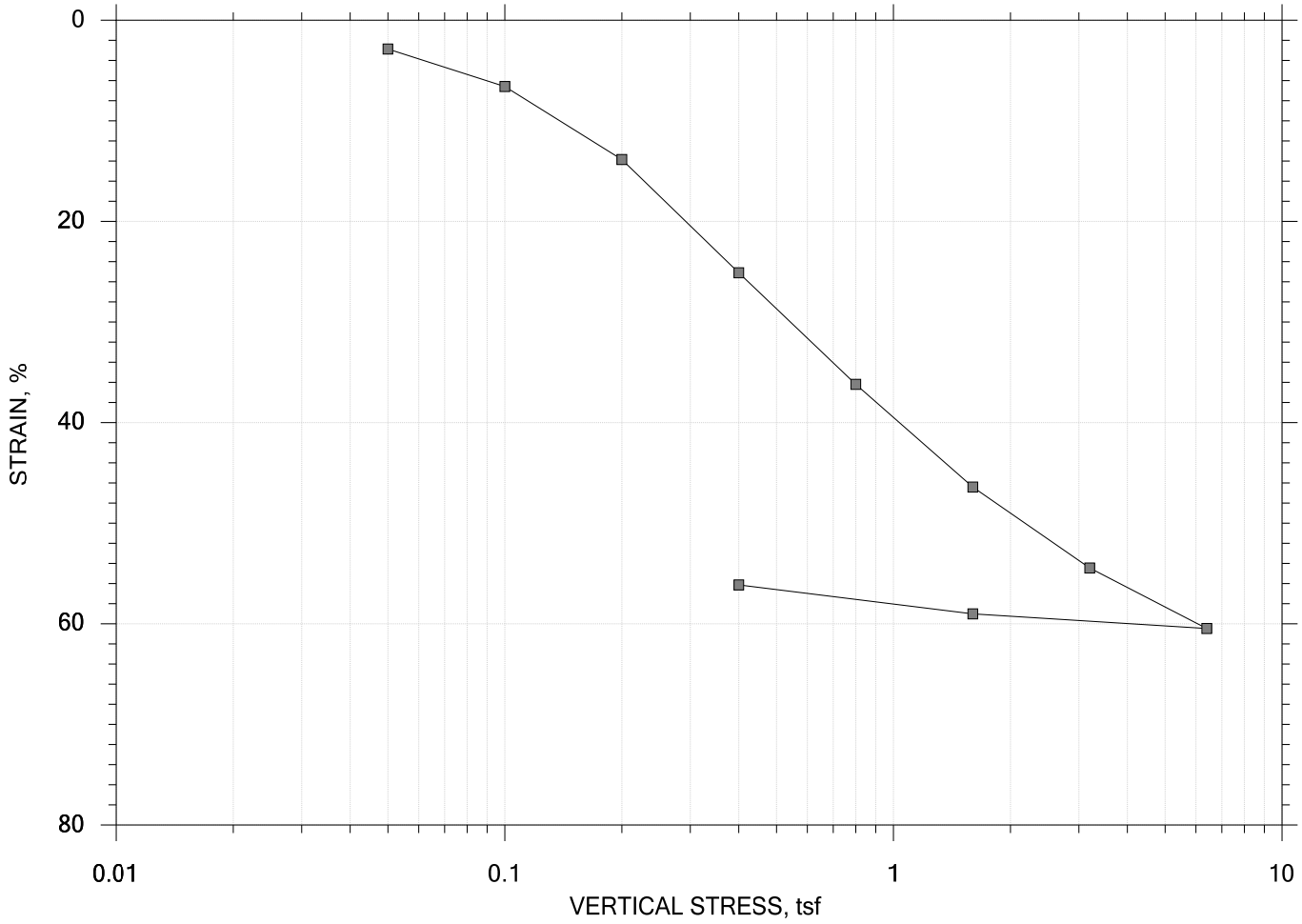


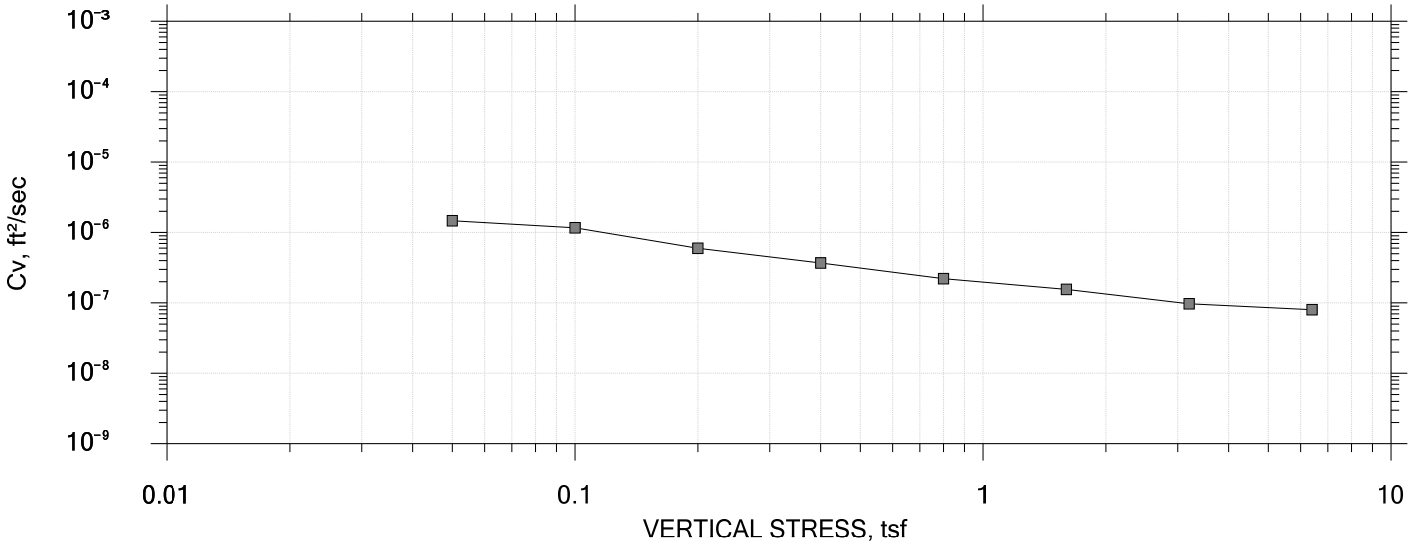


GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-202	Tested By: md	Checked By: njh
	Sample No.: U1	Test Date: 10/16/17	Test No.: IP-2
	Depth: 35-37 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System Y, Swell Pressure = 0.0666 tsf		

One-Dimensional Consolidation by ASTM D2435 - Method A

SUMMARY REPORT

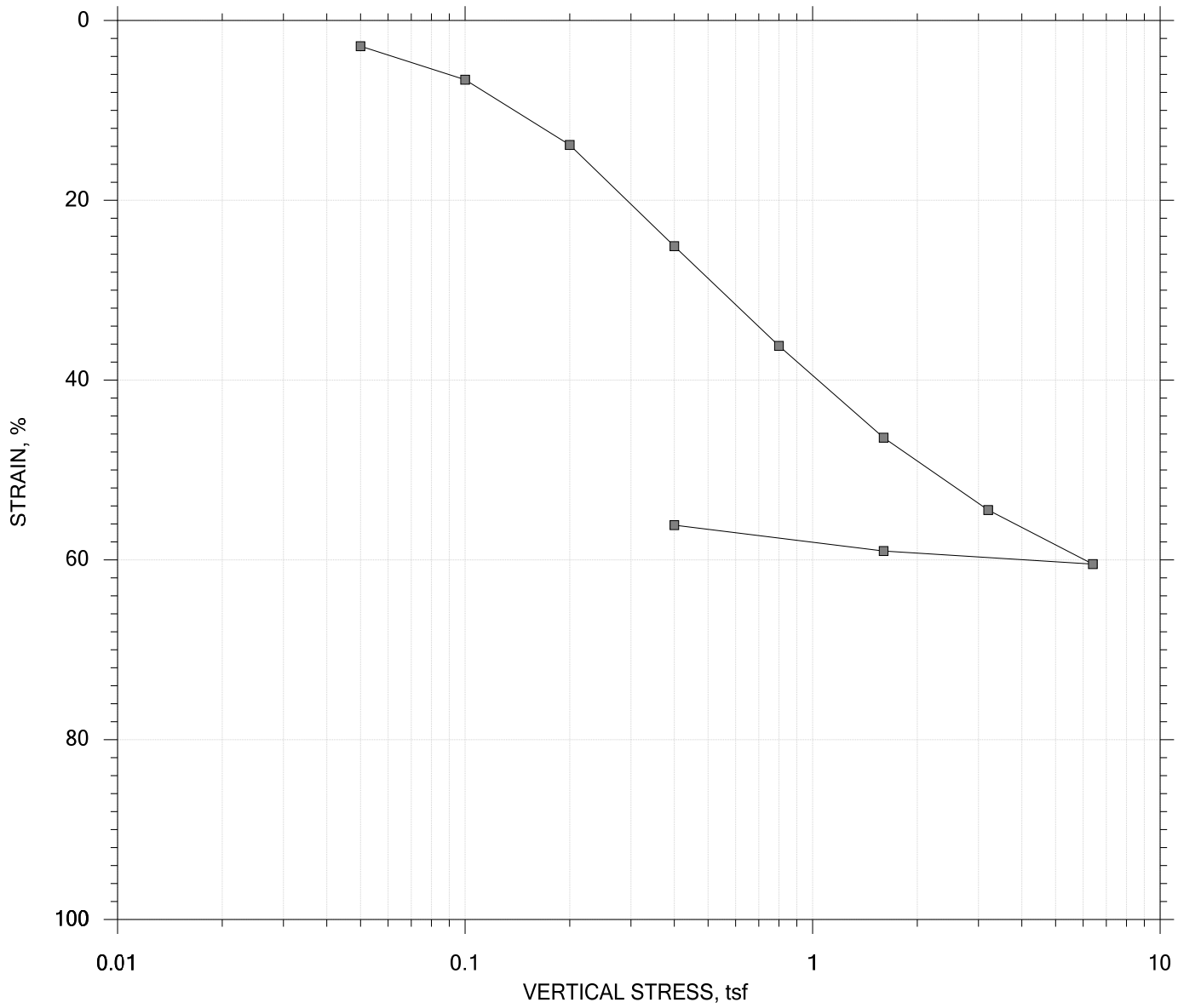




GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-203A	Tested By: md	Checked By: njh
	Sample No.: U-1	Test Date: 10/30/17	Test No.: IP-3
	Depth: 6.5-8.5 ft	Sample Type: intact	Elevation: ---
	Description: Wet, very dark brown silt with sand and organics		
	Remarks: System R&D		
	Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method A

SUMMARY REPORT



					Before Test	After Test
Current Vertical Effective Stress: ---				Water Content, %	178.61	66.58
Preconsolidation Stress: ---				Dry Unit Weight, pcf	26.705	58.054
Compression Ratio: ---				Saturation, %	92.63	100.00
Diameter: 2.5 in		Height: 1 in		Void Ratio	4.71	1.63
LL: 255	PL: 100	PI: 155	GS: 2.44			

GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-203A	Tested By: md	Checked By: njh
	Sample No.: U-1	Test Date: 10/30/17	Test No.: IP-3
	Depth: 6.5-8.5 ft	Sample Type: intact	Elevation: ---
	Description: Wet, very dark brown silt with sand and organics		
	Remarks: System R&D		
	Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method A

Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
Boring No.: BB-MAR-203A	Tested By: md	Checked By: njh
Sample No.: U-1	Test Date: 10/30/17	Depth: 6.5-8.5 ft
Test No.: IP-3	Sample Type: intact	Elevation: ---

Soil Description: Wet, very dark brown silt with sand and organics
Remarks: System R&D

Estimated Specific Gravity: 2.44	Liquid Limit: 255	Specimen Diameter: 2.50 in
Initial Void Ratio: 4.71	Plastic Limit: 100	Initial Height: 1.00 in
Final Void Ratio: 1.63	Plasticity Index: 155	Final Height: 0.46 in

	Before Consolidation		After Consolidation	
	Trimmings	Specimen+Ring	Specimen+Ring	Trimmings
Container ID	D-1450	RING		D1509
Wt. Container + Wet Soil, gm	71.960	206.75	168.20	65.550
Wt. Container + Dry Soil, gm	31.540	145.29	145.29	42.720
Wt. Container, gm	8.4500	110.88	110.88	8.4300
Wt. Dry Soil, gm	23.090	34.410	34.410	34.290
Water Content, %	175.05	178.61	66.58	66.58
Void Ratio	---	4.71	1.63	---
Degree of Saturation, %	---	92.63	100.00	---
Dry Unit Weight, pcf	---	26.705	58.054	---

Note: Specific Gravity and Void Ratios are calculated assuming the degree of saturation equals 100% at the end of the test. Therefore, values may not represent actual values for the specimen.

One-Dimensional Consolidation by ASTM D2435 - Method A

Project: Marsh Bridge | Location: Tremont/Southwest Harbor, ME | Project No.: GTX-306503
Boring No.: BB-MAR-203A | Tested By: md | Checked By: njh
Sample No.: U-1 | Test Date: 10/30/17 | Depth: 6.5-8.5 ft
Test No.: IP-3 | Sample Type: intact | Elevation: ---

Soil Description: Wet, very dark brown silt with sand and organics
Remarks: System R&D

Displacement at End of Increment

	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	Sq.Rt T90 min	Cv ft²/sec	Mv 1/tsf	k ft/day
1	0.0500	0.02862	4.54	2.86	108.875	2.19e-007	5.72e-001	3.38e-004
2	0.100	0.06587	4.33	6.59	112.937	1.97e-007	7.45e-001	3.96e-004
3	0.200	0.1384	3.92	13.8	115.834	1.71e-007	7.25e-001	3.34e-004
4	0.400	0.2510	3.28	25.1	126.225	1.26e-007	5.63e-001	1.91e-004
5	0.800	0.3618	2.64	36.2	130.148	9.07e-008	2.77e-001	6.77e-005
6	1.60	0.4640	2.06	46.4	129.842	6.51e-008	1.28e-001	2.24e-005
7	3.20	0.5445	1.60	54.4	130.629	4.62e-008	5.03e-002	6.26e-006
8	6.40	0.6046	1.26	60.5	128.859	3.45e-008	1.88e-002	1.75e-006
9	1.60	0.5900	1.34	59.0	136.050	2.92e-008	3.03e-003	2.39e-007
10	0.400	0.5614	1.50	56.1	175.916	2.51e-008	2.39e-002	1.62e-006

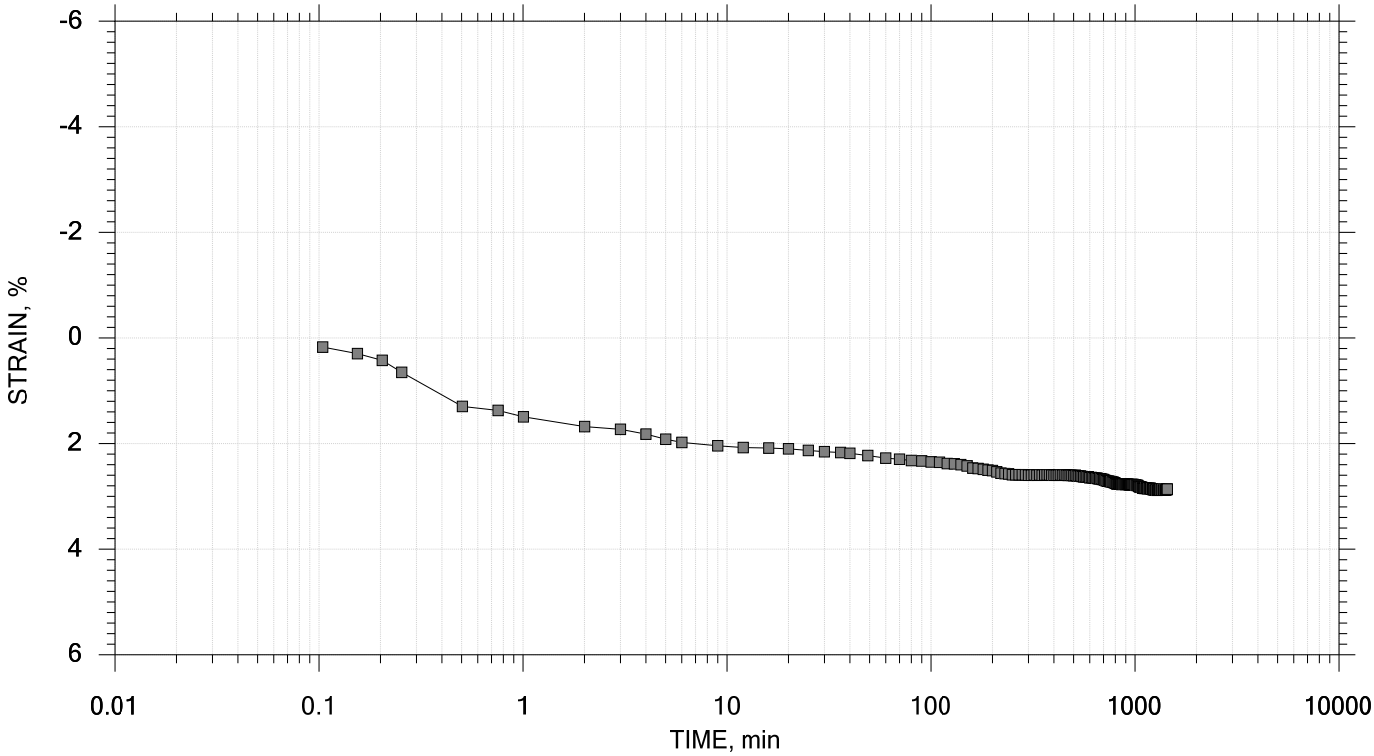
	Applied Stress tsf	Final Displacement in	Void Ratio	Strain at End %	Log T50 min	Cv ft²/sec	Mv 1/tsf	k ft/day	Ca %
1	0.0500	0.02862	4.54	2.86	0.000	0.00e+000	5.72e-001	0.00e+000	0.00e+000
2	0.100	0.06587	4.33	6.59	0.000	0.00e+000	7.45e-001	0.00e+000	0.00e+000
3	0.200	0.1384	3.92	13.8	0.000	0.00e+000	7.25e-001	0.00e+000	0.00e+000
4	0.400	0.2510	3.28	25.1	0.000	0.00e+000	5.63e-001	0.00e+000	0.00e+000
5	0.800	0.3618	2.64	36.2	11.938	2.30e-007	2.77e-001	1.72e-004	0.00e+000
6	1.60	0.4640	2.06	46.4	12.437	1.58e-007	1.28e-001	5.44e-005	0.00e+000
7	3.20	0.5445	1.60	54.4	0.000	0.00e+000	5.03e-002	0.00e+000	0.00e+000
8	6.40	0.6046	1.26	60.5	0.000	0.00e+000	1.88e-002	0.00e+000	0.00e+000
9	1.60	0.5900	1.34	59.0	0.000	0.00e+000	3.03e-003	0.00e+000	0.00e+000
10	0.400	0.5614	1.50	56.1	0.000	0.00e+000	2.39e-002	0.00e+000	0.00e+000

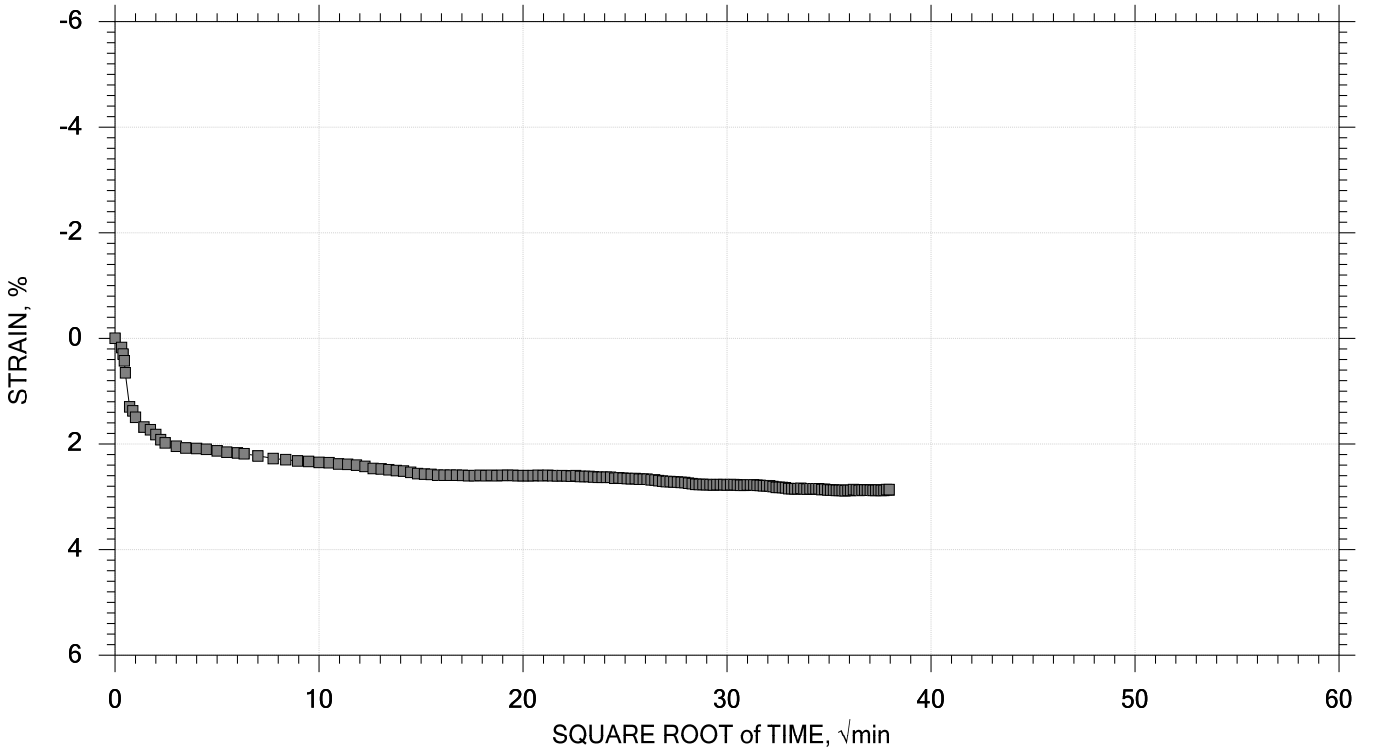
One-Dimensional Consolidation by ASTM D2435 - Method A

TIME CURVES

Constant Load Step 1 of 10

Stress: 0.05 tsf





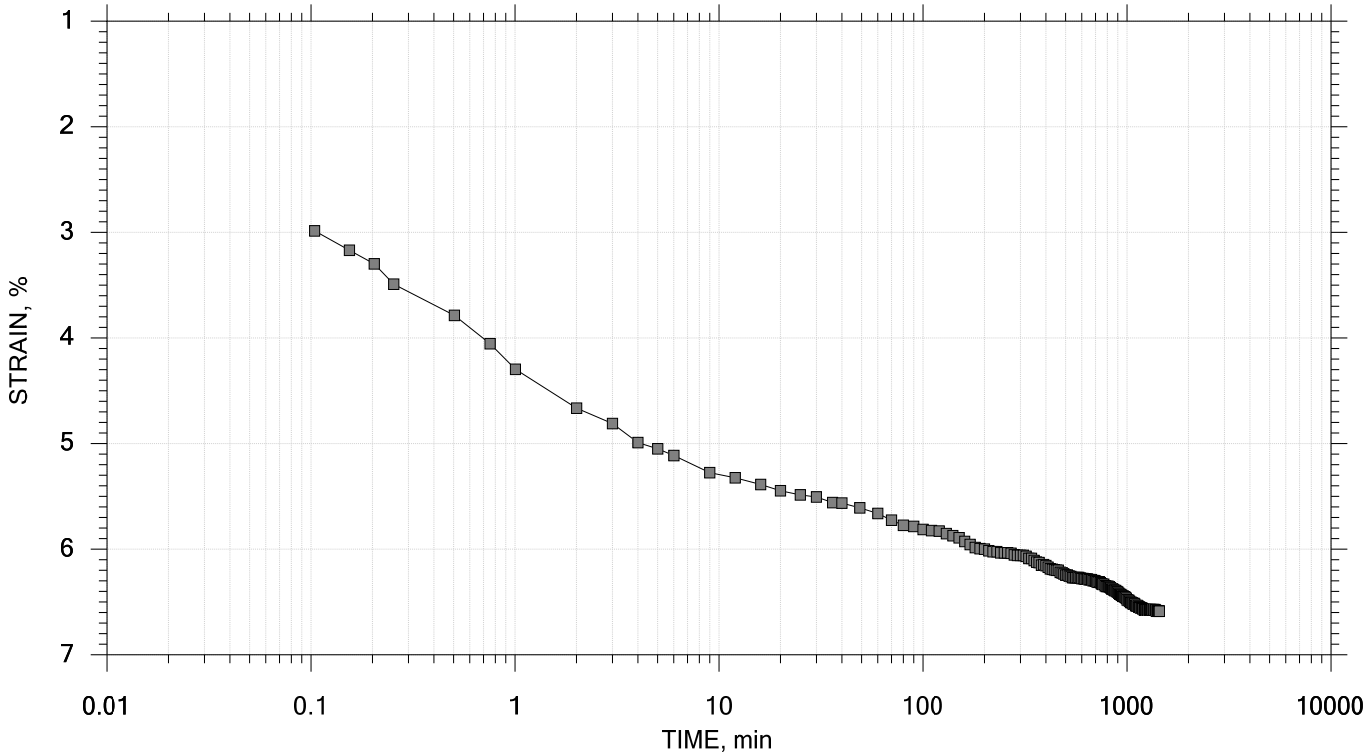
GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-203A	Tested By: md	Checked By: njh
	Sample No.: U-1	Test Date: 10/30/17	Test No.: IP-3
	Depth: 6.5-8.5 ft	Sample Type: intact	Elevation: ---
	Description: Wet, very dark brown silt with sand and organics		
	Remarks: System R&D		

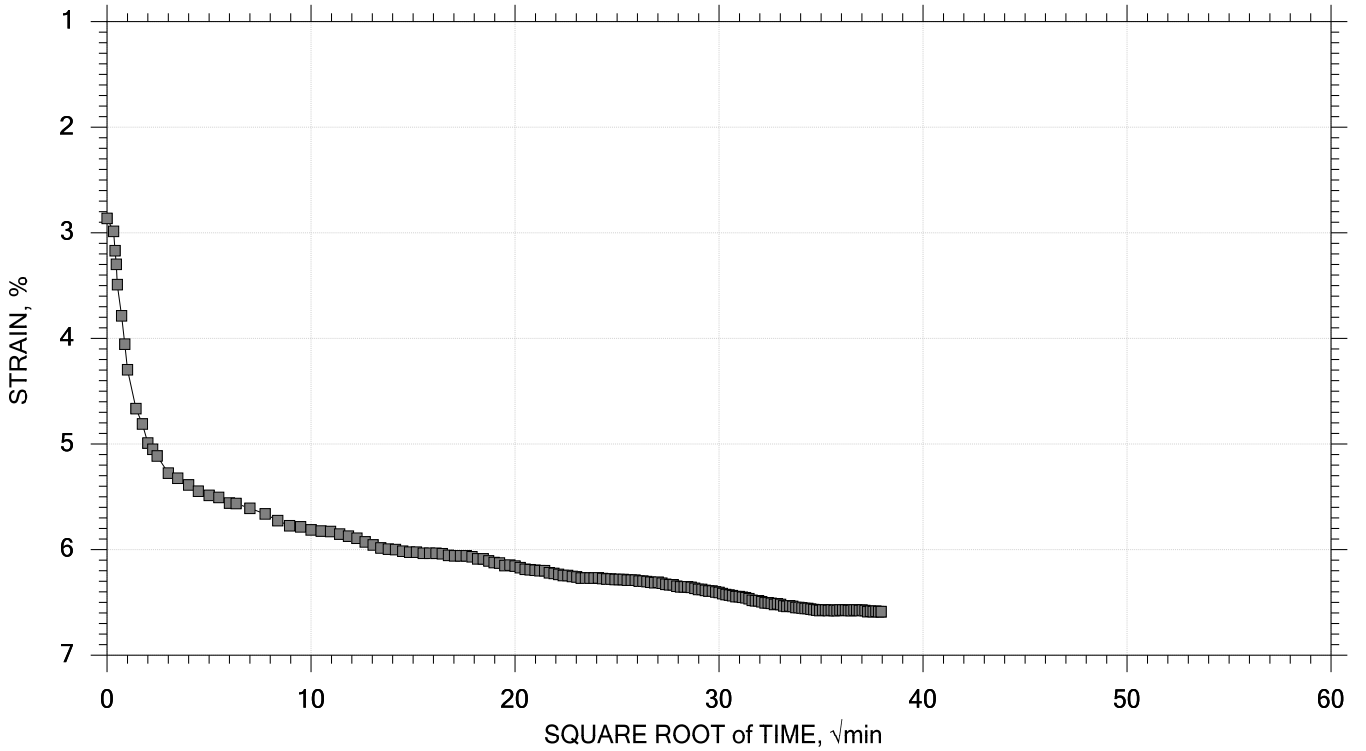
One-Dimensional Consolidation by ASTM D2435 - Method A

TIME CURVES

Constant Load Step 2 of 10

Stress: 0.1 tsf





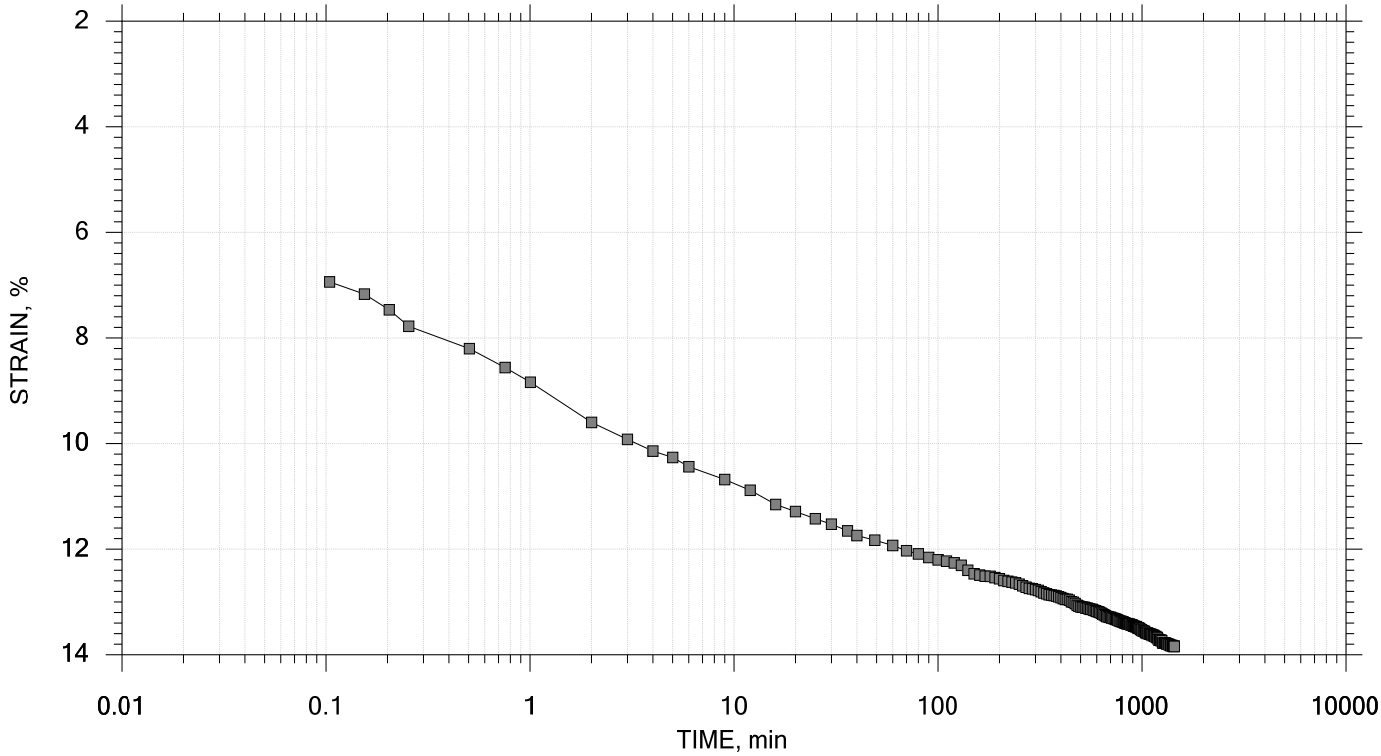
GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-203A	Tested By: md	Checked By: njh
	Sample No.: U-1	Test Date: 10/30/17	Test No.: IP-3
	Depth: 6.5-8.5 ft	Sample Type: intact	Elevation: ---
	Description: Wet, very dark brown silt with sand and organics		
	Remarks: System R&D		

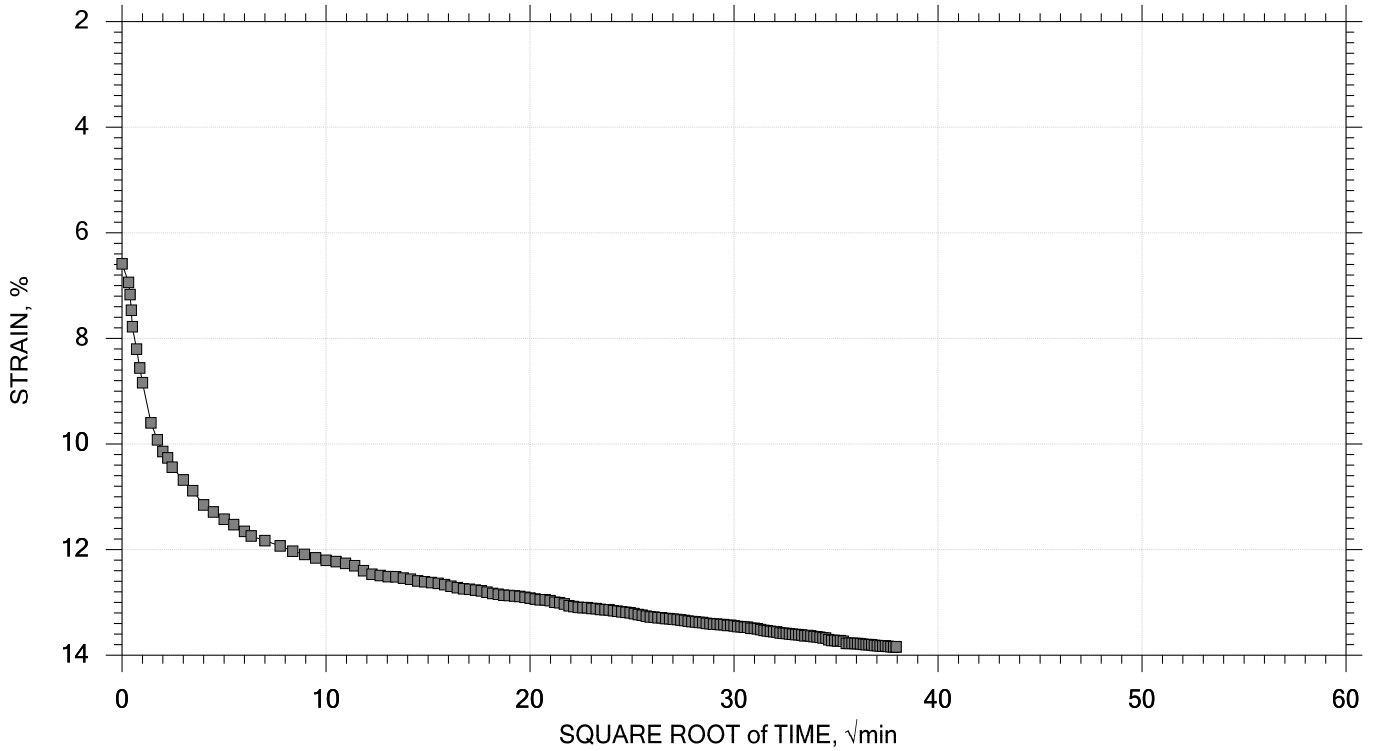
One-Dimensional Consolidation by ASTM D2435 - Method A

TIME CURVES

Constant Load Step 3 of 10

Stress: 0.2 tsf





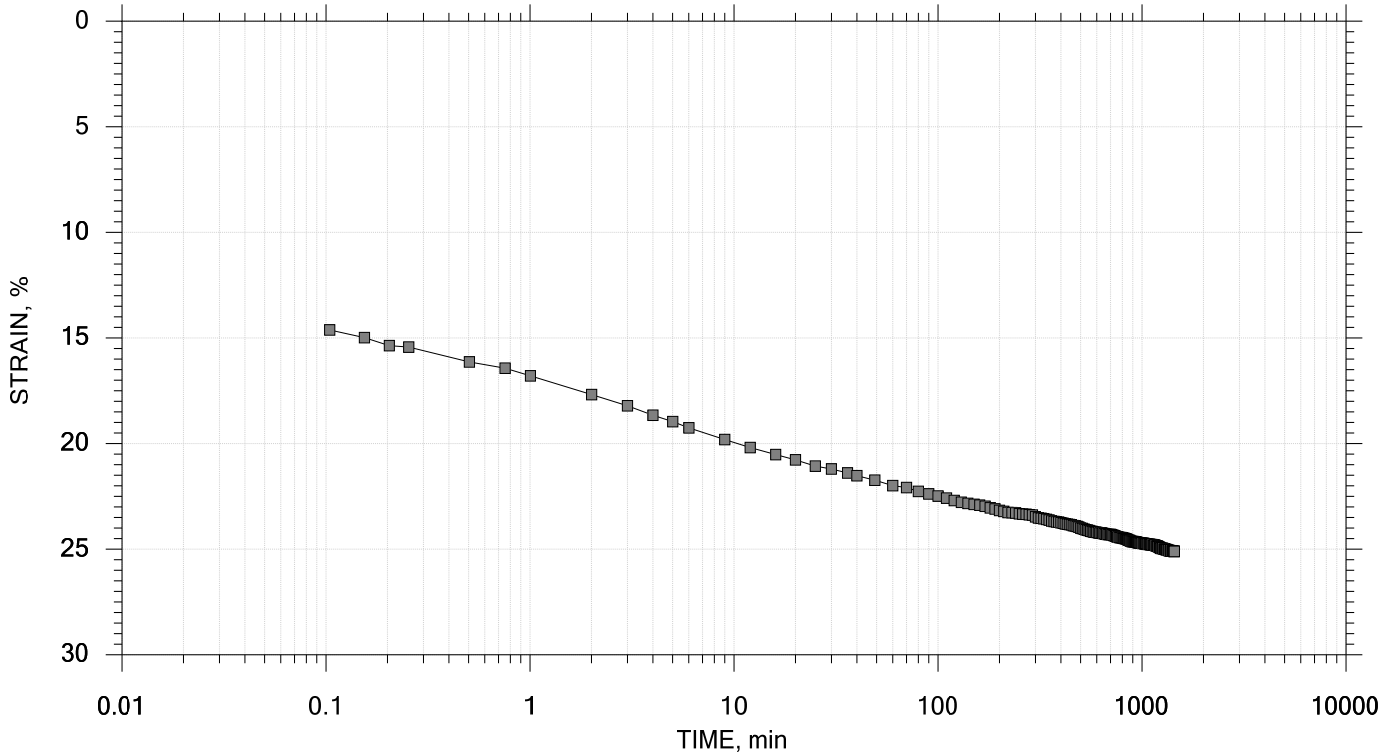
GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-203A	Tested By: md	Checked By: njh
	Sample No.: U-1	Test Date: 10/30/17	Test No.: IP-3
	Depth: 6.5-8.5 ft	Sample Type: intact	Elevation: ---
	Description: Wet, very dark brown silt with sand and organics		
	Remarks: System R&D		

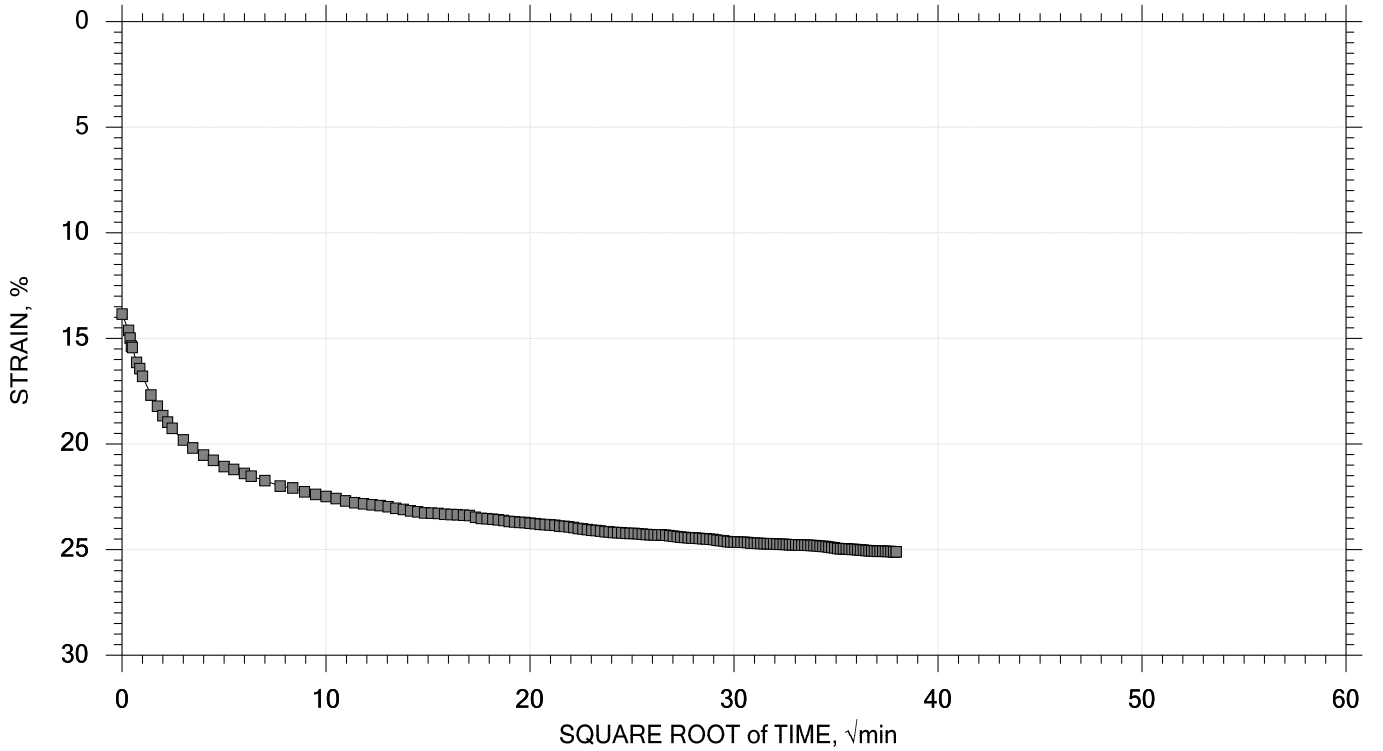
One-Dimensional Consolidation by ASTM D2435 - Method A

TIME CURVES

Constant Load Step 4 of 10

Stress: 0.4 tsf





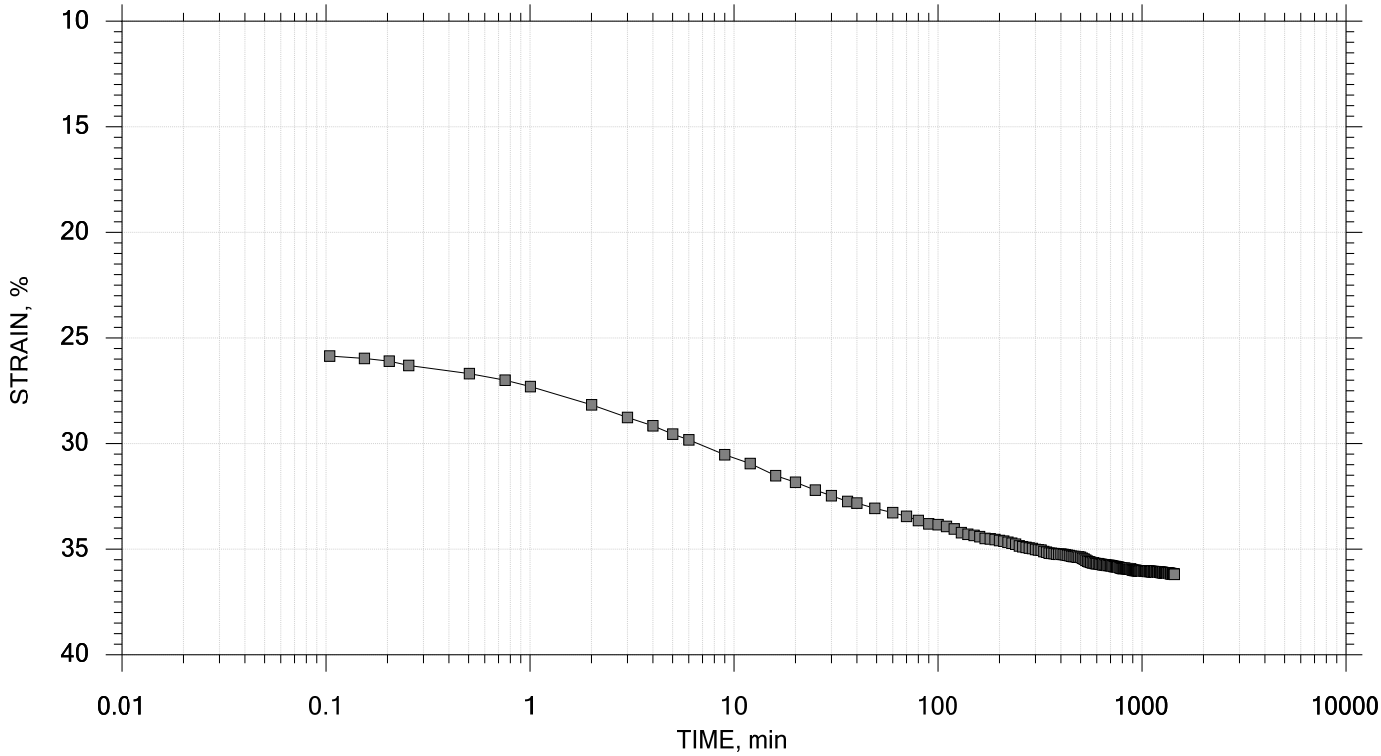
GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-203A	Tested By: md	Checked By: njh
	Sample No.: U-1	Test Date: 10/30/17	Test No.: IP-3
	Depth: 6.5-8.5 ft	Sample Type: intact	Elevation: ---
	Description: Wet, very dark brown silt with sand and organics		
	Remarks: System R&D		

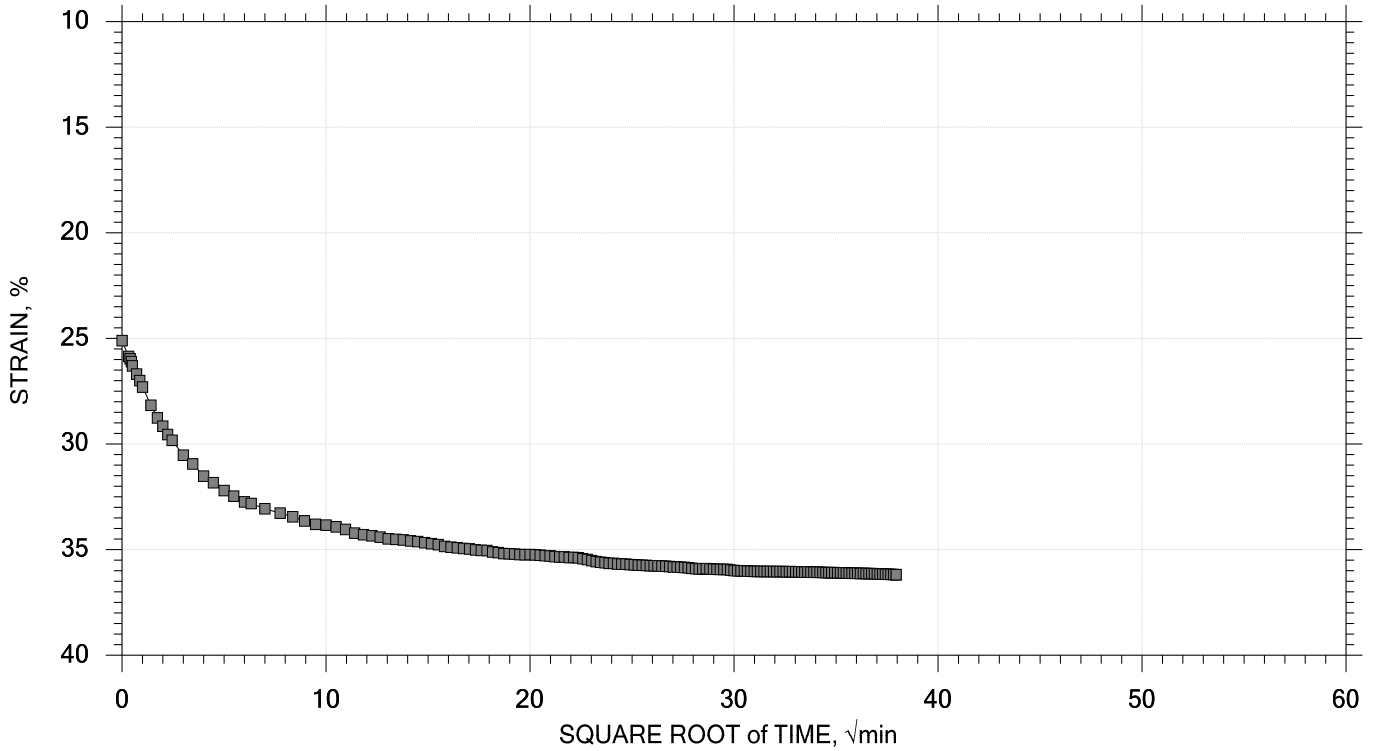
One-Dimensional Consolidation by ASTM D2435 - Method A

TIME CURVES

Constant Load Step 5 of 10

Stress: 0.8 tsf





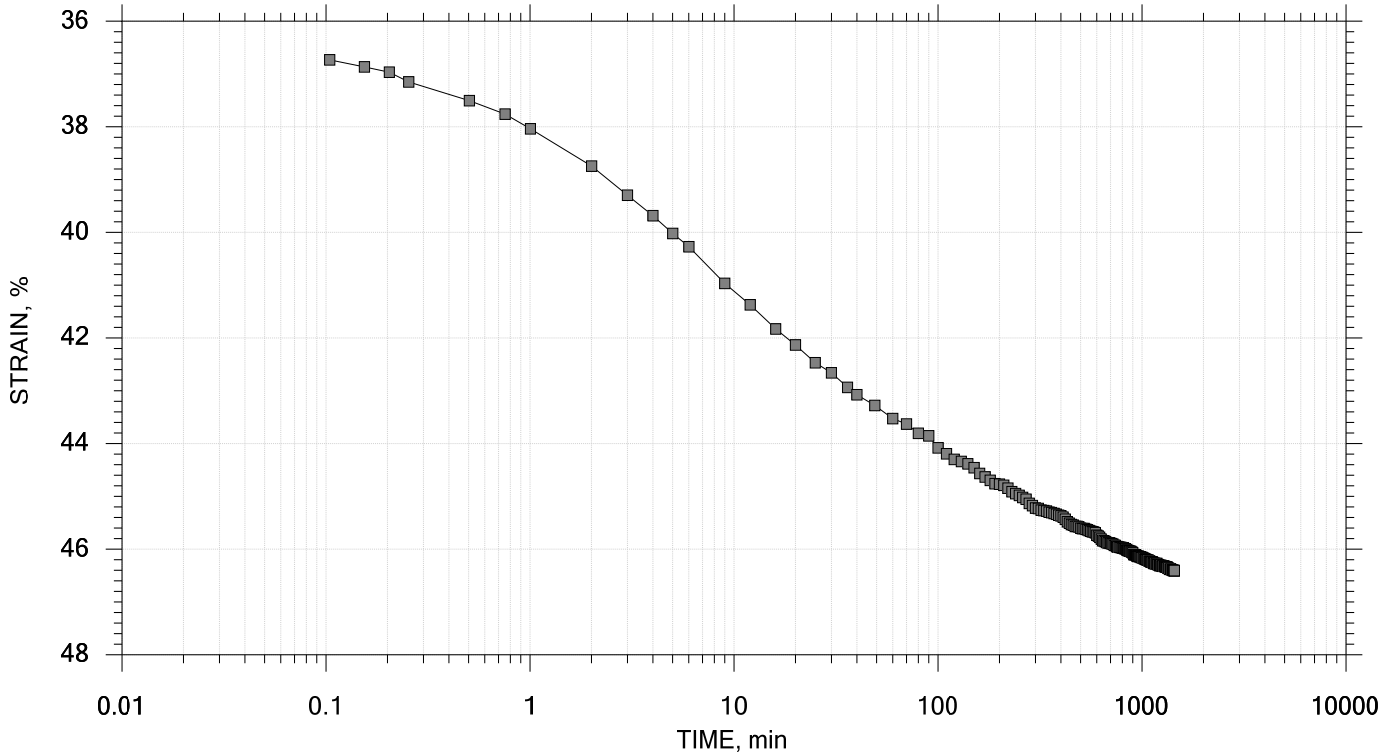
GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-203A	Tested By: md	Checked By: njh
	Sample No.: U-1	Test Date: 10/30/17	Test No.: IP-3
	Depth: 6.5-8.5 ft	Sample Type: intact	Elevation: ---
	Description: Wet, very dark brown silt with sand and organics		
	Remarks: System R&D		

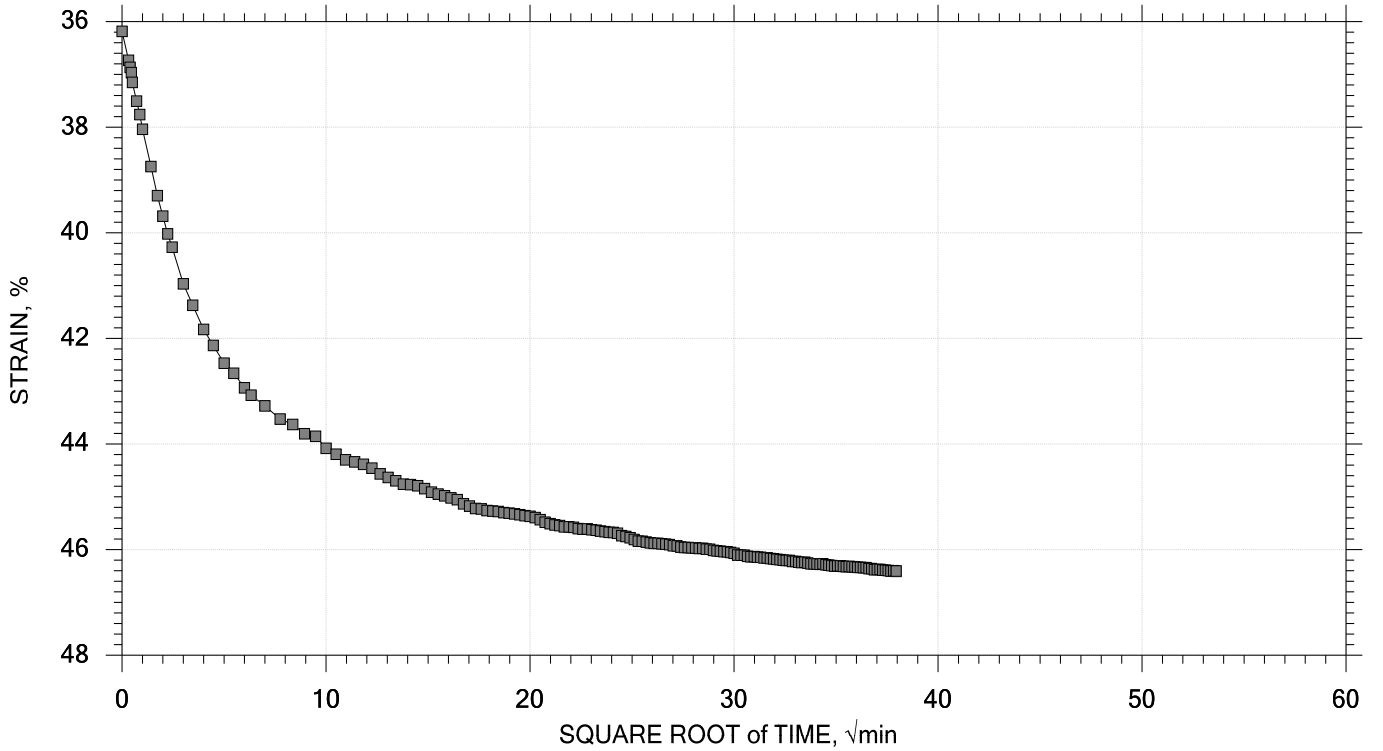
One-Dimensional Consolidation by ASTM D2435 - Method A

TIME CURVES

Constant Load Step 6 of 10

Stress: 1.6 tsf





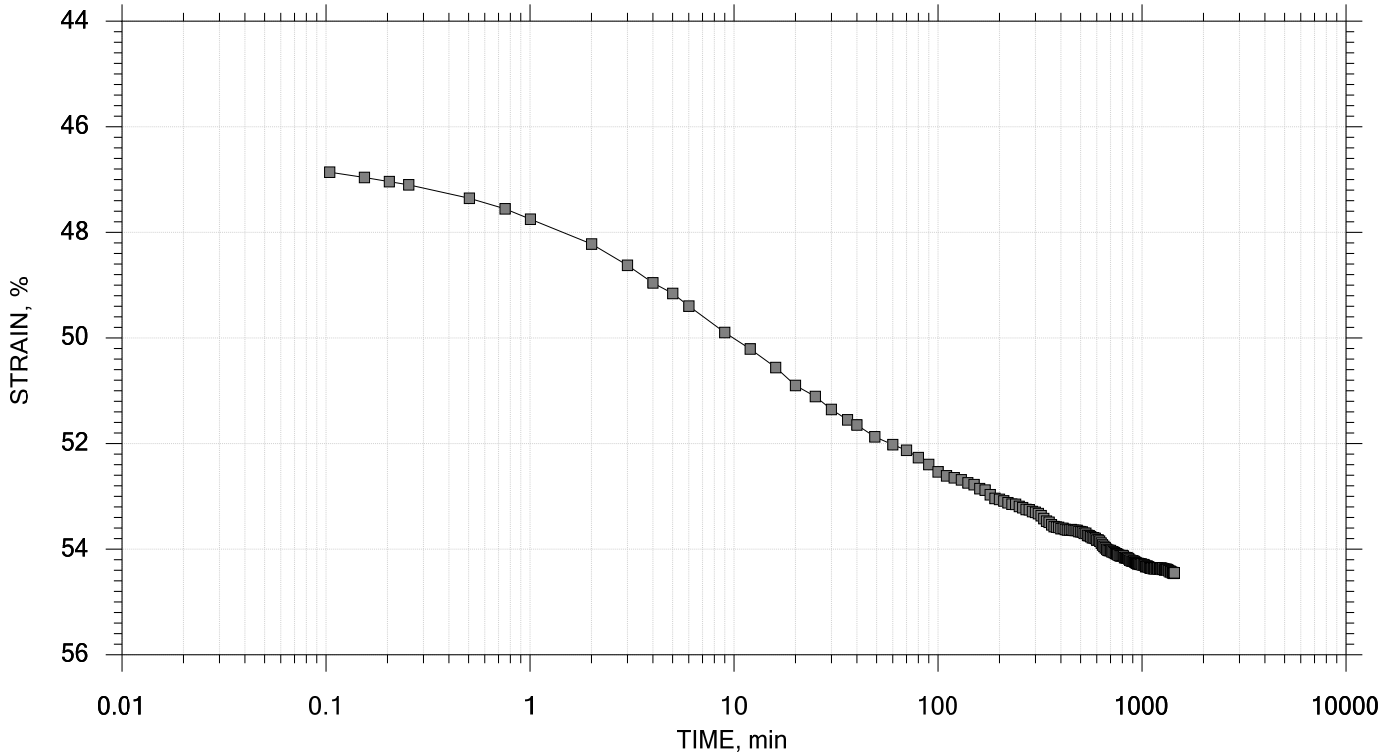
GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-203A	Tested By: md	Checked By: njh
	Sample No.: U-1	Test Date: 10/30/17	Test No.: IP-3
	Depth: 6.5-8.5 ft	Sample Type: intact	Elevation: ---
	Description: Wet, very dark brown silt with sand and organics		
	Remarks: System R&D		

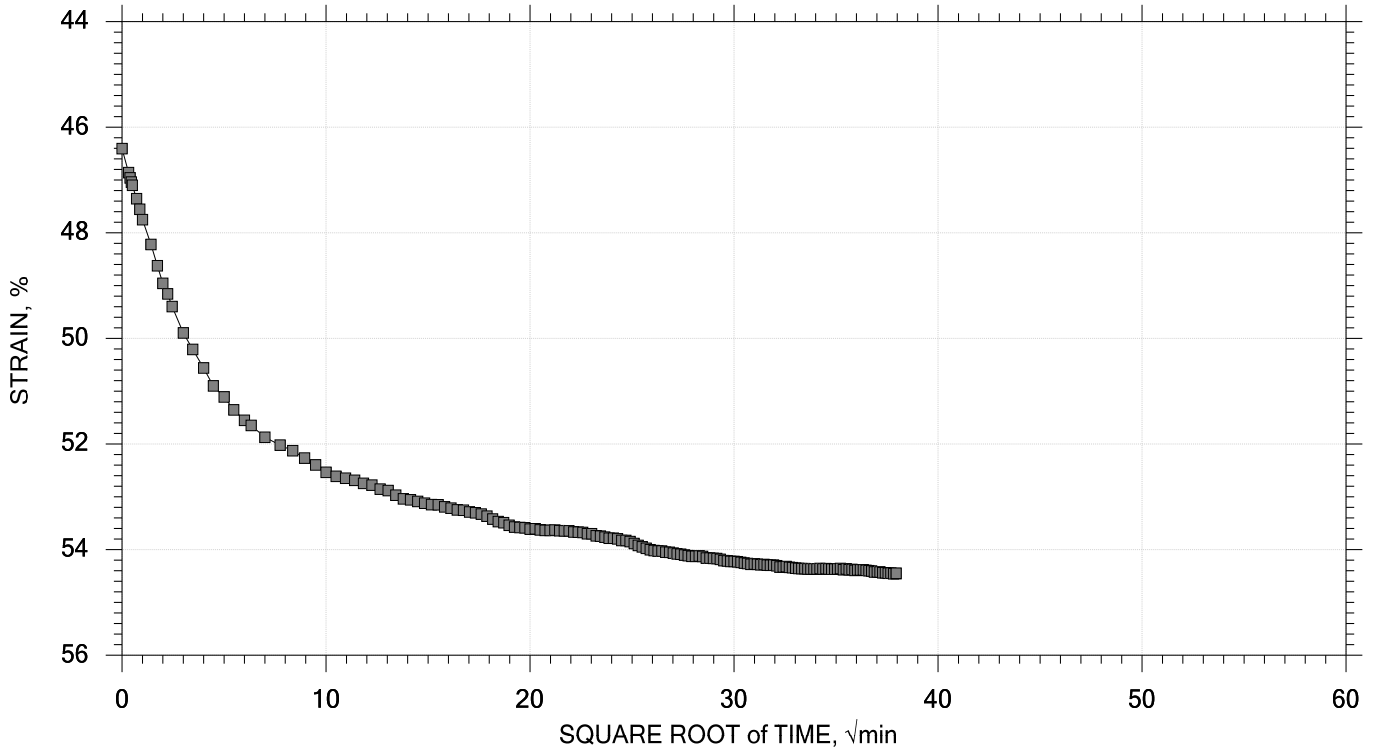
One-Dimensional Consolidation by ASTM D2435 - Method A

TIME CURVES

Constant Load Step 7 of 10

Stress: 3.2 tsf





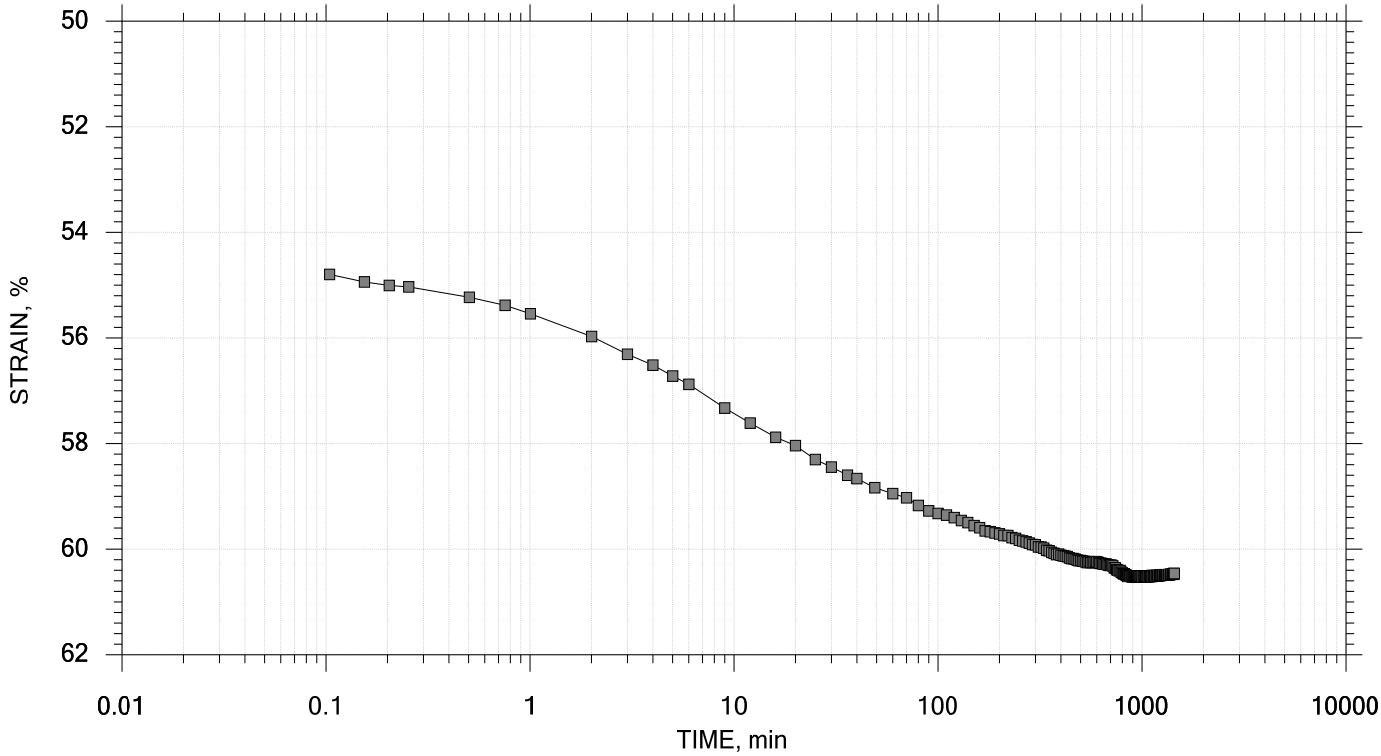
GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-203A	Tested By: md	Checked By: njh
	Sample No.: U-1	Test Date: 10/30/17	Test No.: IP-3
	Depth: 6.5-8.5 ft	Sample Type: intact	Elevation: ---
	Description: Wet, very dark brown silt with sand and organics		
	Remarks: System R&D		

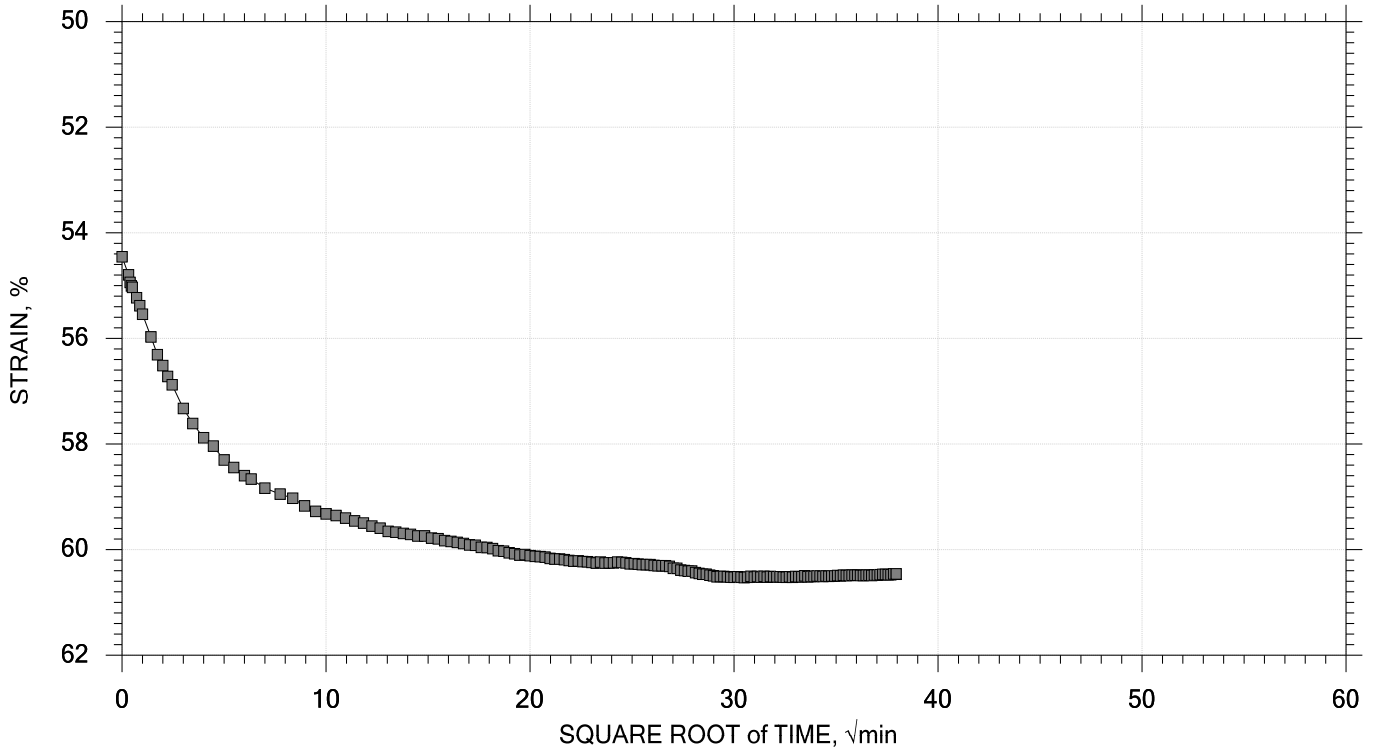
One-Dimensional Consolidation by ASTM D2435 - Method A

TIME CURVES

Constant Load Step 8 of 10

Stress: 6.4 tsf





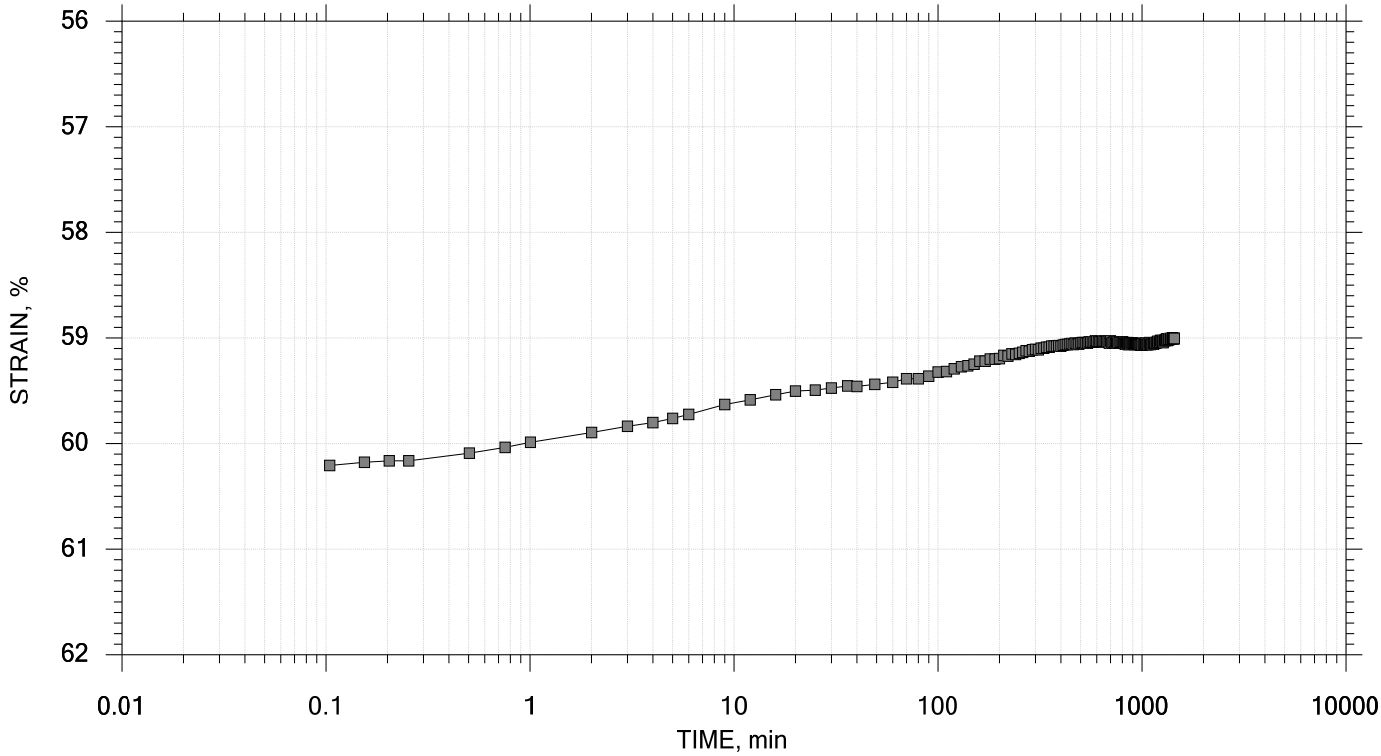
GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-203A	Tested By: md	Checked By: njh
	Sample No.: U-1	Test Date: 10/30/17	Test No.: IP-3
	Depth: 6.5-8.5 ft	Sample Type: intact	Elevation: ---
	Description: Wet, very dark brown silt with sand and organics		
	Remarks: System R&D		

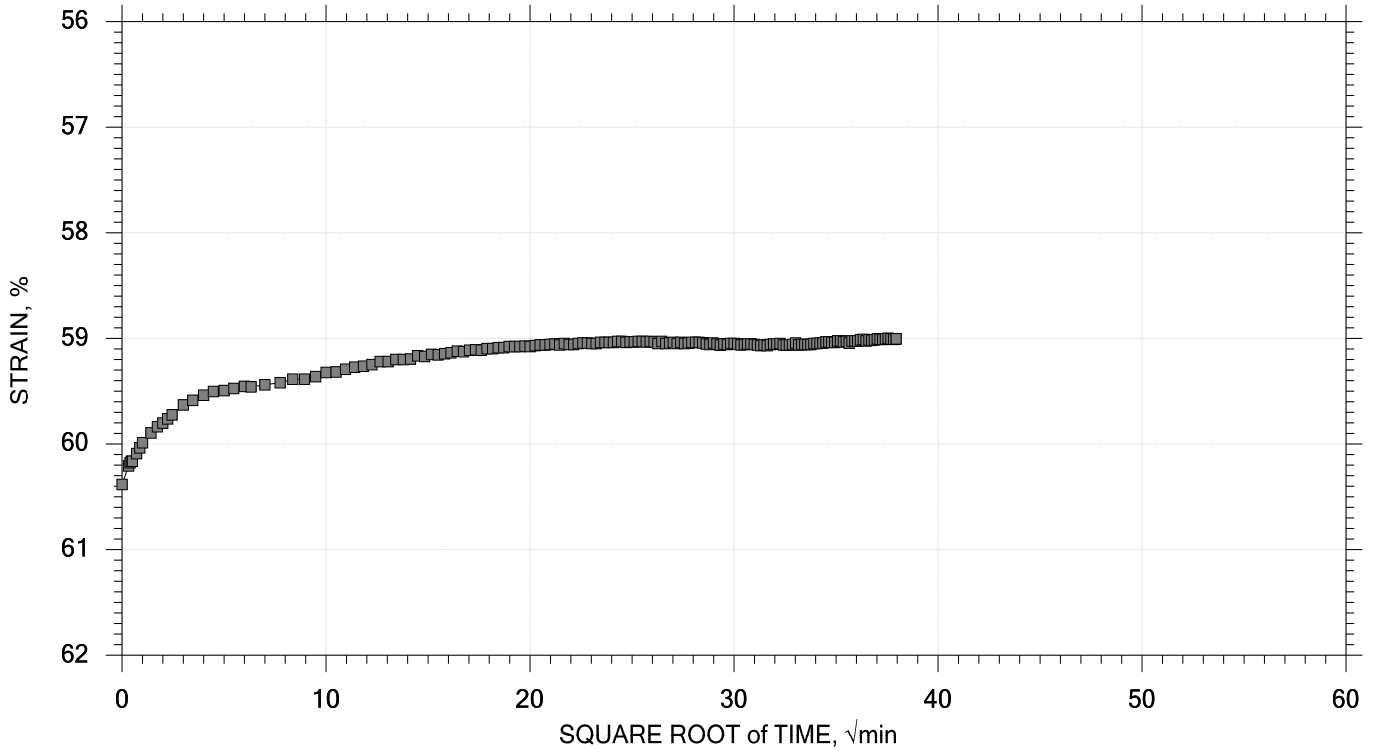
One-Dimensional Consolidation by ASTM D2435 - Method A

TIME CURVES

Constant Load Step 9 of 10

Stress: 1.6 tsf





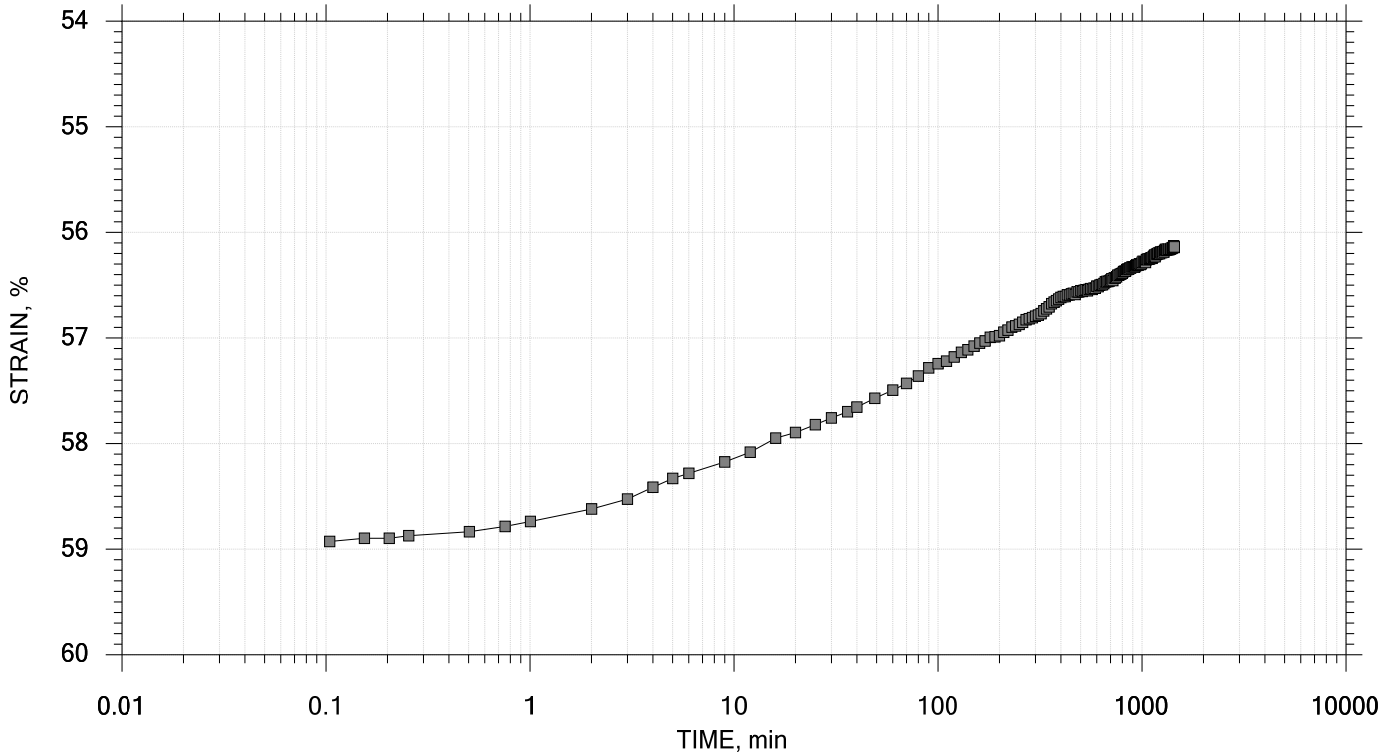
GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-203A	Tested By: md	Checked By: njh
	Sample No.: U-1	Test Date: 10/30/17	Test No.: IP-3
	Depth: 6.5-8.5 ft	Sample Type: intact	Elevation: ---
	Description: Wet, very dark brown silt with sand and organics		
	Remarks: System R&D		

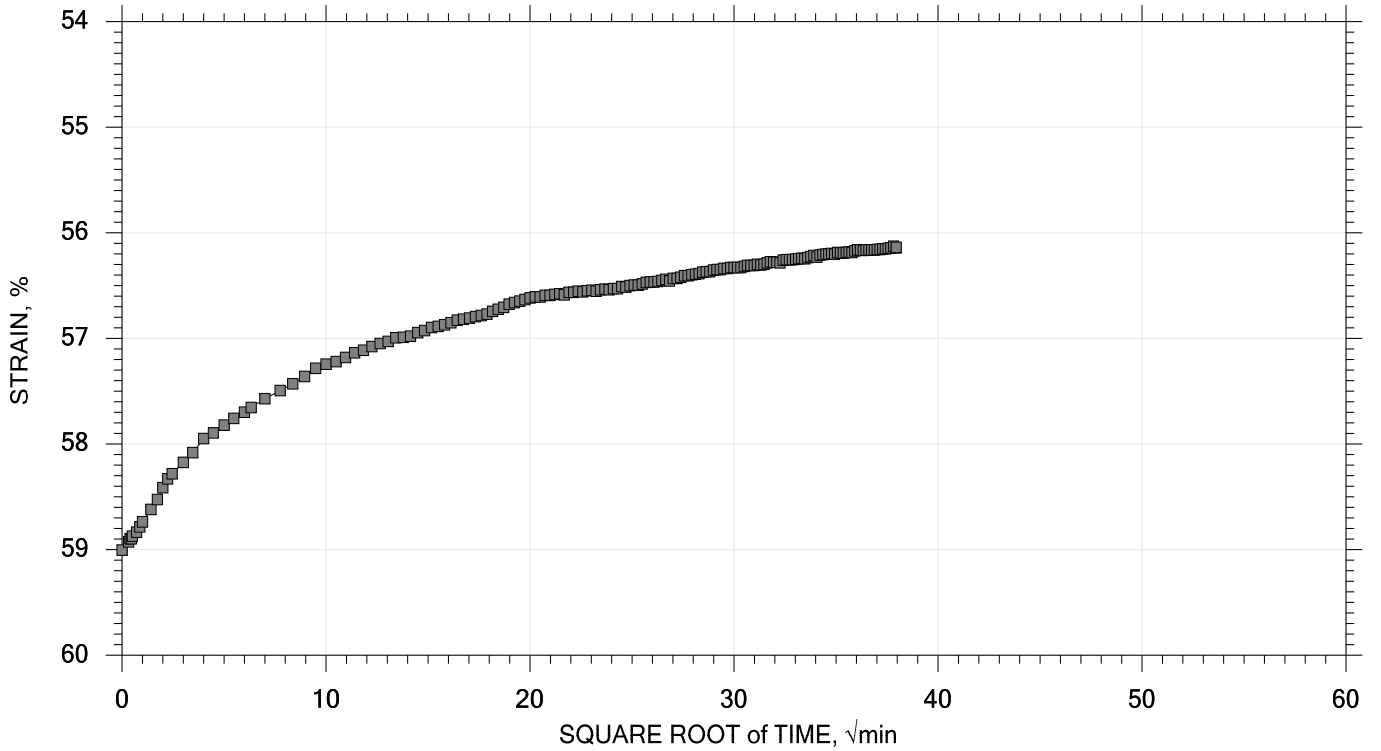
One-Dimensional Consolidation by ASTM D2435 - Method A

TIME CURVES

Constant Load Step 10 of 10

Stress: 0.4 tsf





GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-203A	Tested By: md	Checked By: njh
	Sample No.: U-1	Test Date: 10/30/17	Test No.: IP-3
	Depth: 6.5-8.5 ft	Sample Type: intact	Elevation: ---
	Description: Wet, very dark brown silt with sand and organics		
	Remarks: System R&D		

Consolidated Undrained Direct Simple Shear Testing of Cohesive Soils by ASTM D6528

Client:	Golder Associates	GTX#:	306503
Project Name:	Marsh Bridge	Test Date:	10/30/17
Project Location:	Tremont/Southwest Harbor, ME		

Boring ID: BB-MAR-203A
Sample ID: U1
Depth, ft: 6.5-8.5

Visual Description: Wet, very dark brown silt with sand and organics

Test Equipment: Top and bottom box (circular) = 2.50 in diameter. Load cells and LVDT's connected to data acquisition system for shear force, normal load, horizontal and vertical displacement; surface area = 4.91 in^2, soil height = 1 inch. Stacked rings used. Set up included porous stones with pins.

Test Condition: Inundated prior to consolidation

Sample Type and Preparation: density: Extruded from tube, cut, trimmed and placed into apparatus at as-received and moisture.

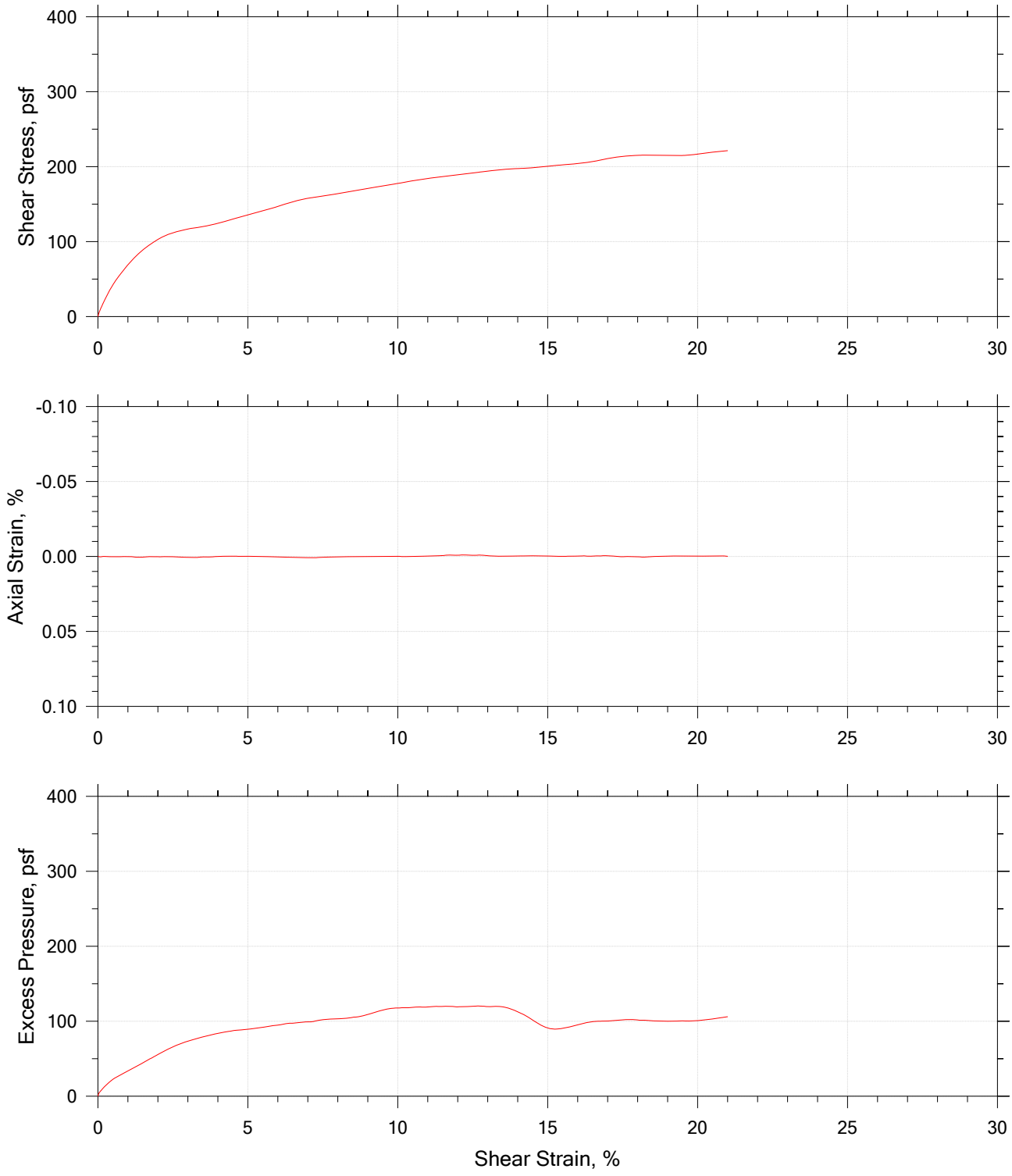
Parameter	Point 1	Point 2	Point 3	Point 4	Point 5
Test No.	DSS-1				
Initial Moisture Content, %	299.0				
Initial Dry Density, pcf	16.40				
Nominal Rate of Shear Strain, %/hr	5.0				
Vertical Consolidation Stress, psf	500				
Final Moisture Content, %	255.0				
Measured Peak Shear Stress, psf	217				
Shear Strain at Peak Shear Stress, %	20.0				
Membrane Correction, psf	49				
Corrected Peak Shear Stress, psf	168				
S_u / σ'_{vc}	0.34				

Comments:

Tested By: md Checked By: njh

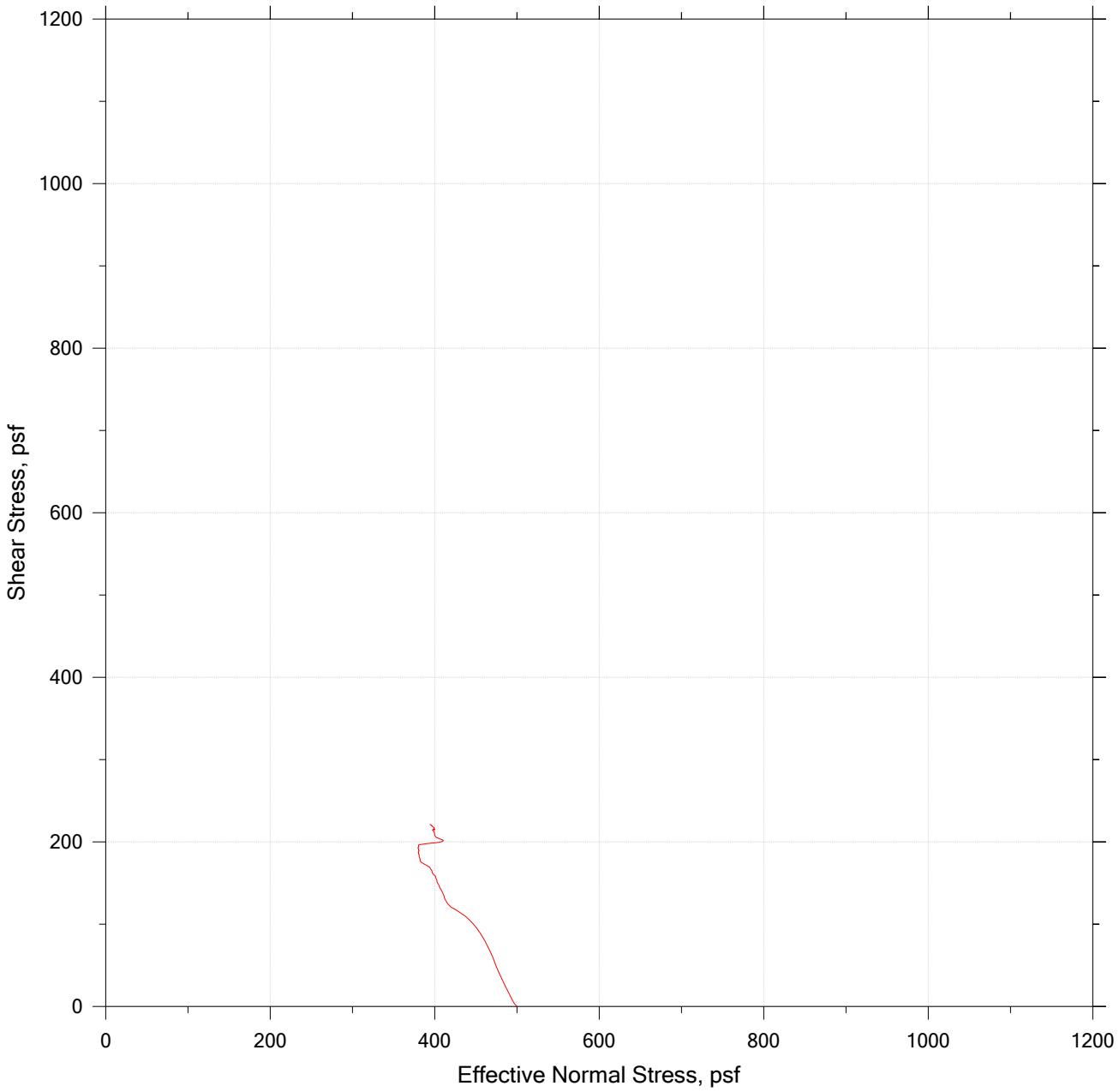
Notes: These results apply only to the sample tested for the specific test conditions. The test procedures employed follow accepted industry practice and the indicated test method. GeoTesting Express has no specific knowledge as to conditioning, origin, sampling procedure or intended use of the material.

DIRECT SIMPLE SHEAR TEST by ASTM D6528



GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/SW Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-203A	Tested By: md	Checked By: njh
	Sample No.: U1	Test Date: 10/30/17	Depth: 6.5-8.5 ft
	Test No.: DSS-1	Sample Type: intact	Elevation: ---
	Description: Wet, very dark brown silt with sand and organics		
	Remarks: System L		

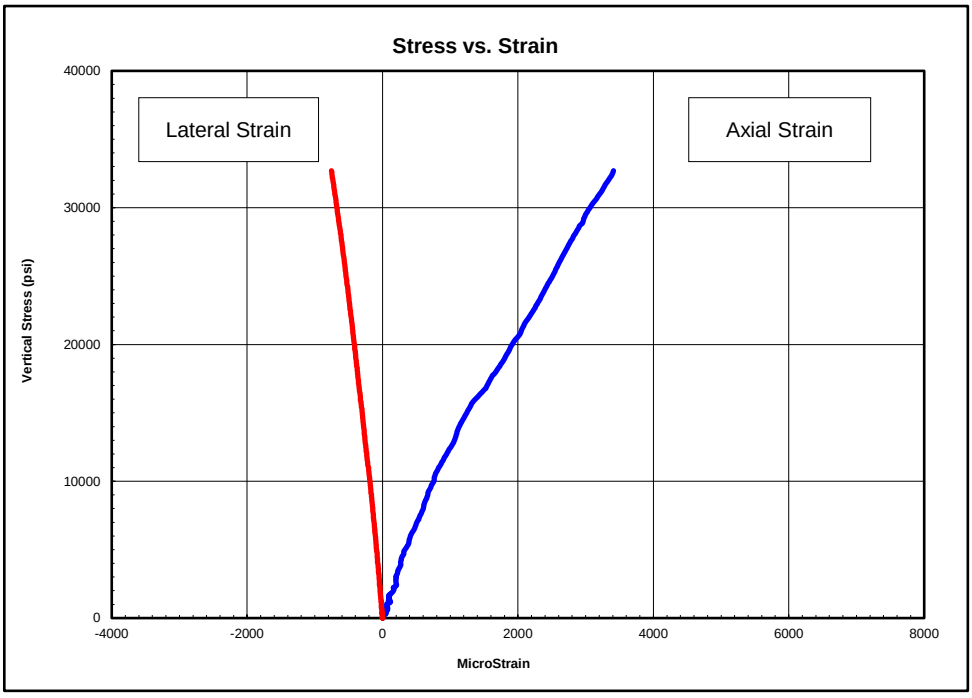
DIRECT SIMPLE SHEAR TEST by ASTM D6528



GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/SW Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-203A	Tested By: md	Checked By: njh
	Sample No.: U1	Test Date: 10/30/17	Depth: 6.5-8.5 ft
	Test No.: DSS-1	Sample Type: intact	Elevation: ---
	Description: Wet, very dark brown silt with sand and organics		
	Remarks: System L		

Client:	Golder Associates
Project Name:	Marsh Bridge
Project Location:	Tremont/Southwest Harbor, ME
GTX #:	306503
Test Date:	6/8/2017
Tested By:	rlc
Checked By:	jsc
Boring ID:	BB-MAR-102
Sample ID:	R2
Depth, ft:	24.75-25.15
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: 33,030 psi

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

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
3300-12000	12,100,000	0.25
12000-20700	7,860,000	0.18
20700-29400	8,940,000	0.23

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:	Golder Associates	Test Date:	5/30/2017
Project Name:	Marsh Bridge	Tested By:	rlc
Project Location:	Tremont/Southwest Harbor, ME	Checked By:	jsc
GTX #:	306503		
Boring ID:	BB-MAR-102		
Sample ID:	R2		
Depth:	24.75-25.15 ft		
Visual Description:	See photographs		

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

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.52	4.52	4.52
Specimen Diameter, in:	1.99	2.00	2.00
Specimen Mass, g:	611.91		
Bulk Density, lb/ft^3	165	**Minimum Diameter Tolerence Met?**	YES
Length to Diameter Ratio:	2.3	**Length to Diameter Ratio Tolerance Met?**	YES

DEVIATION FROM STRAIGHTNESS (Procedure S1)

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

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

END FLATNESS AND PARALLELISM (Procedure FP1)

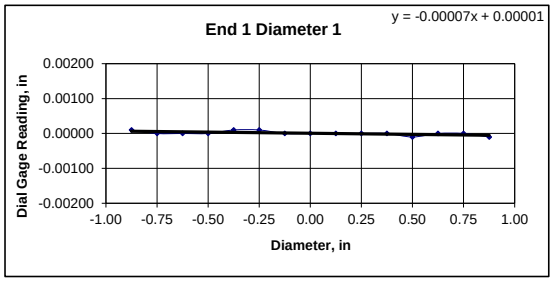
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00010	0.00000	0.00000	0.00000	0.00010	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	0.00000	0.00000	-0.00010
Diameter 2, in (rotated 90°)	0.00010	0.00010	0.00010	0.00000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

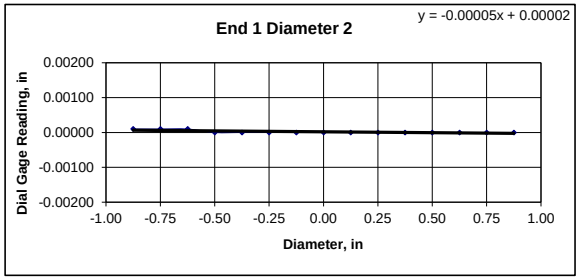
Difference between max and min readings, in:
0° = 0.00020 90° = 0.00010

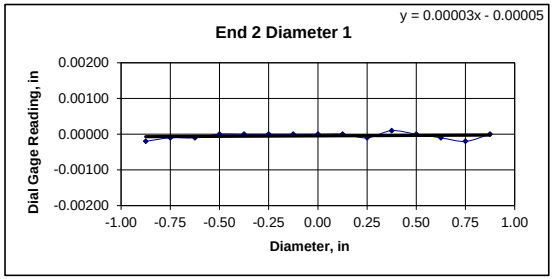
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	-0.00020	-0.00010	-0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	0.00010	0.00000	-0.00010	-0.00020	0.00000
Diameter 2, in (rotated 90°)	0.00040	0.00030	0.00020	0.00010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

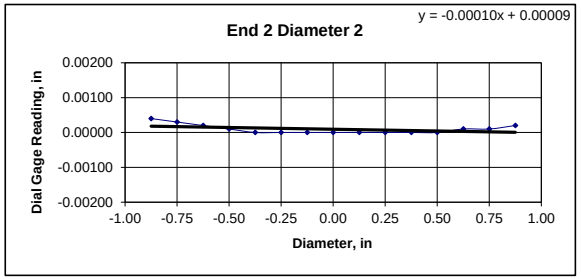
Difference between max and min readings, in:
0° = 0.0003 90° = 0.0004

Maximum difference must be < 0.0020 in. Difference = ± 0.00020
Flatness Tolerance Met? YES









DIAMETER 1

End 1:
Slope of Best Fit Line 0.00007
Angle of Best Fit Line: 0.00401

End 2:
Slope of Best Fit Line 0.00003
Angle of Best Fit Line: 0.00172

Maximum Angular Difference: 0.00229

Parallelism Tolerance Met? YES
Spherically Seated

DIAMETER 2

End 1:
Slope of Best Fit Line 0.00005
Angle of Best Fit Line: 0.00286

End 2:
Slope of Best Fit Line 0.00010
Angle of Best Fit Line: 0.00573

Maximum Angular Difference: 0.00286

Parallelism Tolerance Met? YES
Spherically Seated

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

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00020	1.995	0.00010	0.006	YES
Diameter 2, in (rotated 90°)	0.00010	1.995	0.00005	0.003	YES
END 2					
Diameter 1, in	0.00030	1.995	0.00015	0.009	YES
Diameter 2, in (rotated 90°)	0.00040	1.995	0.00020	0.011	YES

Maximum angle of departure must be ≤ 0.25°
Perpendicularity Tolerance Met? YES

Client:	Golder Associates
Project Name:	Marsh Bridge
Project Location:	Tremont/Southwest Harbor, ME
GTX #:	306503
Test Date:	6/8/2017
Tested By:	rlc
Checked By:	jsc
Boring ID:	BB-MAR-102
Sample ID:	R2
Depth, ft:	24.75-25.15



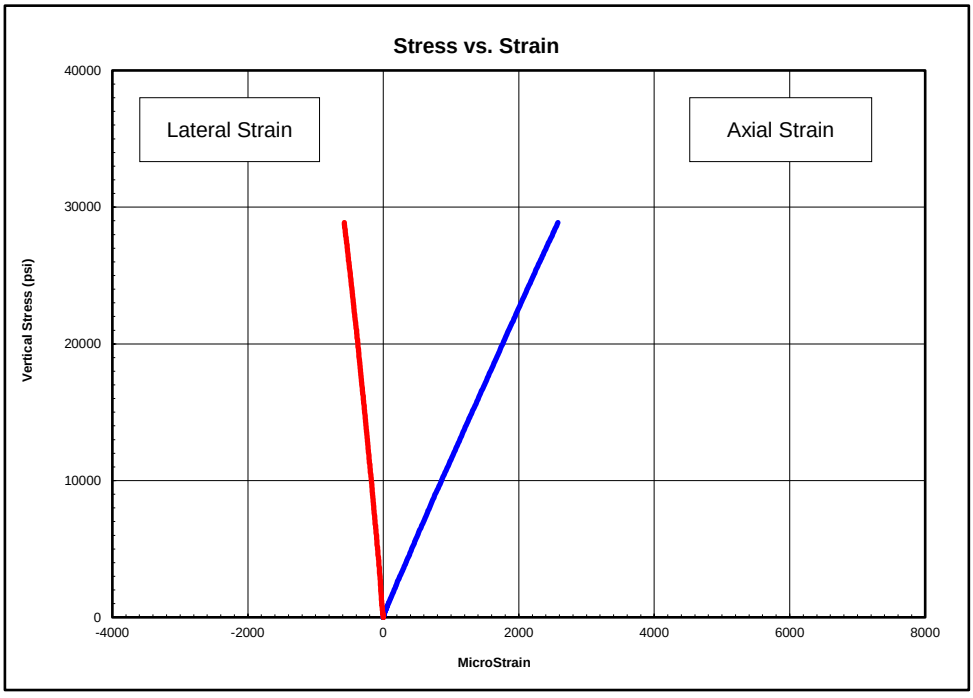
After cutting and grinding



After break

Client:	Golder Associates
Project Name:	Marsh Bridge
Project Location:	Tremont/Southwest Harbor, ME
GTX #:	306503
Test Date:	6/8/2017
Tested By:	rlc
Checked By:	jsc
Boring ID:	BB-MAR-103B
Sample ID:	R1
Depth, ft:	34.76-35.14
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: 28,876 psi

Stress Range, psi	Young's Modulus, psi	Poisson's Ratio
2900-10600	11,500,000	0.21
10600-18300	11,100,000	0.22
18300-26000	11,000,000	0.24

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:	Golder Associates	Test Date:	5/30/2017
Project Name:	Marsh Bridge	Tested By:	rlc
Project Location:	Tremont/Southwest Harbor, ME	Checked By:	jsc
GTX #:	306503		
Boring ID:	BB-MAR-103B		
Sample ID:	R1		
Depth:	34.76-35.14 ft		
Visual Description:	See photographs		

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

BULK DENSITY

	1	2	Average
Specimen Length, in:	4.35	4.35	4.35
Specimen Diameter, in:	1.99	1.99	1.99
Specimen Mass, g:	581.76		
Bulk Density, lb/ft^3	163	**Minimum Diameter Tolerence Met?**	YES
Length to Diameter Ratio:	2.2	**Length to Diameter Ratio Tolerance Met?**	YES

DEVIATION FROM STRAIGHTNESS (Procedure S1)

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

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

END FLATNESS AND PARALLELISM (Procedure FP1)

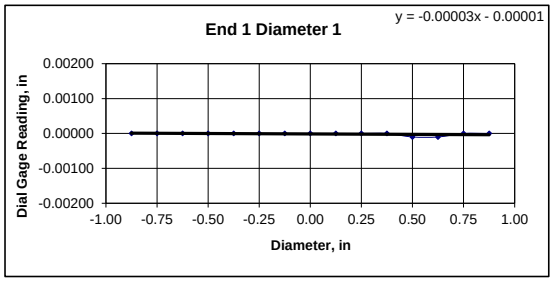
END 1	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	-0.00010	-0.00010	0.00000	0.00000
Diameter 2, in (rotated 90°)	-0.00030	-0.00020	-0.00010	0.00000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

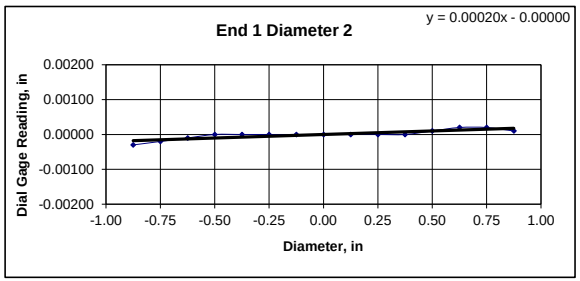
Difference between max and min readings, in:
0° = 0.00010 90° = 0.00050

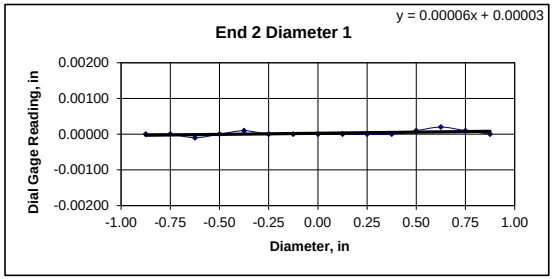
END 2	-0.875	-0.750	-0.625	-0.500	-0.375	-0.250	-0.125	0.000	0.125	0.250	0.375	0.500	0.625	0.750	0.875
Diameter 1, in	0.00000	0.00000	-0.00010	0.00000	0.00010	0.00000	0.00000	0.00000	0.00000	0.00000	0.00000	0.00010	0.00020	0.00010	0.00000
Diameter 2, in (rotated 90°)	-0.00030	-0.00020	-0.00010	0.00000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

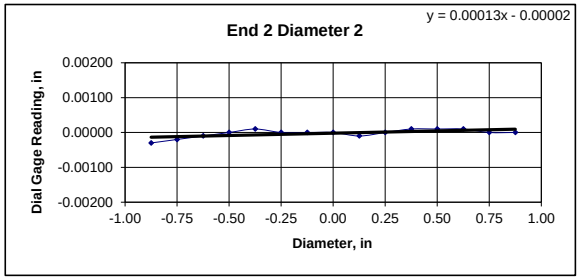
Difference between max and min readings, in:
0° = 0.0003 90° = 0.0004

Maximum difference must be < 0.0020 in. Difference = ± 0.00025
Flatness Tolerance Met? YES









DIAMETER 1

End 1:
Slope of Best Fit Line 0.00003
Angle of Best Fit Line: 0.00172

End 2:
Slope of Best Fit Line 0.00006
Angle of Best Fit Line: 0.00344

Maximum Angular Difference: 0.00172

Parallelism Tolerance Met? YES
Spherically Seated

DIAMETER 2

End 1:
Slope of Best Fit Line 0.00020
Angle of Best Fit Line: 0.01146

End 2:
Slope of Best Fit Line 0.00013
Angle of Best Fit Line: 0.00745

Maximum Angular Difference: 0.00401

Parallelism Tolerance Met? YES
Spherically Seated

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

END 1	Difference, Maximum and Minimum (in.)	Diameter (in.)	Slope	Angle°	Perpendicularity Tolerance Met?
Diameter 1, in	0.00010	1.990	0.00005	0.003	YES
Diameter 2, in (rotated 90°)	0.00050	1.990	0.00025	0.014	YES
END 2					
Diameter 1, in	0.00030	1.990	0.00015	0.009	YES
Diameter 2, in (rotated 90°)	0.00040	1.990	0.00020	0.012	YES

Maximum angle of departure must be ≤ 0.25°
Perpendicularity Tolerance Met? YES

Client:	Golder Associates
Project Name:	Marsh Bridge
Project Location:	Tremont/Southwest Harbor, ME
GTX #:	306503
Test Date:	6/8/2017
Tested By:	rlc
Checked By:	jsc
Boring ID:	BB-MAR-103B
Sample ID:	R1
Depth, ft:	34.76-35.14



After cutting and grinding



After break

APPENDIX D
GEOPHYSICAL SURVEY REPORT

HAGER-RICHTER
GEOSCIENCE, INC.

GEOPHYSICAL SURVEY
MARSH BRIDGE REPLACEMENT PROJECT
STATE ROUTE 102, TREMONT, MAINE

Prepared for:

Golder Associates, Inc.
670 North Commercial Street., Suite 103
Manchester, New Hampshire 03101

Prepared by:

Hager-Richter Geoscience, Inc.
8 Industrial Way - D10
Salem, New Hampshire 03079

File 17MH08
June, 2017

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HAGER-RICHTER
GEOSCIENCE,INC.

GEOPHYSICISTS FOR THE ENGINEERING COMMUNITY
8 INDUSTRIAL WAY - D10
SALEM, NEW HAMPSHIRE 03079-5820
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FAX (603) 893-8313

June 7, 2017
File 17MH08

Cameron Stuart
Staff Engineer
Golder Associates Inc.
670 North Commercial Street., Suite 103
Manchester, New Hampshire 03101

Tel: 603-668-0880
Fax: 603-668-1199
Email: Cameron_Stuart@golder.com

RE: Geophysical Survey
Marsh Bridge Replacement Project
State Route 102
Tremont, Maine

Dear Mr. Stuart:

In this letter, we report the results of a geophysical survey conducted by Hager-Richter Geoscience, Inc. (Hager-Richter) along a portion of State Route 102 in Tremont, Maine for Golder Associates, Inc. (Golder) in May, 2017. The geophysical survey was performed in support of a geotechnical investigation by Golder for the replacement of the Marsh Bridge carrying State Route 102 over Marshall Creek.

INTRODUCTION

The Site is a portion of State Route 102 (Tremont Road) in Tremont, Maine that includes Marsh Bridge. The bridge is located approximately 1,000 feet northeast of the junction of Route 102 and Harbor Drive. The general location of the Site is shown in Figure 1. The Marsh Bridge is an approximately 20-foot long concrete span built in the 1930's. The Marsh Bridge is a two-lane bridge that carries State Route 102 over Marshall Creek, a tidal creek that drains into Bass Harbor.

According to the 1931 construction plans for the March Bridge provided by Golder, the bridge rests on reinforced concrete abutments supported by piles. Construction details about the approaches were not included in information provided by Golder. The roadway is approximately 15 feet wide and is flanked by steel guardrails for the approach sections. Golder indicated that the bridge and approaches together comprise the area of interest (AOI) for the geophysical survey. Figure 2 is a site plan showing limits of the geophysical survey.

In support of a geotechnical investigation prior to replacement of the Marsh Bridge,

Golder requested a geophysical survey to determine the depth and configuration of bedrock in the vicinity of the existing bridge. Golder installed four borings (BB-MAR-101, BB-MAR-102, BB-MAR-103, and BB-MAR-104) in the roadway near the bridge. Depth of bedrock was determined by coring at depths of approximately 9.1 feet to 51.5 feet below ground surface. Preliminary logs for the borings provided by Golder are attached to this report in Appendix 1.

OBJECTIVE

The objective of the geophysical survey was to determine the depth and configuration of the bedrock surface under the bridge and approaches.

THE SURVEY

The geophysical survey was conducted using ground penetrating radar (GPR) and seismic refraction methods. Michael Howley, P.G., and Bryan Carnahan of Hager-Richter conducted the geophysical survey on May 4, 2017. The fieldwork was coordinated with Mr. Cameron Stuart of Golder. Mr. Taylor Mantey, also of Golder, was on site for the duration of the survey, specified the area of interest, and coordinated traffic control services. Data analysis and interpretation were completed at the Hager-Richter offices. Original data and field notes will be retained in the Hager-Richter files for a minimum of three years.

GPR data were acquired along traverses spaced two feet apart oriented parallel to the travel lanes, and along traverses spaced 50 feet apart oriented perpendicular to the travel lanes. The area of interest for the GPR survey extended approximately 200 feet southwest and 220 feet northeast of the Marsh Bridge along the paved roadway. In addition, seismic refraction profiling was conducted along two 235-foot long seismic lines located both southwest and northeast of the Marsh Bridge along the paved roadway. Golder provided site plans showing site features and surface topography. The locations of borings installed by Golder were recorded using a differential corrected GPS. Figure 2 is a modified site plan showing the limits of the area surveyed with GPR, the location of the seismic refraction lines, and other site features. Photo 1 shows the seismic refraction equipment setup on the northeast approach.



Photo 1. View looking southwest at the Marsh Bridge. Seismic refraction geophones were installed at 5 foot intervals.

EQUIPMENT

GPR. The GPR survey was conducted using a Geophysical Survey Systems, Inc. SIR 4000 digital radar system using a 350 MHz hyper-stacking antenna with 150 ns[1] time window. The system includes a survey wheel that triggers the recording of data at fixed intervals, thereby increasing the accuracy of the locations of features detected along the survey lines.

GPR uses a high-frequency electromagnetic pulse (referred to herein as "radar signal") transmitted from a radar antenna to probe the subsurface. The transmitted radar signals are reflected from subsurface interfaces of materials with contrasting electrical properties. Travel times of the radar signal can be converted to approximate depth below the surface by correlation with targets of known depths and by a curve matching routine. We monitor the acquisition of GPR data in the field and record the GPR data digitally for subsequent processing. Interpretation of the records is based on the nature and intensity of the reflected signals and on the resulting patterns.

Data from the GPR survey were processed using RADAN 7.4 GPR processing software from Geophysical Survey Systems, Inc. We reviewed profile images and created plan view time slice maps of the GPR data.

Seismic Refraction. For the seismic refraction surveys, we used two 24-channel Geometrics Geode digital seismographs, coupled to 48 geophones spaced 5 feet apart. Five shot points were used per seismic spread - one located internal to the spread, one at each end of the spread, and two offset shots located in-line but outside of the spread of geophones. The seismic source was a sledgehammer striking a metal plate. The seismic refraction data were processed using IXRefraX™ software licensed by Interpex.

LIMITATIONS OF THE METHODS

HAGER-RICHTER GEOSCIENCE, INC. MAKES NO GUARANTEE THAT THE DEPTH OF BEDROCK WAS ACCURATELY DETERMINED IN THIS SURVEY. HAGER-RICHTER GEOSCIENCE, INC. IS NOT RESPONSIBLE FOR DETERMINING THE DEPTH OF BEDROCK WHERE THE INTERFACE CANNOT BE DETECTED BECAUSE OF SITE CONDITIONS. THE BEDROCK DEPTHS DETERMINED SHOULD NOT BE USED FOR CONTRACT BEDROCK REMOVAL QUANTITIES.

[1]ns, abbreviation for nanosecond, 1/1,000,000,000 second. Light and the GPR signal require about 1 ns to travel 1 ft in air. The GPR signal requires about 3.5 ns to travel 1 ft in unsaturated sandy soil.

GPR. There are limitations of the GPR technique: (1) surface conditions, (2) electrical conductivity and thickness of the subsurface layers, (3) electrical properties of the target(s), and (4) spacing of the traverses. Of these restrictions, only the last is controllable by us in most cases.

The condition of the survey surface can affect the quality of the GPR data and the depth of penetration of the GPR signal. For exterior sites, a surface covered with obstacles such as automobiles, dumpsters, thick leaf debris, materials piles, etc. limit the survey access. Similarly, for interior sites, a surface covered with obstacles such as desks, benches, laboratory equipment, etc. also limit access. Some floor coverings may limit the coupling of the GPR antenna with the subsurface.

The electrical conductivity of the subsurface determines the attenuation of the GPR signals, and thereby limits the maximum depth of exploration. The GPR signal does not penetrate clay-rich soils or soils contaminated with road salt. In some cases, the GPR signal may not penetrate below concrete pavement, and some asphalts are electrically conducting.

A strong contrast in the electrical conductivities of the ground and the target (for examples, UST, pipe, void, dry well, drum, contaminant plume) is required to obtain a reflection of the GPR signal. If the contrast is too small, then the reflection may be too weak to recognize, and the target can be missed.

Spacing of the traverses is limited by access at many sites, but where flexibility of traverse spacing is possible, the spacing is adjusted on the basis of the size of the target.

Seismic Refraction. Like all geophysical methods, the seismic refraction method is based on the assumption that the local geology is uncomplicated. In particular, the seismic refraction method assumes that interfaces between geologic materials correlate with sharp increases in seismic velocity and that the interfaces between geologic units are relatively flat-lying. The method is not very sensitive to lateral variations within layers, and relatively subtle features such as fracture zones within bedrock are generally difficult to detect unless there is a topographic expression of the feature. The accuracy of the method is degraded in areas with strong topographic relief and/or where the interfaces have apparent dips greater than about 20°. *In general, the standard error of depths determined is about 10% or 2 feet, whichever is greater.*

Where two materials do not exhibit contrasting velocities, or where velocities gradually increase with depth, a clear refracted signal is not generated, and the seismic refraction method cannot be used to distinguish the two materials. In some cases, the "geophysical contact" between materials with contrasting velocities does not correlate exactly with the "geologic contact." For example, where a highly weathered bedrock is overlain by a dense material such as

till, the velocity range of the weathered bedrock might overlap or approach the velocity range of the till, and the two materials cannot be distinguished seismically.

RESULTS

The geophysical survey of the roadway in the vicinity of the Marsh Bridge consisted of a ground penetrating radar (GPR) survey across the area of interest and two seismic refraction profiles, one on each of the approaches. The GPR survey was conducted in an area measuring approximately 30-feet by 550-feet with traverses spaced 2 feet apart oriented parallel to the travel lanes, and along traverses spaced 50 feet apart oriented perpendicular to the travel lanes. The seismic refraction profile consisted of two 235-foot long seismic refraction survey lines. The area surveyed with GPR and the locations of the seismic refraction lines are shown in Figure 2. The interpretation of the GPR data are shown in Figure 3, and the results of the seismic refraction surveys are shown in profile form in Figure 4, and are listed in Tables 1 and 2.

GPR. Apparent GPR signal penetration was generally fair to good across the area of interest, with two-way traveltime reflections received for 60 to 80 ns of the 150 ns records recorded. Based on site-specific time-to-depth conversions for the GPR signal at the Site, the GPR signal penetration is estimated to have been approximately 8 to 10 feet.

GPR reflections consistent with those expected for the top of bedrock are not evident in the GPR records acquired on the approaches to the Marsh Bridge. The depth of bedrock based on boring logs provided by Golder ranges from 9.1 feet to 51.5 feet below ground surface. The bedrock depths indicated by borings in most of the area of interest are either greater than the effective penetration of the GPR signal, or the contrast between the bedrock surface and the overlying soils is not sufficient to generate a detectable GPR reflection.

GPR reflections consistent with three shallow possible buried structures are present in the GPR records, and the locations of such are shown on Figure 3. Two of the possible structures are located near the abutments of the existing bridge, approximately where existing timber cribbing is shown on plans provided by Golder. The third possible structure is located approximately 110 feet southwest of the Marsh Bridge.

Seismic Refraction Survey. The seismic survey consisted of two 235-foot long seismic refraction lines. The locations of the seismic lines are shown in Figure 2. The results of the seismic survey are shown in profile form in Figure 4 and are listed in Table 1.

The quality of the seismic refraction data ranges from good to excellent. A measure of the accuracy of the data can be obtained by comparing the bedrock depths determined seismically with depths reported from nearby borings that encountered refusals on assumed bedrock, or cored

bedrock. For the present survey, four borings (BB-MAR-101, BB-MAR-102, BB-MAR-103, and BB-MAR-104) were drilled in the roadway within 70 feet of the bridge, and within 12 feet of the two seismic refraction lines. The four borings encountered bedrock at depths ranging from approximately 9.1 feet to 51.5 feet below the road surface. The locations of the borings along the seismic line and depth of bedrock determined from each boring are shown on the seismic profiles in Figure 4.

Materials with two distinct velocity ranges were detected southwest of the Marsh Bridge between Maine DOT stations 13+30 and 15+65. The upper material exhibits a velocity ranging from 2,450 to 3,550 feet per second (fps) and is interpreted to consist of both unsaturated and saturated soils. The lower material exhibits a velocity range of 13,900 and 16,000 fps and is interpreted to be competent bedrock.

Materials with three distinct velocity ranges were detected northeast of the Marsh Bridge between Maine DOT stations 15+90 and 18+25. The upper material exhibits a velocity ranging from 1,400 to 1,500 feet per second (fps) and is interpreted to consist of unsaturated soils. The intermediate material exhibits a velocity ranging from 4,650 to 4,700 feet per second (fps) and is interpreted to consist of saturated soils. The lower material exhibits a velocity range of 14,200 and 15,900 fps and is interpreted to be competent bedrock.

Where the top of bedrock is highly fractured and/or deeply weathered, it might exhibit lower velocities that cannot be detected as a distinct layer on the basis of the seismic refraction data. Thus, the top of rock determined on the basis of seismic refraction data generally is the top of competent bedrock, which might be located somewhat below the geologic contact between the overburden and bedrock.

Comparison of seismic refraction data with results from the GPR survey are not possible due to the lack of GPR reflections from bedrock in the area surveyed, as stated above. Based on the good quality of the seismic data for this project and on the results from other similar seismic refraction surveys, we estimate the accuracy (standard deviation) of the apparent depths of competent bedrock determined by the seismic refraction survey in most locations to be about ± 10% of the depth of bedrock, or ± 2 feet), whichever is greater.

The seismic profiles shown in Figure 4 do show that bedrock is deeper northeast of the existing bridge, which is consistent with the boring logs for BB-MAR-103 and BB-MAR-104 indicating bedrock depths of 32.4 and 51.5 feet, respectively. The depth of competent bedrock along the seismic lines varies between about 9 and 52 feet below ground surface, for a total relief of 43 feet. The elevation of the road at discrete points along the seismic profiles was determined from the plans supplied by Golder. Bedrock depths and elevations at station spacings of 5 feet along the seismic lines are listed in Tables 1 and 2.

The profile shown in Figure 4 represents a non-unique model for bedrock depths, and the depths of competent bedrock at any particular location between actual data points may differ from that shown. Bedrock depths based on additional data, such as additional borings or seismic data, may differ significantly from those shown on the plates. **The bedrock model shown as profiles or listed as tabular data should not be used for contract bedrock removal quantities.**

CONCLUSIONS

Based on the results of the geophysical survey conducted by Hager-Richter Geoscience, Inc. along a portion of Route 102 at the Marsh Bridge in Tremont, Maine in May, 2017, we conclude that:

- Depth of bedrock below the surveyed portion of the roadway varies between about 9 and 52 feet and is deepest northeast of the Marsh Bridge
- Three shallow possible buried structures, two of which are located near the indicated positions of timber cribbing installed during bridge construction, were detected on the basis of GPR records

LIMITATIONS

This letter report was prepared for the exclusive use of Golder Associates, Inc. (Client). No other party shall be entitled to rely on this Report or any information, documents, records, data, interpretations, advice or opinions given to Client by Hager-Richter Geoscience, Inc. (H-R) in the performance of its work. The Report relates solely to the specific project for which H-R has been retained and shall not be used or relied upon by Client or any third party for any variation or extension of this project, any other project or any other purpose without the express written permission of H-R. Any unpermitted use by Client or any third party shall be at Client's or such third party's own risk and without any liability to H-R.

H-R has used reasonable care, skill, competence and judgment in the performance of' its services for this project consistent with professional standards for those providing similar services at the same time, in the same locale, and under like circumstances. Unless otherwise stated, the work performed by H-R should be understood to be exploratory and interpretational in character and any results, findings or recommendations contained in this Report or resulting from the work proposed may include decisions which are judgmental in nature and not necessarily based solely on pure science or engineering. It should be noted that our conclusions might be modified if subsurface conditions were better delineated with additional subsurface exploration including, but not limited to, test pits, soil borings with collection of soil and water samples, and laboratory testing.

Except as expressly provided in this limitations section, H-R makes no other representation or warranty of any kind whatsoever, oral or written, expressed or implied; and all implied warranties of merchantability and fitness for a particular purpose, are hereby disclaimed.

If you have any questions or comments on this letter report, please contact us at your convenience. It has been a pleasure to work with you on this project. We look forward to working with you again in the future.

Sincerely yours,
HAGER-RICHTER GEOSCIENCE, INC.

Michael W. Howley, P.G.
Geophysicist

Gene Simmons, Ph.D., P.G.
Vice President & Principal Geophysicist

Attachments: Table 1 - Depth of Bedrock from Seismic Refraction Southwest of Existing Bridge
Table 2 - Depth of Bedrock from Seismic Refraction Northeast of Existing Bridge
Figure 1 - General Site Location
Figure 2 - Site Plan
Figure 3 - GPR Survey
Figure 4 - Seismic Refraction Lines 1 & 2
Appendix 1 - Draft Boring Logs

Table 1
Depth of Bedrock from Seismic Refraction
Southwest of Existing Bridge
Marsh Bridge Replacement Project
State Route 102, Tremont, Maine

Profile Distance (feet)	Approximate Maine DOT Stationing (feet)	Maine State Plane, Zone 1802 NAD83, US Survey Feet		Approximate Bedrock Depth (feet)
		East (feet)	North (feet)	
0	13+30	2174321.6	153368.2	13.9
5	13+35	2174326.2	153370.1	13.9
10	13+40	2174330.8	153372.1	13.0
15	13+45	2174335.4	153374.0	12.3
20	13+50	2174340.1	153375.9	12.0
25	13+55	2174344.7	153377.9	11.2
30	13+60	2174349.3	153379.8	10.7
35	13+65	2174353.9	153381.7	10.7
40	13+70	2174358.5	153383.7	10.6
45	13+75	2174363.1	153385.6	10.5
50	13+80	2174367.7	153387.5	10.9
55	13+85	2174372.3	153389.5	11.2
60	13+90	2174377.0	153391.4	11.2
65	13+95	2174381.6	153393.3	11.0
70	14+00	2174386.2	153395.2	10.6
75	14+05	2174390.8	153397.2	9.9
80	14+10	2174395.4	153399.1	9.4
85	14+15	2174400.0	153401.0	8.5
90	14+20	2174404.6	153403.0	9.1
95	14+25	2174409.2	153404.9	10.1
100	14+30	2174413.9	153406.8	9.5
105	14+35	2174418.5	153408.8	9.8
110	14+40	2174423.1	153410.7	9.3
115	14+45	2174427.7	153412.6	7.9
120	14+50	2174432.3	153414.6	7.4
125	14+55	2174436.9	153416.5	8.2
130	14+60	2174441.5	153418.4	9.2
135	14+65	2174446.1	153420.4	9.8
140	14+70	2174450.8	153422.3	10.3
145	14+75	2174455.4	153424.2	10.6
150	14+80	2174460.0	153426.2	11.6

Table 1 continued

Profile Distance (feet)	Approximate Maine DOT Stationing (feet)	Maine State Plane, Zone 1802 NAD83, US Survey Feet		Approximate Bedrock Depth (feet)
		East (feet)	North (feet)	
155	14+85	2174464.6	153428.1	11.9
160	14+90	2174469.2	153430.0	11.6
165	14+95	2174473.8	153432.0	11.9
170	15+00	2174478.4	153433.9	12.5
175	15+05	2174483.0	153435.8	13.0
180	15+10	2174487.7	153437.7	13.3
185	15+15	2174492.3	153439.7	14.1
190	15+20	2174496.9	153441.6	13.9
195	15+25	2174501.5	153443.5	13.8
200	15+30	2174506.1	153445.5	14.6
205	15+35	2174510.7	153447.4	19.1
210	15+40	2174515.3	153449.3	19.1
215	15+45	2174519.9	153451.3	19.5
220	15+50	2174524.6	153453.2	19.9
225	15+55	2174529.2	153455.1	21.7
230	15+60	2174533.8	153457.1	21.7
235	15+65	2174538.4	153459.0	21.7

Estimated standard deviation of depth of interfaces for seismic lines is normally taken as 10% or 2 feet, whichever is greater. Depths and elevations of bedrock determined here are for competent bedrock. Heavily weathered or highly fractured bedrock may occur at shallower depths. East and North coordinates for the seismic lines are given in feet in Maine State Plane West 1802, NAD1983, US Survey Feet, and were determined using differentially corrected GPS positions acquired at the time of the survey.

Table 2
Depth of Bedrock from Seismic Refraction
Northeast of Existing Bridge
Marsh Bridge Replacement Project
State Route 102, Tremont, Maine

Profile Distance (feet)	Approximate Maine DOT Stationing (feet)	Maine State Plane, Zone 1802 NAD83, US Survey Feet		Approximate Bedrock Depth (feet)
		East (feet)	North (feet)	
0	15+90	2174564.0	153463.1	34.1
5	15+95	2174568.6	153465.1	36.0
10	16+00	2174573.2	153467.1	37.4
15	16+05	2174577.8	153469.0	41.3
20	16+10	2174582.4	153471.0	42.6
25	16+15	2174587.0	153473.0	45.7
30	16+20	2174591.6	153475.0	47.0
35	16+25	2174596.2	153476.9	48.0
40	16+30	2174600.7	153478.9	50.1
45	16+35	2174605.3	153480.9	52.0
50	16+40	2174609.9	153482.9	50.9
55	16+45	2174614.5	153484.8	45.5
60	16+50	2174619.1	153486.8	41.0
65	16+55	2174623.7	153488.8	39.4
70	16+60	2174628.3	153490.8	37.9
75	16+65	2174632.9	153492.7	36.4
80	16+70	2174637.5	153494.7	35.4
85	16+75	2174642.1	153496.7	34.8
90	16+80	2174646.7	153498.7	33.9
95	16+85	2174651.3	153500.7	33.1
100	16+90	2174655.9	153502.6	32.4
105	16+95	2174660.5	153504.6	31.0
110	17+00	2174665.1	153506.6	31.6
115	17+05	2174669.7	153508.6	32.0
120	17+10	2174674.2	153510.5	31.4
125	17+15	2174678.8	153512.5	31.1
130	17+20	2174683.4	153514.5	29.4
135	17+25	2174688.0	153516.5	29.2
140	17+30	2174692.6	153518.4	27.4
145	17+35	2174697.2	153520.4	27.5
150	17+40	2174701.8	153522.4	26.8

Table 2 continued

Profile Distance (feet)	Approximate Maine DOT Stationing (feet)	Maine State Plane, Zone 1802 NAD83, US Survey Feet		Approximate Bedrock Depth (feet)
		East (feet)	North (feet)	
155	17+45	2174706.4	153524.4	28.0
160	17+50	2174711.0	153526.4	25.2
165	17+55	2174715.6	153528.3	25.4
170	17+60	2174720.2	153530.3	25.1
175	17+65	2174724.8	153532.3	25.5
180	17+70	2174729.4	153534.3	25.4
185	17+75	2174734.0	153536.2	26.3
190	17+80	2174738.6	153538.2	25.8
195	17+85	2174743.2	153540.2	25.0
200	17+90	2174747.7	153542.2	26.1
205	17+95	2174752.3	153544.1	26.2
210	18+00	2174756.9	153546.1	25.6
215	18+05	2174761.5	153548.1	27.6
220	18+10	2174766.1	153550.1	27.3
225	18+15	2174770.7	153552.0	27.3
230	18+20	2174775.3	153554.0	27.3
235	18+25	2174779.9	153556.0	27.3

Estimated standard deviation of depth of interfaces for seismic lines is normally taken as 10% or 2 feet, whichever is greater. Depths and elevations of bedrock determined here are for competent bedrock. Heavily weathered or highly fractured bedrock may occur at shallower depths. East and North coordinates for the seismic lines are given in feet in Maine State Plane West 1802, NAD1983, US Survey Feet, and were determined using differentially corrected GPS positions acquired at the time of the survey.



N

APPROXIMATE SCALE (feet)

0 1000 2000

NOTE:

Modified from Google Earth Pro aerial photograph.

ME

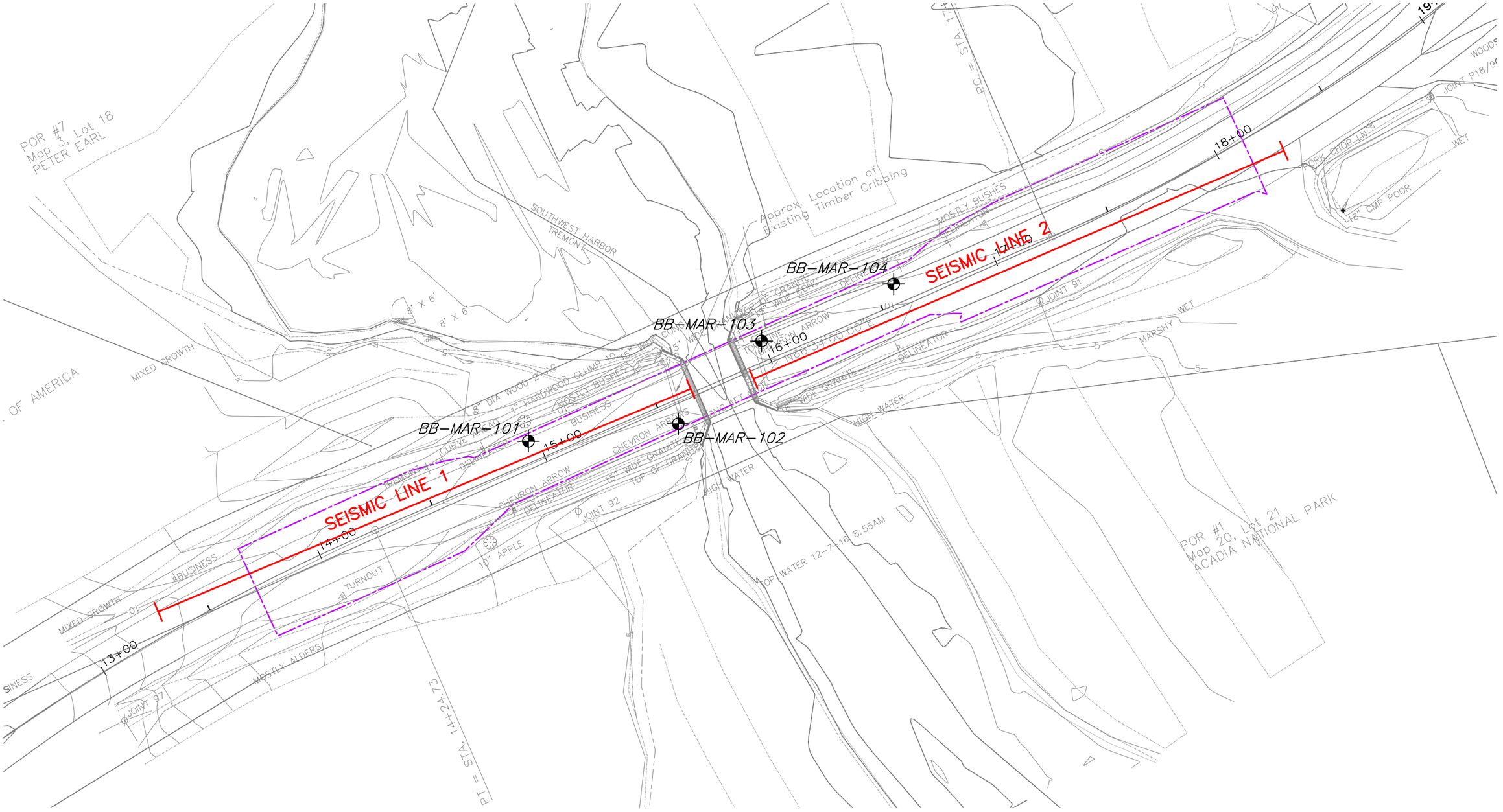
LOCATION

Figure 1
General Site Location
Marsh Bridge Replacement Project
State Route 102, Tremont, Maine

File 17MH08 | June, 2017

HAGER–RICHTER

Salem, NH | Fords, NJ



NOTE:

Modified from site plan provided by Golder Associates, Inc., identified as 001_GeneralPlan1.dwg.

SCALE (feet)

0 40 80

LEGEND

SEISMIC LINE

APPROXIMATE LIMITS OF GPR SURVEY AREA

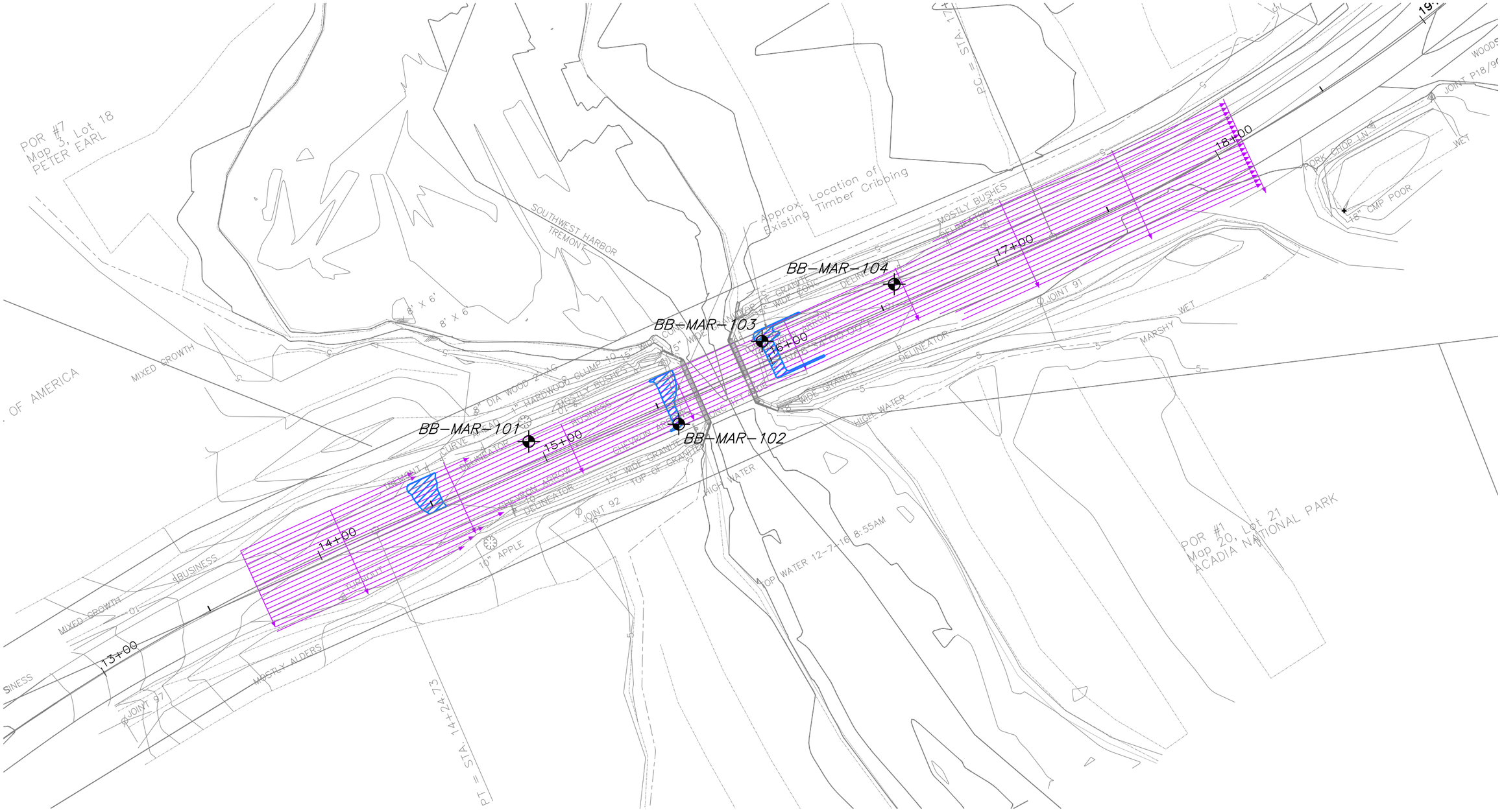
BORING

N

Figure 2
Site Plan
Marsh Bridge Replacement Project
State Route 102, Tremont, Maine

File 17MH08 | June, 2017

HAGER-RICHTER
Salem, NH | Fords, NJ



LEGEND

GPR TRAVERSE

POSSIBLE BURIED STRUCTURE

BORING

N

SCALE (feet)

0 40 80

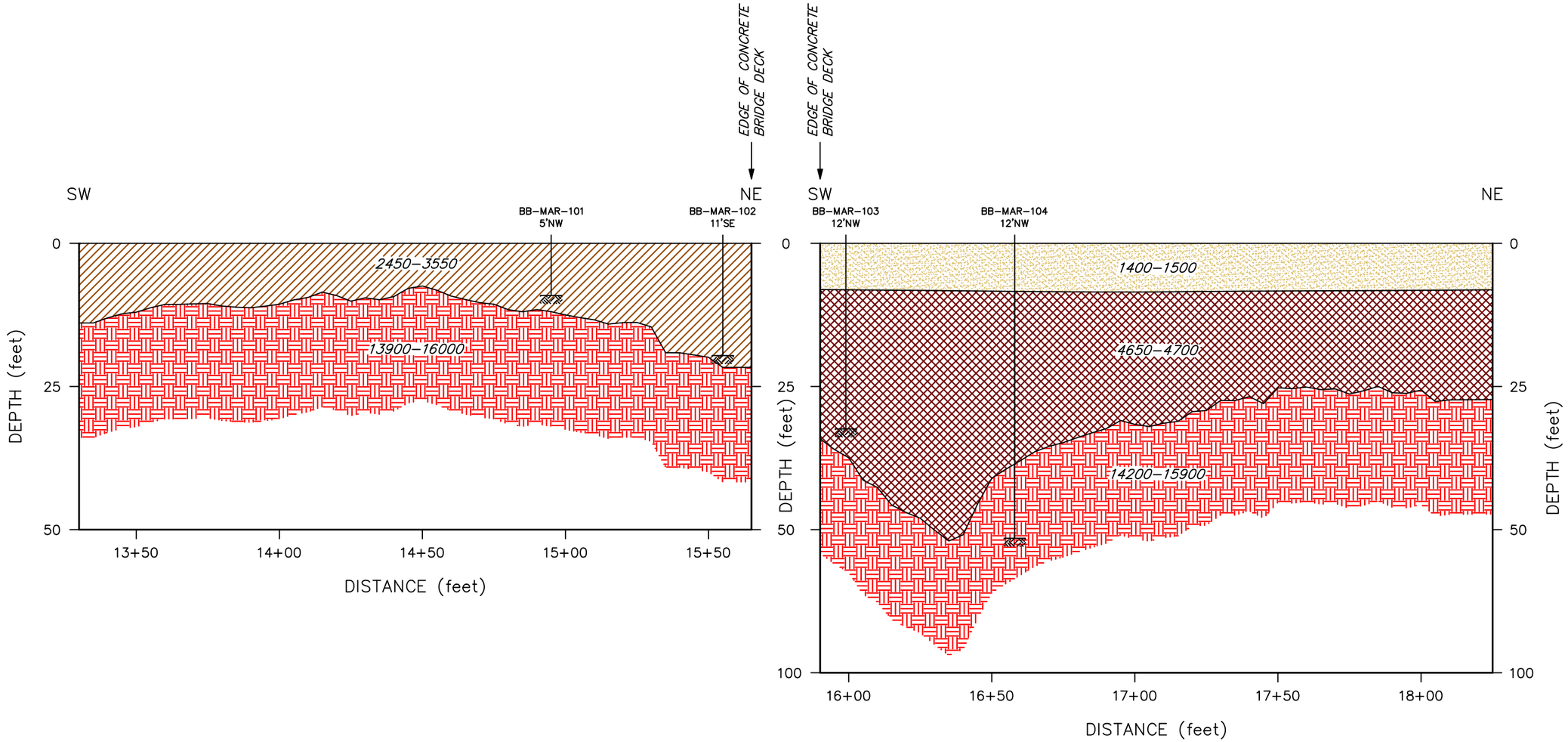
NOTE:

Modified from site plan provided by Golder Associates, Inc., identified as 001_GeneralPlan1.dwg.

Figure 3
GPR Survey
Marsh Bridge Replacement Project
State Route 102, Tremont, Maine

File 17MH08 | June, 2017

HAGER-RICHTER
Salem, NH | Fords, NJ



LEGEND

- Unsaturated soils
- Unsaturated/ saturated soils
- Saturated soils
- Competent bedrock
- 13900–16000 — Velocity (fps)
- BB–MAR–101 5'NW — Boring with identification, distance from traverse, and depth of bedrock based on logs provided by Golder Associates, Inc.

HORIZONTAL SCALE (feet)

0 40 80

Vertical Exaggeration = 2X

NOTES:

1. Estimated standard deviation of depth of bedrock is ±10% or 2 feet, whichever is greater.
2. The depths determined for bedrock are depths of competent rock; weathered and/or fractured bedrock might occur at shallower depths.
3. Data were analyzed using the Generalized Reciprocal Method.

Figure 4
Seismic Lines 1 & 2
Marsh Bridge Replacement Project
Tremont, Maine

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APPENDIX 1

Draft Boring Logs

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Marsh Bridge
Location: Termont / Southwest Harbour, ME

Boring No.: BB-MAR-103 A/B
WIN: _____

Driller:	NEBC	**Elevation (ft.)**		**Auger ID/OD:**	4"
Operator:	Mike Porter	**Datum:**		**Sampler:**	
Logged By:	TRM	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140 lbs. / 30"
Date Start/Finish:	5/8/17 (9:10AM) / 5/9/17 (11:30PM)	**Drilling Method:**	SSA / D+W	**Core Barrel:**	
Boring Location:		**Casing ID/OD:**		**Water Level*:**	
Hammer Efficiency Factor:		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0	1D	24/14.4	0.50 - 2.50	35/24/16/20	40	0	SSA	-0.50		6" Asphalt	
							5			0.50	
	2D	24/6	2.50 - 4.50	20/14/16/13	30	0	2			Top ??: Bottom 0.4': Brown, damp, medium dense to dense, fine to coarse SAND, little gravel (Fill). Brown, damp, medium dense to dense, fine to coarse SAND, some silt, trace gravel	
							3				
	3D	24/8.4	4.50 - 6.50	9/4/4/3	8	0	21				
5							24			Brown, wet, loose, fine to coarse silty SAND, some gravel.	
							D W				
	4D	22/6	6.50 - 8.33	2/4/12/(50/4")	16	0	15			Brown, wet, dense, silty fine to coarse SAND, some gravel, trace organics (wood pieces)	
							14			Drill grinding, brown wash	
	5D	24/18	8.00 - 10.00	5/3/1/3	4	0	16			Alternating layers of gray, moist, soft, clayey SILT; brown, moist, soft, silty SAND; moist, soft, fine SAND trace organics Timber cribs filled with cuttings and gravel, cold patch.	
							9				
10	6D	24/14.4	10.00 - 12.00	1/1/2/1	3	0	7	-10.00		10.00 Upper 0.8': Grey, wet, SILT, little fine sand, trace gravel, strong odor Lower 0.4': Brown, damp, very soft, SILT, trace organics, strong odor	
							8				
	7D	24/20.4	12.00 - 14.00	WOH/3/3/2	6	0	7			Upper 0.9': Brown, moist, medium stiff, fine sandy SILT, trace organics. Lower 0.8': Grey, moist, medium stiff, clayey SAND, trace silt, trace organics, higher plasticity than above.	
							12				
	8D	24/4.8	14.00 - 16.00	4/4/6/6	10	0	12			Grey, moist, stiff, sandy CLAY, some gravel, trace silt, trace organics. TV: 0.14 kg/cm^2	
15							21				
	9D	24/18	16.00 - 18.00	4/2/3/2	5	0	19			Grey, damp, medium stiff, silty CLAY, trace fine sand, medium plasticity, large piece of gravel. PP: 1.75 TSF; TV: 0.35 kg/cm^2	
	MV1			Could Not Push			23			Sand seam at 17.9'	
	10D	24/24	18.00 - 20.00	WOH			16			Obstruction at 18', failed 65 x 130 vane test. Upper 1.5': Grey, moist, very soft, sandy SILT, some clay.	
							9			Lower 0.5': Black seam, same material as above. PP: 0	
20	11D	24/12	20.00 - 22.00	1/WOH			8			Drill rig grinding, wash turned black	
							17			grey, damp, very soft, silty CLAY. PP: 0; TV: 0.1 kg/cm^2	
	12D	24/24	22.00 - 24.00	WOH			13			Grey with dark staining, damp, very soft, silty CLAY, sticky, will not roll out. PP: 0; TV: 0.05 kg/cm^2	
							12				
							10				
25	MV2	24/24	25.00 - 27.00	Could Not Push			9			Failed 65x130 vane. Felt like gravel obstucting; soft clay stuck on vane.	
	13D			WOR			15			Grey with dark staining, damp, very soft, silty CLAY, sticky, will not roll out. PP: 0; TV: 0.05 kg/cm^2	
	14D	24/24	27.00 - 29.00	WOR			12			Grey with dark staining, damp, very soft, silty CLAY, sticky, will not roll out. PP: 0; TV: 0.05 kg/cm^2	
							9			Gritty wash.	
30							7				

DRAFT

Remarks:

1. Started with safety winch drive hammer. At 103B started with rope and cathead, drive and wash.
2. Auger (SSA) used for first 5' of both 103A and 103B.
3. Water levels:
   - 7.9' b.g.s. at end of day 5/8/17; 5.3' b.g.s. at 9:20 AM 5/9/17; 8.8' b.g.s. at 12:00 5/9/17.
4. Hole collapsed back to 33.3' when casing removed. 5. Borhole filled with cuttings, 4 bags of gravel, 1.5 bags of sand, and 0.5 bags of coldpatch.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Marsh Bridge
Location: Termont / Southwest Harbour, ME

Boring No.: BB-MAR-103 A/B
WIN: _____

Driller:	NEBC	**Elevation (ft.)**		**Auger ID/OD:**	4"
Operator:	Mike Porter	**Datum:**		**Sampler:**	
Logged By:	TRM	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140 lbs. / 30"
Date Start/Finish:	5/8/17 (9:10AM) / 5/9/17 (11:30PM)	**Drilling Method:**	SSA / D+W	**Core Barrel:**	
Boring Location:		**Casing ID/OD:**		**Water Level*:**	
Hammer Efficiency Factor:		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☐		

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$S_{u(lab)}$ = Lab Vane Undrained Shear Strength (psf)
q_p = Unconfined Compressive Strength (ksf)
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Hammer Efficiency Factor = Rig Specific Annual Calibration Value
N_{60} = SPT N-uncorrected Corrected for Hammer Efficiency
N_{60} = (Hammer Efficiency Factor/60%)*N-uncorrected
T_v = Pocket Torvane Shear Strength (psf)
WC = Water Content, percent
LL = Liquid Limit
PL = Plastic Limit
PI = Plasticity Index
G = Grain Size Analysis
C = Consolidation Test

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
30	15D	24/2.4	30.00 - 32.00	WOR/14/11/8	25	0	22	-32.00		grey with dark staining, wet, very stiff, silty CLAY. Blow counts do not represent sample density / consistency. Rocks and gravel in wash from 30' to 32'.	
	16D		32.00 - 32.33 ~~32.40 - 37.40~~				21 22	-32.40		32.00 3" Spoon: Grey, wet, silty SAND, some clay, some black sub-angular gravel with rock fragments.	
35										32.40 Top of bedrock at 32.4' based on chattering/ grinding & casing refusal. Run 1: Pink granite with black intrusions, 2" - 15" spacing, dipping at about 45 degrees. R1 Core Times (min:sec): 32.4' - 33.4': (4:12) 33.4' - 34.4': (2:00) 34.4' - 35.4': (1:45) 35.4' - 36.4': (1:42) 36.4' - 37.4': (1:50)	
			37.40 - 42.40					-37.40		37.40 **Bottom of Exploration at 37.40 feet below ground surface.** Run 2: PINK GRANITE with black intrusions dipping 20 to 45 degrees, increasing grain size with depth, 4" - 14" spacing, vertical fractures. R2 Core Times (min:sec): 37.4' - 38.4': (1:24) 38.4' - 39.4': (1:45) 39.4' - 40.4': (1:40) 40.4' - 41.4': (1:40) 41.4' - 42.4': (1:15)	
40											
45											
50											
55											
60											

DRAFT

Remarks:

1. Started with safety winch drive hammer. At 103B started with rope and cathead, drive and wash.
2. Auger (SSA) used for first 5' of both 103A and 103B.
3. Water levels:
   - 7.9' b.g.s. at end of day 5/8/17; 5.3' b.g.s. at 9:20 AM 5/9/17; 8.8' b.g.s. at 12:00 5/9/17.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Marsh Bridge
Location: Termont / Southwest Harbour, ME

Boring No.: BB-MAR-104
WIN:

Driller:	NEBC	**Elevation (ft.)**		**Auger ID/OD:**	
Operator:	Mike Porter	**Datum:**		**Sampler:**	
Logged By:	TRM	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140 lbs. / 30"
Date Start/Finish:	5/9/17 (12:40) / 5/10/17 (4:15 PM)	**Drilling Method:**	SSA / D+W	**Core Barrel:**	
Boring Location:		**Casing ID/OD:**		**Water Level*:**	
Hammer Efficiency Factor:		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA	-0.75		9" Asphalt 0.75	
	1D	24/13.2	1.00 - 3.00	8/11/9/9	20	0				Brown, damp, medium dense, fine to coarse SAND, some gravel, some silt (FILL).	
	2D	24/13.2	3.00 - 5.00	8/7/3/5	10	0				Brown, moist, loose, fine to coarse SAND, some gravel, some clay, little silt (FILL).	
5	3D	24/8.4	5.00 - 7.00	4/6/2/1	8	0	D+W			Top 0.2': Grey, moist, medium stiff to stiff, clayey fine SAND, little gravel, trace silt. Bottom 0.5': Brown/grey, wet, loose, fine to coarse SAND, some gravel.	
							20				
	4D	24/24	7.00 - 9.00	1/2/4/11	6	0	8			Top 1.5': Grey, moist, loose, fine to coarse SAND, some clay, trace silt.	
							13	-8.00		Bottom 0.5': Grey, damp, medium stiff, silty CLAY, some fine sand and organics (fibrous material, wood chunks)	
							32			Casing dropped 0.5' during wash from 5' to 10'. wash turned grey at about 7', large gravel or rock fragments in wash.	
10	5D	24/18	10.50 - 12.50	1/1/2/1	3	0	22			8.00 Brown, damp, soft, organic SILT, strong odor; PEAT, organics	
							9				
	6D	24/24	12.50 - 14.50	5/4/5/4	9	0	8	-12.50		12.50 Top 1.5': Brown-grey, damp, stiff, silty fine SAND, little clay, trace organics.	
							9			Bottom 0.5': Grey, damp, stiff, silty CLAY, some fine sand; very sticky.	
							14				
15	6D MV1	24/24	15.00 - 17.00	4/2/2/2 Could Not Push	4	0	13			PP: 1.0, 1.25, 1.0 TSF; TV: 0.3, 0.35 kg/cm$_2$ 3" Spoon: Grey, moist, soft to medium stiff, silty CLAY, trace organics, 0.5" woody/fibrous layer in 3" spoon at 16.4' depth, 2" & 3" gravel pieces, most liekey pushed down hole by previous samples.	
							26				
	8D	24/19.2	17.00 - 19.00	WOR	Note 1		9			PP: 0; TV: 0.2, 0.1 kg/cm$_2$ Failed vane due to gravel observed.	
	V2		17.60 - 18.00	Su= 687/82 psf							
	V3		18.60 - 19.00	Su= 632/55 psf			9			Grey, moist, very soft, silty CLAY, little fine sand; sticky / will not roll	
							8			PP:0; TV:0.04 kg/cm$_2$	
20	1U	30/16.8	20.00 - 22.50				8			V2: 25 / 3 ft.-lb. V3: 23 / 2 ft.-lb. Shelby Tube recovery 1.4' (20' - 21.4')	
	V4		22.00 - 22.40	Su= 275/55 psf						V4: 10 / 2 ft.-lb.	
	V5		23.00 - 23.40	Su= 385/55 psf			11			V5: 14 / 2 ft.-lb.	
							10				
25	9D	24/24	25.00 - 27.00	WOR	Note 1		8			Grey, damp, very soft, CLAY, trace silt, very sticky, will not roll. PP: 0	
	V6		25.60 - 26.00	Su= 275/55 psf							
	V7		26.60 - 27.00	Su= 302/55 psf			10			V6: 10 / 2 ft.-lb. V7: 11 / 2 ft.-lb.	
							8				
							8				
30							8				

DRAFT

Remarks:

1. Blow counts for spoons driven through a disturbed field vane test location cannot be used to determine N-value.
2. Water Levels:
   - 4.6' b.g.s. at 5:15 PM 5/9/17; 4.6' b.g.s. at 10:00 AM 5/10/17; 2.4' b.g.s. at 4:30 PM 5/10/17.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Marsh Bridge
Location: Termont / Southwest Harbour, ME

Boring No.: BB-MAR-104
WIN:

Driller:	NEBC	**Elevation (ft.)**		**Auger ID/OD:**	
Operator:	Mike Porter	**Datum:**		**Sampler:**	
Logged By:	TRM	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140 lbs. / 30"
Date Start/Finish:	5/9/17 (12:40) / 5/10/17 (4:15 PM)	**Drilling Method:**	SSA / D+W	**Core Barrel:**	
Boring Location:		**Casing ID/OD:**		**Water Level*:**	
Hammer Efficiency Factor:		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
30	10D	24/24	30.50 - 32.50	WOR	Note 1		9			Grey wash. Rod dropped to 30.5'.	
	V8			Su= 385/82 psf			13			Grey with some brown staining, damp, very soft, CLAY, trace silt, very sticky, will not roll.	
	V9			Su= 385/82 psf			10			PP: 0 V8: 14 / 3 ft.-lb.	
							10			V9: 14 / 3 ft.-lb.	
							11				
35	11D	24/24	35.00 - 37.00	WOR	Note 2		11			Grey with some black staining, damp, very soft, CLAY, trace silt, very sticky, will not roll.	
	~~V10~~			~~Su= 385/82 psf~~						PP: 0	
	V11			Su= 467/82 psf			15			Grey wash	
							10			V9: 14 / 3 ft.-lb.	
							9			V9: 17 / 3 ft.-lb.	
							7				
40								-40.00		40.00	
	MV12	0/0	40.00 - 40.00	7/12/15/11			5			Failed 65 x 130 mm vane. Pushed a rock.	
							8				
							8				
							6				
							64			Chattering/ drinding during washout at 43.8; possibly gravel.	
45	12D	24/6	45.00 - 47.00	24/18/11/10	29	0	89			Gravel/rock fragments with little grey clay, trace silt, trace fine sand.	
							42			Grinding/chattering, rock fragments in wash to 45'.	
							62				
							98				
							93				
50	13D	21/3.6	50.00 - 51.75	26/12/9/(34/3")	21	0	87			Gravel/rock fragments, little grey clay, trace fine sand, trace silt, fragments of pink gravel. (2.0")	
	R1		51.50 - 52.50				73	-51.50		Wash to 50' grey with gravel & rock fragments.	
							97/6"			Grinding/chattering past 50', wash turned clear.	
	R2		52.50 - 55.50							51.50	
										Top of bedrock at 51.5', split spoon and casing refusal.	
										Run 1: Top 0.2'rock fragments, remaining is PINK GRANITE, very fractured, 60 degree dip.	
55	R3		55.20 - 60.20							R1 Core Times (min:sec):	
										51.5' - 52.5': (1:50)	
										Not smooth coring, drill rod rattled, 4" casing too much. Brought up bit and added 3" casing up to surface.	
										Pinkish-grey GRANITE, very fractured 2" to 6" spacing, dipping at about 45 degrees.	
										R2 Core Times (min:sec):	
										52.5' - 53.5': (2:20)	
60										53.5' - 54.5': (1:20)	

Remarks:

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   - 4.6' b.g.s. at 5:15 PM 5/9/17; 4.6' b.g.s. at 10:00 AM 5/10/17; 2.4' b.g.s. at 4:30 PM 5/10/17.

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Location: Termont / Southwest Harbour, ME

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WIN:

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Logged By:	TRM	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140 lbs. / 30"
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60								-60.20		54.5' - 55.2': (1:43) Pinkish-grey GRANITE, very fractured, 2" to 9" spacing, dipping at about 45 degrees. R3 Core Times (min:sec): 55.2' - 56.2': (1:38) 56.2' - 57.2': (2:00) 57.2' - 58.2': (2:43) 58.2' - 59.2': (1:48) 59.2' - 60.2': (2:00)	
65										60.20 **Bottom of Exploration at 60.20 feet below ground surface.**	
70											
75											
80											
85											
90											

DRAFT

Remarks:

1. Blow counts for spoons driven through a disturbed field vane test location cannot be used to determine N-value.
2. Water Levels:
   - 4.6' b.g.s. at 5:15 PM 5/9/17; 4.6' b.g.s. at 10:00 AM 5/10/17; 2.4' b.g.s. at 4:30 PM 5/10/17.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Marsh Bridge

Location: Termont / Southwest Harbour, ME

Boring No.: BB-MAR-101

WIN:

Driller:	NEBC	**Elevation (ft.)**		**Auger ID/OD:**	
Operator:	Mike Porter	**Datum:**		**Sampler:**	
Logged By:	TRM	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140 lbs. / 30"
Date Start/Finish:	05/11/17 8:50 AM/11:00 AM	**Drilling Method:**	SSA / D+W	**Core Barrel:**	
Boring Location:		**Casing ID/OD:**		**Water Level*:**	4.6' b.g.s. at 11:20 AM on 5/
Hammer Efficiency Factor:		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA			0.9' Asphalt	
								-0.90		0.90	
	1D	24/16.8	1.00 - 3.00	4/8/14/12	22	0	14			Brown, damp, medium dense, fine to coarse SAND, some clay, some silt, trace gravel, 'petrol odor' (FILL)	
							3				
							8				
							14				
5								-5.00		5.00	
	2D	24/12	5.00 - 7.00	7/4/9/1	13	0	27			Brownish-grey, dry, stiff, silty CLAY, trace gravel, trace organics.	
							D+W 30				
							27				
							56				
								-9.10		Brownish-grey wash to 9' eith fragments of rock, looks like piesces of granite. Grinding/chattering at 9'.	
10				RQD = 42%			103			9.10	
										Top of Rock at 9.1'.	
										Run 1: Greyish PINK GRANITE, little black intrusions, spacing 1" to 8", dipping 35 to 45 degrees, some vertical fracturing, highly fractured zone from 11.1" to 12.1".	
								-13.30		R1 Core Times (min:sec):	
				RQD = 70%						9.1' - 10.1': (2:00)	
15										10.1' - 11.1': (1:45)	
										11.1 - 12.1: (1:45)	
										12.1' - 13.1: (2:15)	
										13.1 - 13.3: (0:30) 74% Recovery	
										13.30	
										Bottom of Exploration at 13.30 feet below ground surface.	
										Run 2: Pinkish grey GRANITE, with some black intrusions, spacing 2" to 11", dipping 30 to 45 degrees.	
										R1 Core Times (min:sec):	
20										13.3' - 14.3': (1:45)	
										14.3' - 15.3': (1:45)	
										15.3' - 16.3: (2:12)	
										16.3' - 17.3: (2:08)	
										17.3' - 18.1: (1:28) 100% Recovery	
25											
30											

DRAFT

Remarks:

1. Hole open to 16.3' b.g.s. 2. Borehole filled with gravelly cuttings, roadside gravel, 25 lbs of gravel, and 20 lbs of coldpatch.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Marsh Bridge
Location: Termont / Southwest Harbour, ME

Boring No.: BB-MAR-102
WIN:

Driller:	NEBC	**Elevation (ft.)**		**Auger ID/OD:**	
Operator:	Mike Porter	**Datum:**		**Sampler:**	
Logged By:	TRM	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140 lbs. / 30"
Date Start/Finish:	15/11/17 11:50 AM/4:00 PM	**Drilling Method:**	SSA / D+W	**Core Barrel:**	
Boring Location:		**Casing ID/OD:**		**Water Level*:**	6.8' b.g.s. at 4:20 PM 5/11/17
Hammer Efficiency Factor:		**Hammer Type:**	Automatic ☐ Hydraulic ☐ Rope & Cathead ☐		

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

Depth (ft.)	Sample No.	Pen./Rec. (in.)	Sample Depth (ft.)	Blows (/6 in.) Shear Strength (psf) or RQD (%)	N-uncorrected	N_{60}	Casing Blows	Elevation (ft.)	Graphic Log	Visual Description and Remarks	Laboratory Testing Results/ AASHTO and Unified Class.
0							SSA			0.95' Asphalt.	
								-0.95		0.95	
	1D	24/14.4	1.00 - 3.00	14/10/9/8	19	0	29			Brown, damp, medium dense, fine to coarse SAND, some gravel, trace silt (FILL).	
							3				
							3				
							9				
5								-5.00		5.00	
	2D	24/12	5.00 - 7.00	3/1/1/2	3	0	12			Brown-grey with brown-red staining, silty CLAY, little gravel. PP: 2.25 TSF	
							D+W 9				
							50				
							130			Hard drilling from 8' to 9'.	
							142			Wash from 5' to 10' started brown, sandy, gravelly; turned milky brown to dark brown, organic smell.	
10		24/0	10.00 - 12.00	3/1/1/2	2	0	22			Chattering/grinding at 8.8' Pushed a rock/gravel.	
	MV1						30			Grey-brown clayey wash from 10' to 12'. Failed 65x130mm vane. Gravel observed.	
	3D	24/21.6	12.00 - 14.00	3/4/5/7	9	0	21			3" Spoon: Grey, damp, loose, fine SAND, with some silt and clay, trace organics, increasing sand with depth. PP: 1.5,1.5,1.5 TSF; TV: 0.375 (TOP), 0. 24 (BOTTOM) kg/cm2.	
							30				
	MV2						35			Failed 65x130mm vane. Gravel observed.	
15	4D	24/8.4	15.00 - 17.00	14/16/14/17	30	0	40			3" Spoon: Grey with dark staining, damp, medium stiff, CLAY, sticky, pushed a piece of grave;, sample sheared in half vertically. PP: 2.25, 1.0 TSF.	
	MV3						32			Failed 65x130mm vane. Vane pushed 6" then refusal. Sticky grey clay on vane.	
							28				
							20				
				RQD = 35%			17/6"			Top of rock at 19.6' based on casing refusal.	
20										Run 1: Grey-pink, coarse grained, GRANITE with black intrusions spaced 2" to 5", dipping about 45 degrees, some vertical fractures, cement infilling. Core Times: (min:sec): 19.6' - 20.6': (2:42) 20.6' - 21.6': (1:40) 21.6' - 22.6': (1:25) 22.6' - 23.6': (1:43) 23.6' - 24.6': (2:10) 86% Recovery	
				RQD = 51%				-24.60		24.60	
25										**Bottom of Exploration at 24.60 feet below ground surface.** Run 2: Grey-pink, coarse grained, GRANITE with black intrusions spaced 2" to 5", mostly dipping about 45 degrees, two at 80 degrees. Core Times: (min:sec): 24.6' - 25.6': (2:12) 25.6' - 26.6': (1:40) 26.6' - 27.6': (1:40)	
30											

DRAFT

Remarks:

1. Borehole filled with cuttings, roadside gravel, 0.5 bag of gravel, 15 lbs. cold patch.

Stratification lines represent approximate boundaries between soil types; transitions may be gradual.

* Water level readings have been made at times and under conditions stated. Groundwater fluctuations may occur due to conditions other than those present at the time measurements were made.

Maine Department of Transportation

Soil/Rock Exploration Log
US CUSTOMARY UNITS

Project: Marsh Bridge
Location: Termont / Southwest Harbour, ME

Boring No.: BB-MAR-102
WIN:

Driller:	NEBC	**Elevation (ft.)**		**Auger ID/OD:**	
Operator:	Mike Porter	**Datum:**		**Sampler:**	
Logged By:	TRM	**Rig Type:**	Mobile B53	**Hammer Wt./Fall:**	140 lbs. / 30"
Date Start/Finish:	15/11/17 11:50 AM/4:00 PM	**Drilling Method:**	SSA / D+W	**Core Barrel:**	
Boring Location:		**Casing ID/OD:**		**Water Level*:**	6.8' b.g.s. at 4:20 PM 5/11/17
Hammer Efficiency Factor:		**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: 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.
30										27.6' - 28.6': (2:25) 28.6' - 29.6': (2:03) 96% Recovery	
35											
40											
45											
50											
55											
60											

DRAFT

Remarks:

1. Borehole filled with cuttings, roadside gravel, 0.5 bag of gravel, 15 lbs. cold patch.

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.

APPENDIX E
CALCULATIONS

APPENDIX E.1
CLAY DESIGN PARAMETERS

CALCULATIONS

Date:	12/5/2017	**Made by:**	CJS
Project No.:	1668741	**Checked by:**	MCM
Subject:	Organic and Clay Design Parameters Summary	**Reviewed by:**	MSP
Project Short Title:	VHB Marsh Bridge Final Design		

1.0 Objective

Determine appropriate design parameters for the Presumpscot Clay layers based on field observations, field data/measurements, laboratory test results, and correlations to published data.

2.0 References

1. Field SPT and VST data collected during the Phase I and II field explorations as presented on the boring logs shown in Appendix A of the Final Geotechnical Design Report.
2. Geotesting Express laboratory testing data as shown in Appendix C of the Final Geotechnical Design Report.
3. Ladd, Charles C. and Degroot, Don J. (1993, revised 1994), "Recommended Practice for Soft Ground Site Characterization." The Arthur Casagrande Lecture, *Proc. of the 12th Panamerican Conference on Soil Mechanics and Geotechnical Engineering* , Boston, MA, Vol. 1, pp 3-57. 4. Internal project data base for Presumpscot Clay Properties.

3.0 Attachments

1. Shear Strength Design Curves
2. OCR Design Curve
3. Casagrande Constructions on laboratory consolidation data.

4.0 Assumptions

1. Clay properties were collected at discrete locations. The design clay properties are assigned based on similarities in field observations and laboratory data.

5.0 Calculation

5.1 Design Shear Strength Profile

Field VST values from Reference 1 were corrected for plasticity index (PI) using the method outlined in Reference 3. PI values were tested at discrete intervals and were applied to each of the VST elevations. The actual PI at each of the VST locations may vary from those assumed. The correction factor is taken from the figure below (Ref. 3).

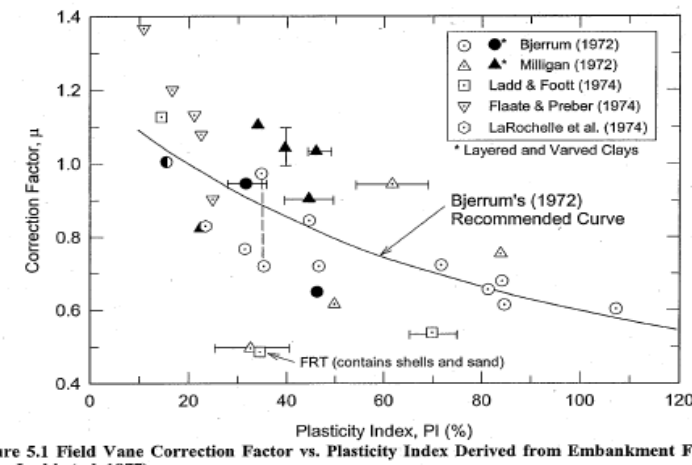


Figure 5.1 Field Vane Correction Factor vs. Plasticity Index Derived from Embankment Failures (after Ladd et al. 1977)

The resulting corrected VST values are plotted on a figure shown in Attachment 1. This figure also has trend lines for recommended design strengths.

Date:	12/5/2017	**Made by:**	CJS
Project No.:	1668741	**Checked by:**	MCM
Subject:	Organic and Clay Design Parameters Summary	**Reviewed by:**	MSP
Project Short Title:	VHB Marsh Bridge Final Design		

5.2 Presumpscot Clay Overconsolidation Ratio (OCR) Profile

The overconsolidation ratios (OCR) with the Presumpscot clay profile, are calculated based on VST data, in situ effective stress, and plasticity index. This calculation procedure is outlined in Reference 3. A sample calculation is performed below for boring BB-MAR-205 at a depth of 19 feet:

$$OCR = \left(\frac{s_u(FV)/\sigma'_{v0}}{S_{FV}} \right)^{1.05}$$ (Reference 3)

Where:

$s_u(FV)$ = Undrained shear strength from field vane data

S_{FV} = Field vane undrained strength ratio (su(FV)/s'v0) at OCR =1, from Figure 5.2 (Reference 3).

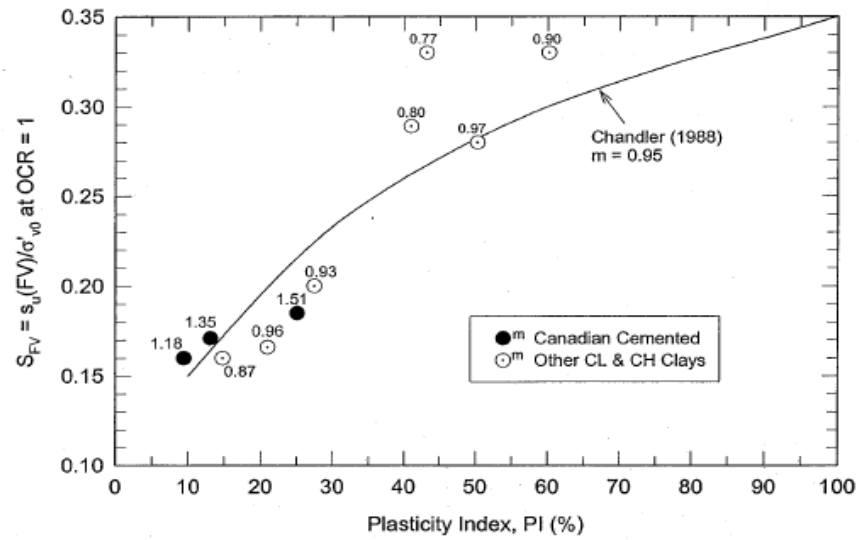


Figure 5.2 Field Vane Undrained Strength Ratio at OCR = 1 vs. Plasticity Index for Homogeneous Clays (no shells or sand) [data points from Lacasse et al. 1978 and Jamiolkowski et al. 1985]

15

σ'_{v0} =	1080	psf, (effective stress in BB-MAR-205 at a depth of 19 feet)
$s_u(FV)$ =	746	psf, (vane shear strength at a depth of 19 feet in BB-MAR-205)
S_{FV} =	0.20	
OCR =	3.7	

The resulting calculated OCR values are plotted on a figure shown in Attachment 2. This figure also has a trend line for the recommended design OCR with elevation.

Date:	12/5/2017	**Made by:**	CJS
Project No.:	1668741	**Checked by:**	MCM
Subject:	Organic and Clay Design Parameters Summary	**Reviewed by:**	MSP
Project Short Title:	VHB Marsh Bridge Final Design		

5.3 Consolidation parameters

Using consolidation data from Appendix C, complete Casagrande constructions to determine $c_{c\varepsilon}$ and $c_{r\varepsilon}$ values and select representative c_v values from the lab data. Refer to the Casagrande Constructions shown in Attachment 3 and the laboratory data in Appendix C for compression/recompression ratios and coefficient of consolidation respectively.

Based on Golder's project experience, the following range of compression/recompression ratios can be expected:
Organic Silts & Clays: $c_{c\varepsilon}$ = 0.25-0.58, $c_{r\varepsilon}$ = NOT USED FOR THIS PROJECT (assumed OCR = 1)
Presumpscot Clay: $c_{c\varepsilon}$ = 0.09-0.35, $c_{r\varepsilon}$ = 0.010-0.033

For the conditions encountered on this project (water content, consistency, etc.), the values obtained from the Casagrande constructions appear to be reasonable.

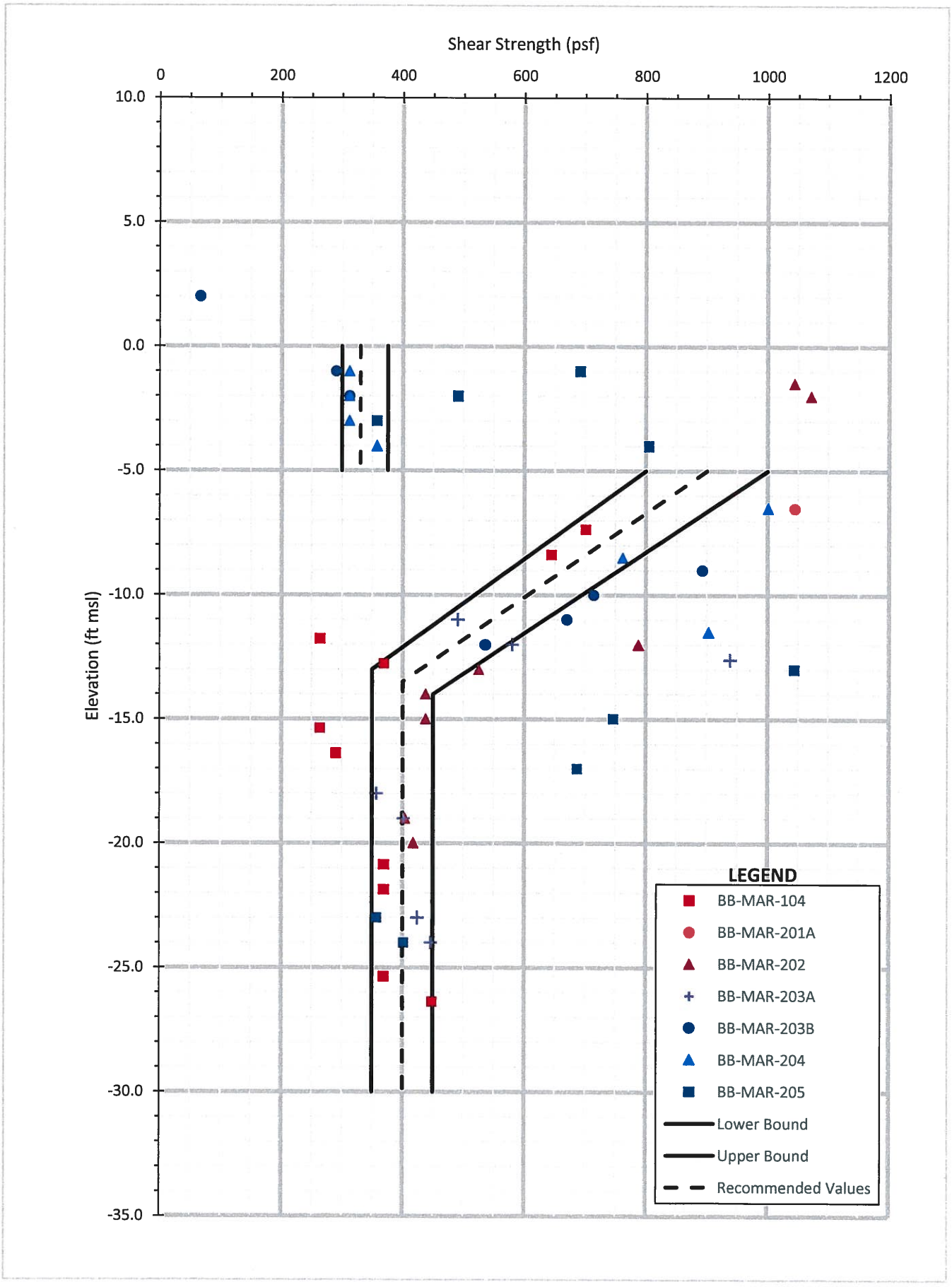
6.0 Conclusions

1. Use the design curve in Attachment 1 when modeling organic silts/clay and/or Presumpscot Clay shear strengths.
2. Use the design curve in Attachment 2 when selecting OCR for the Presumpscot Clay.
3. Use the following consolidation parameters in design:

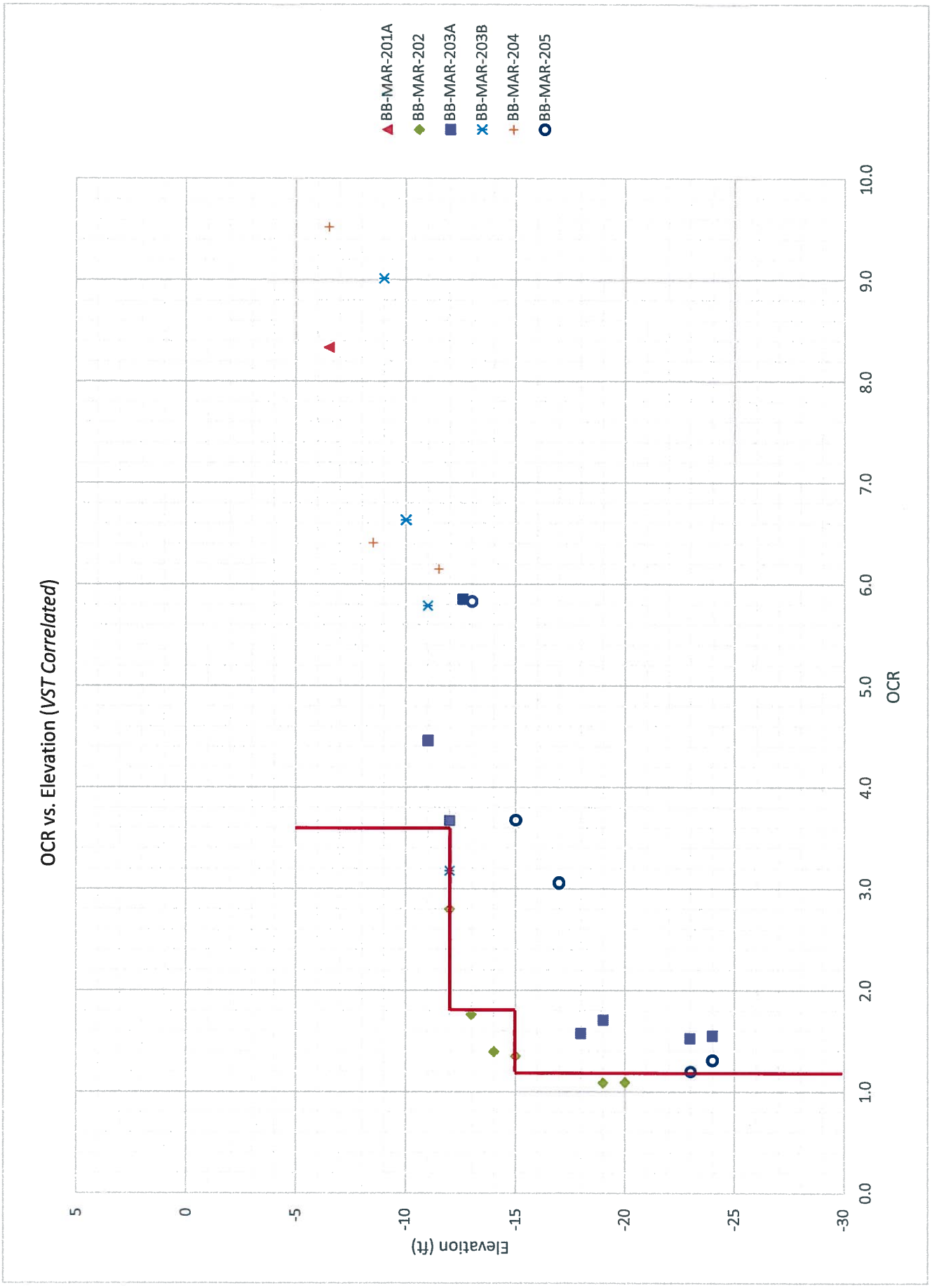
Organic Silts & Clays: ccε = 0.37, crε = NOT USED (assumed OCR = 1), cv =4.0x10^{-7} ft^2/s

Presumpscot Clay: ccε = 0.14, crε = 0.02, cv = 5.0x10^{-6} ft^2/s

Attachment 1



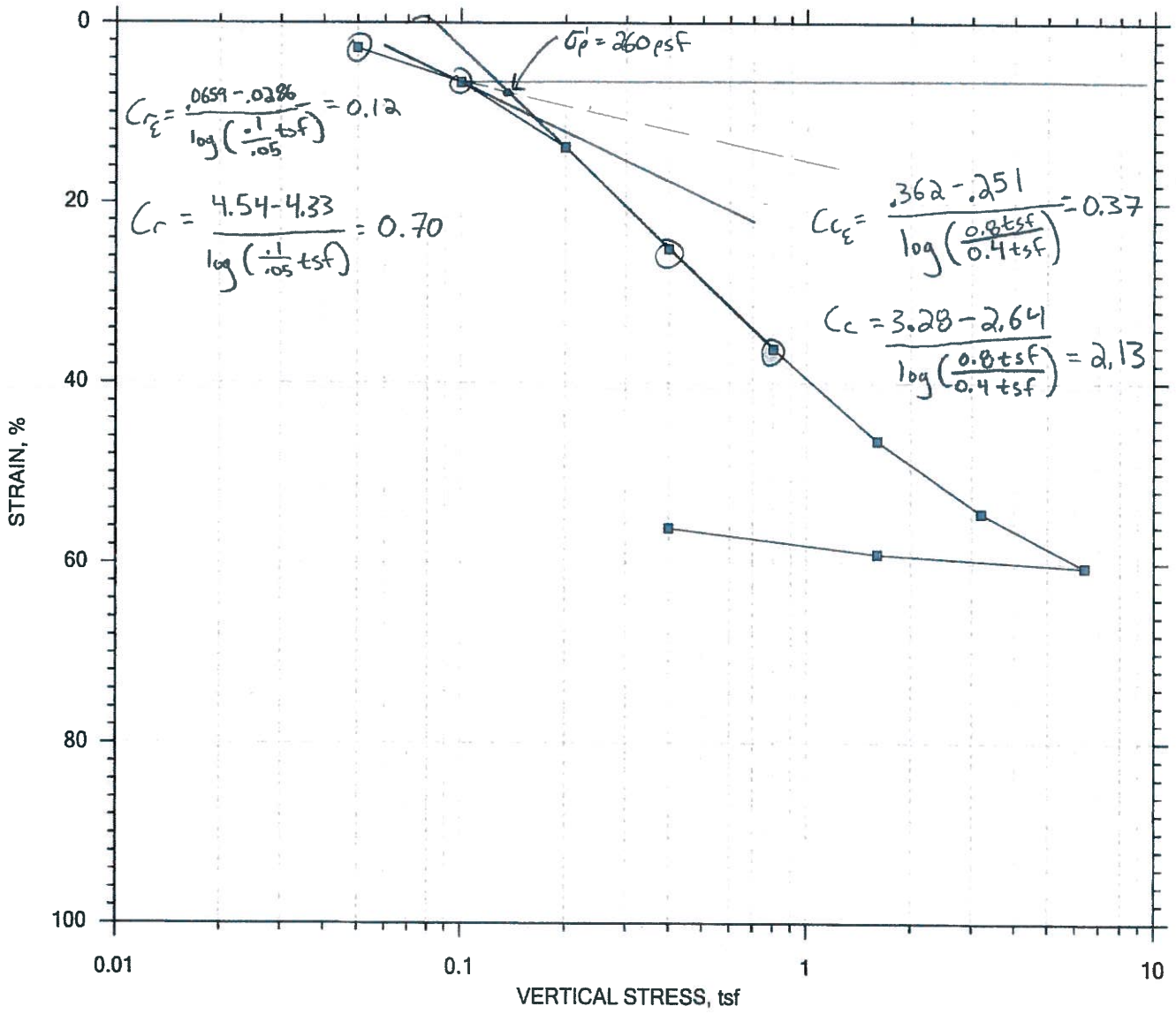
Attachment 2



Attachment 3

One-Dimensional Consolidation by ASTM D2435 - Method A

SUMMARY REPORT



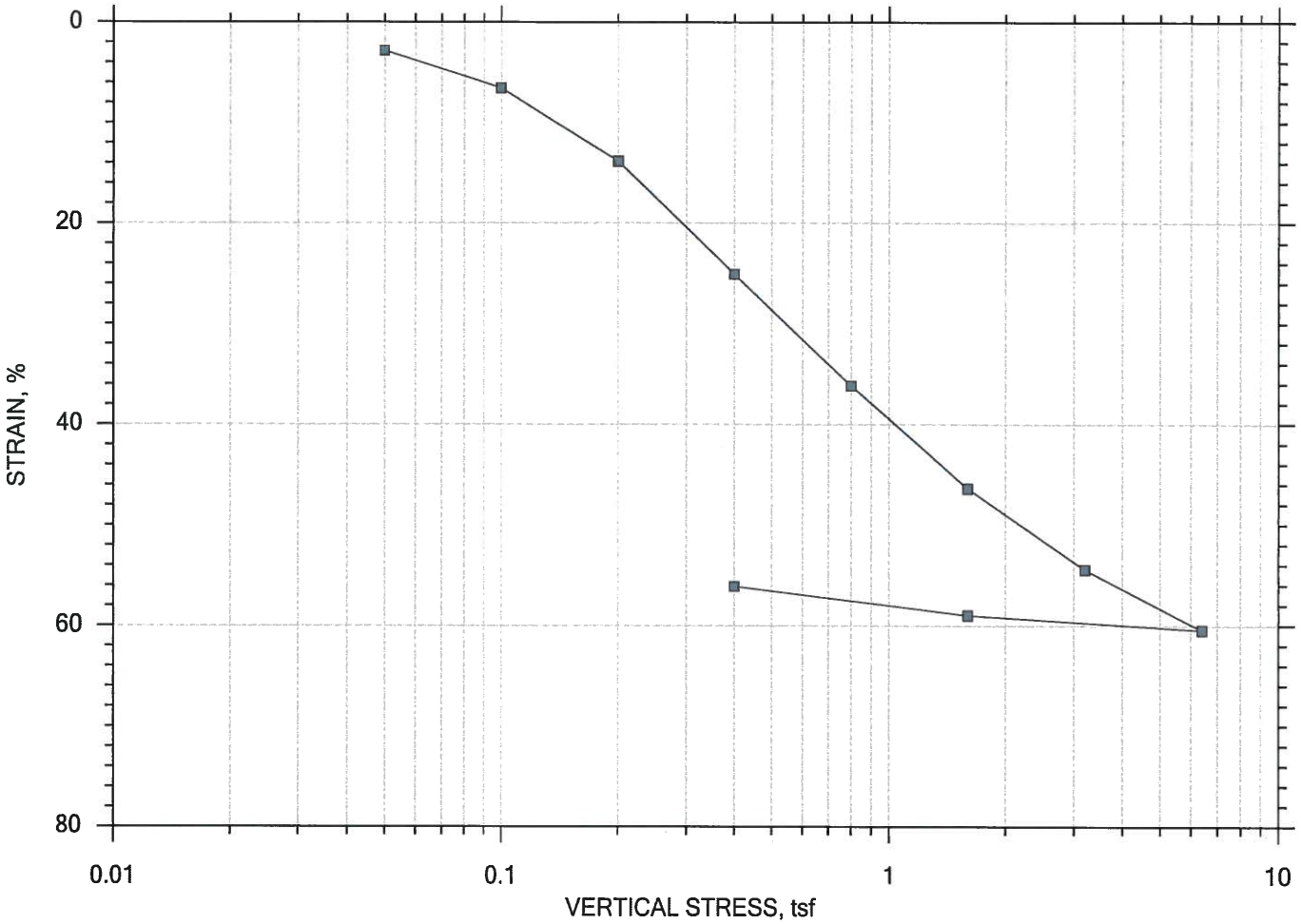
				Before Test	After Test
Current Vertical Effective Stress: —				Water Content, %	178.61	66.58
Preconsolidation Stress: —				Dry Unit Weight, pcf	26.705	58.054
Compression Ratio: —				Saturation, %	92.63	100.00
Diameter: 2.5 in		Height: 1 in		Void Ratio	4.71	1.63
LL: 255	PL: 100	PI: 155	GS: 2.44			

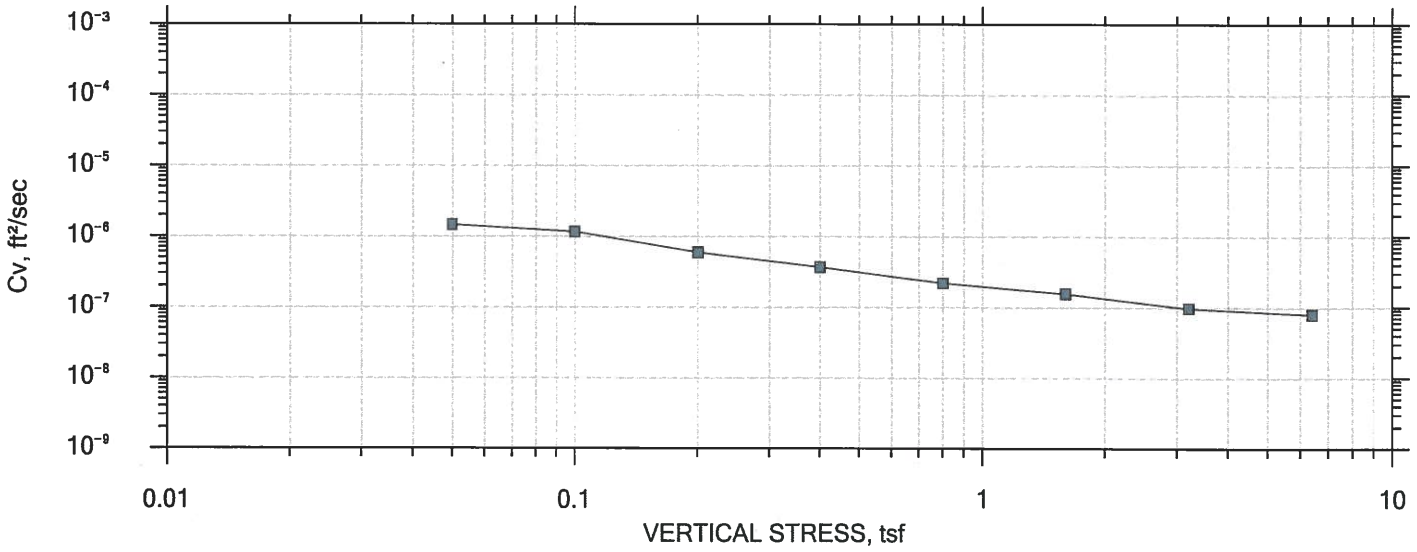
GeoTesting EXPRESS

Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
Boring No.: BB-MAR-203A	Tested By: md	Checked By: njh
Sample No.: U-1	Test Date: 10/30/17	Test No.: IP-3
Depth: 6.5-8.5 ft	Sample Type: intact	Elevation: —
Description: Wet, very dark brown silt with sand and organics		
Remarks: System R&D		
Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method A

SUMMARY REPORT



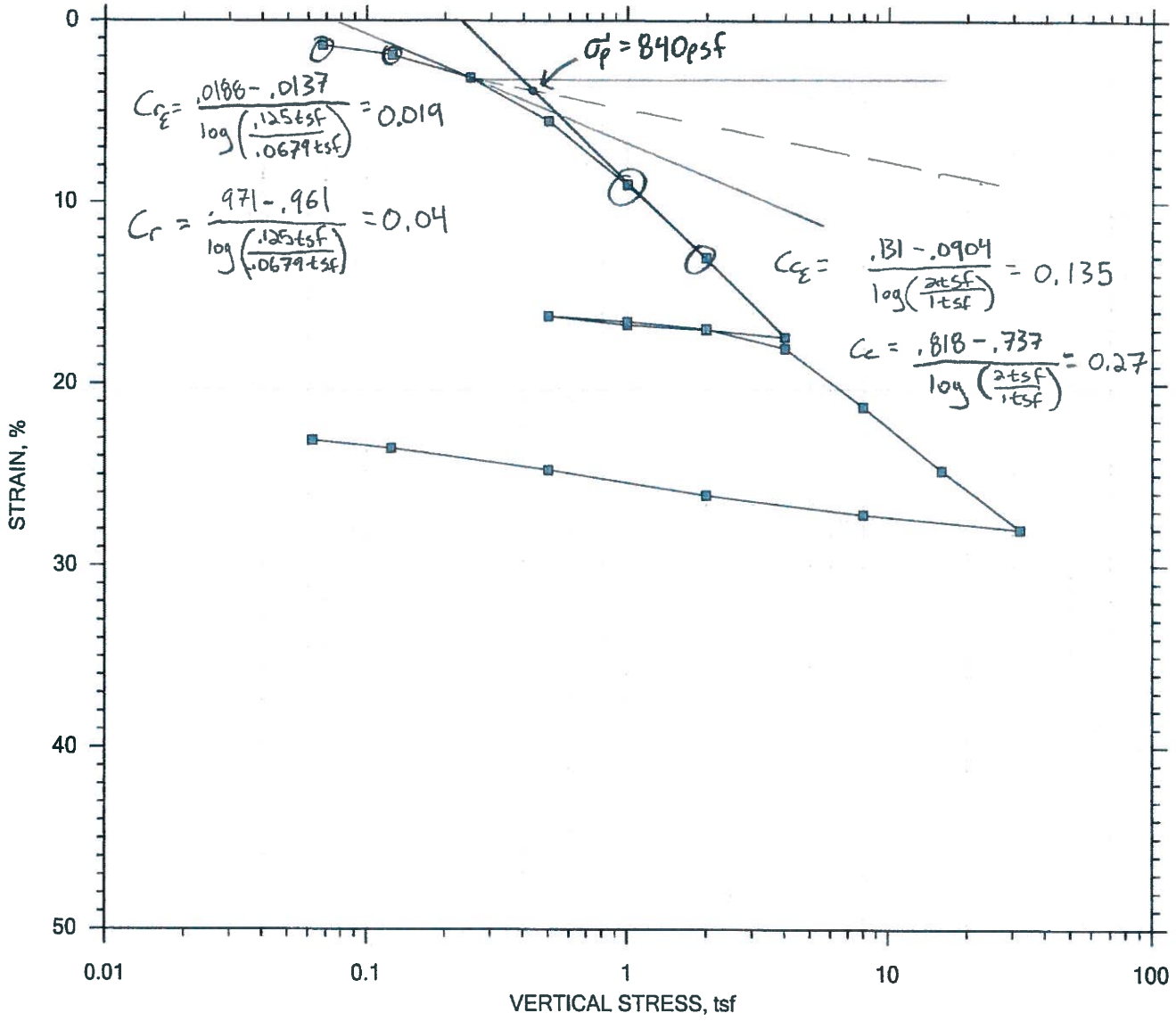


GeoTesting EXPRESS

Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
Boring No.: BB-MAR-203A	Tested By: md	Checked By: njh
Sample No.: U-1	Test Date: 10/30/17	Test No.: IP-3
Depth: 6.5-8.5 ft	Sample Type: intact	Elevation: ---
Description: Wet, very dark brown silt with sand and organics		
Remarks: System R&D		
Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method B

SUMMARY REPORT



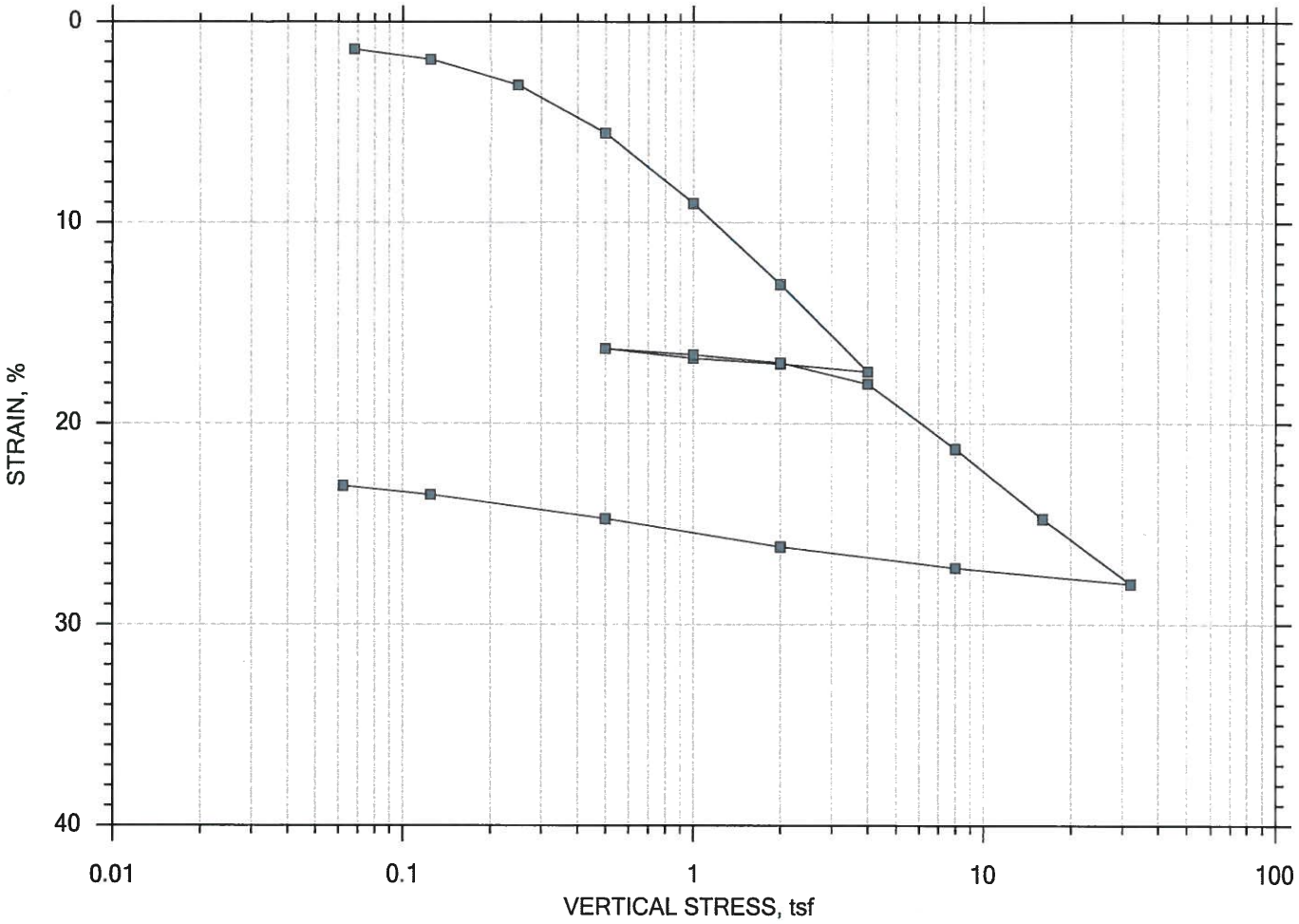
				Before Test	After Test
Current Vertical Effective Stress: —			Water Content, %	36.34	20.35
Preconsolidation Stress: —			Dry Unit Weight, pcf	85.751	109.94
Compression Ratio: —			Saturation, %	99.91	100.00
Diameter: 2.5 in		Height: 1 in	Void Ratio	1.00	0.56
LL: —	PL: —	PI: —	GS: 2.74		

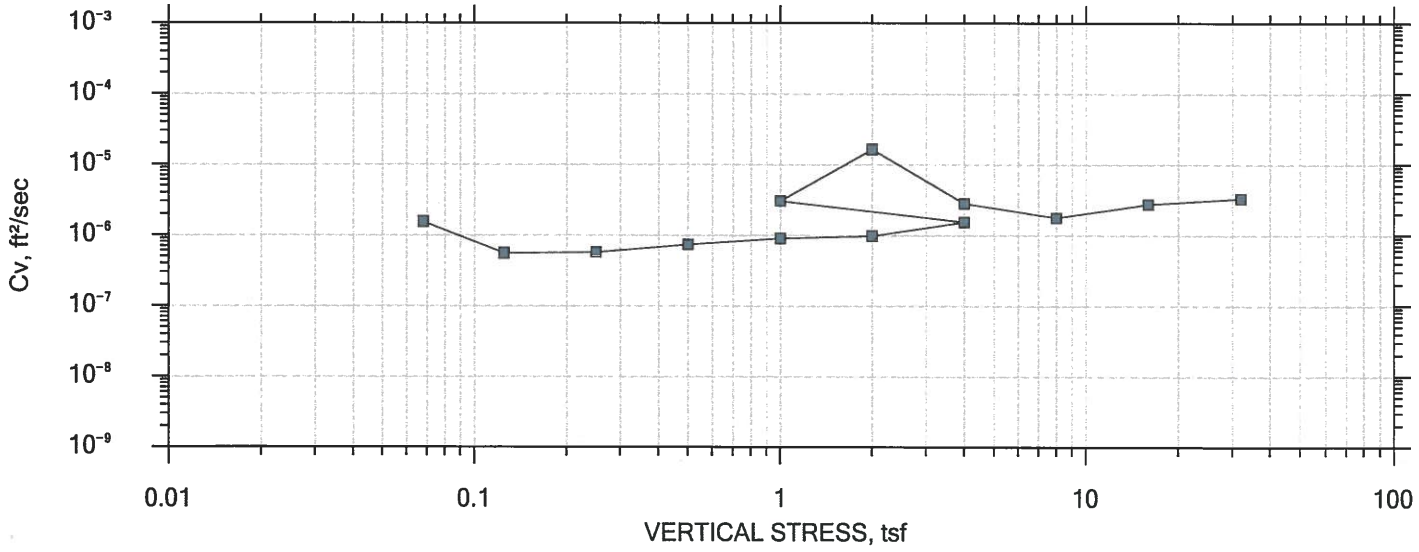
GeoTesting EXPRESS

Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
Boring No.: BB-MAR-104	Tested By: md	Checked By: njh
Sample No.: U-1	Test Date: 05/30/17	Test No.: IP-1
Depth: 20-21.4 ft	Sample Type: intact	Elevation: —
Description: Moist, gray silty clay		
Remarks: System O, Swell Pressure = 0.0679 tsf		
Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method B

SUMMARY REPORT



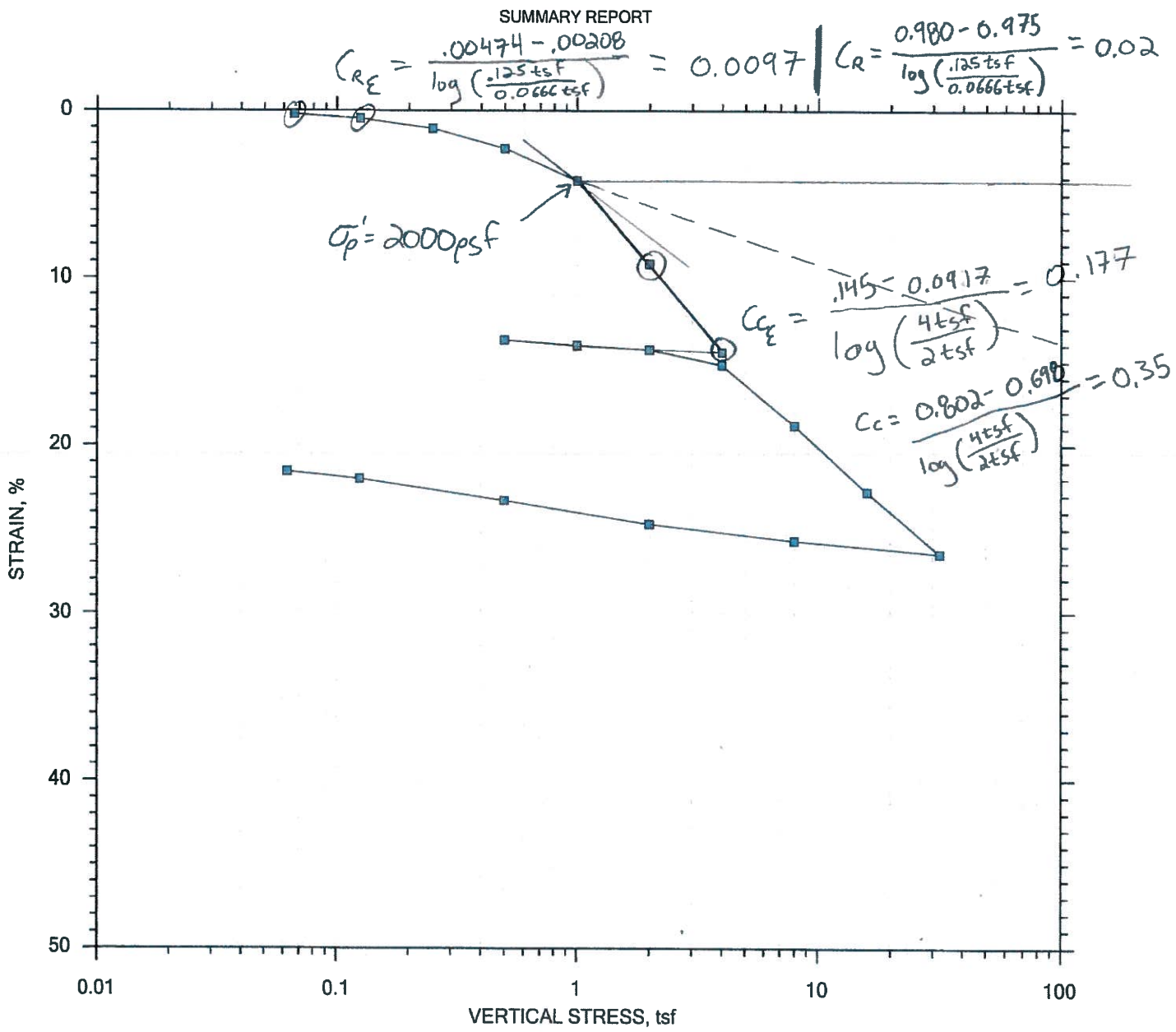


GeoTesting EXPRESS

Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
Boring No.: BB-MAR-104	Tested By: md	Checked By: njh
Sample No.: U-1	Test Date: 05/30/17	Test No.: IP-1
Depth: 20-21.4 ft	Sample Type: intact	Elevation: ---
Description: Moist, gray silty clay		
Remarks: System O, Swell Pressure = 0.0679 tsf		
Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method B

SUMMARY REPORT



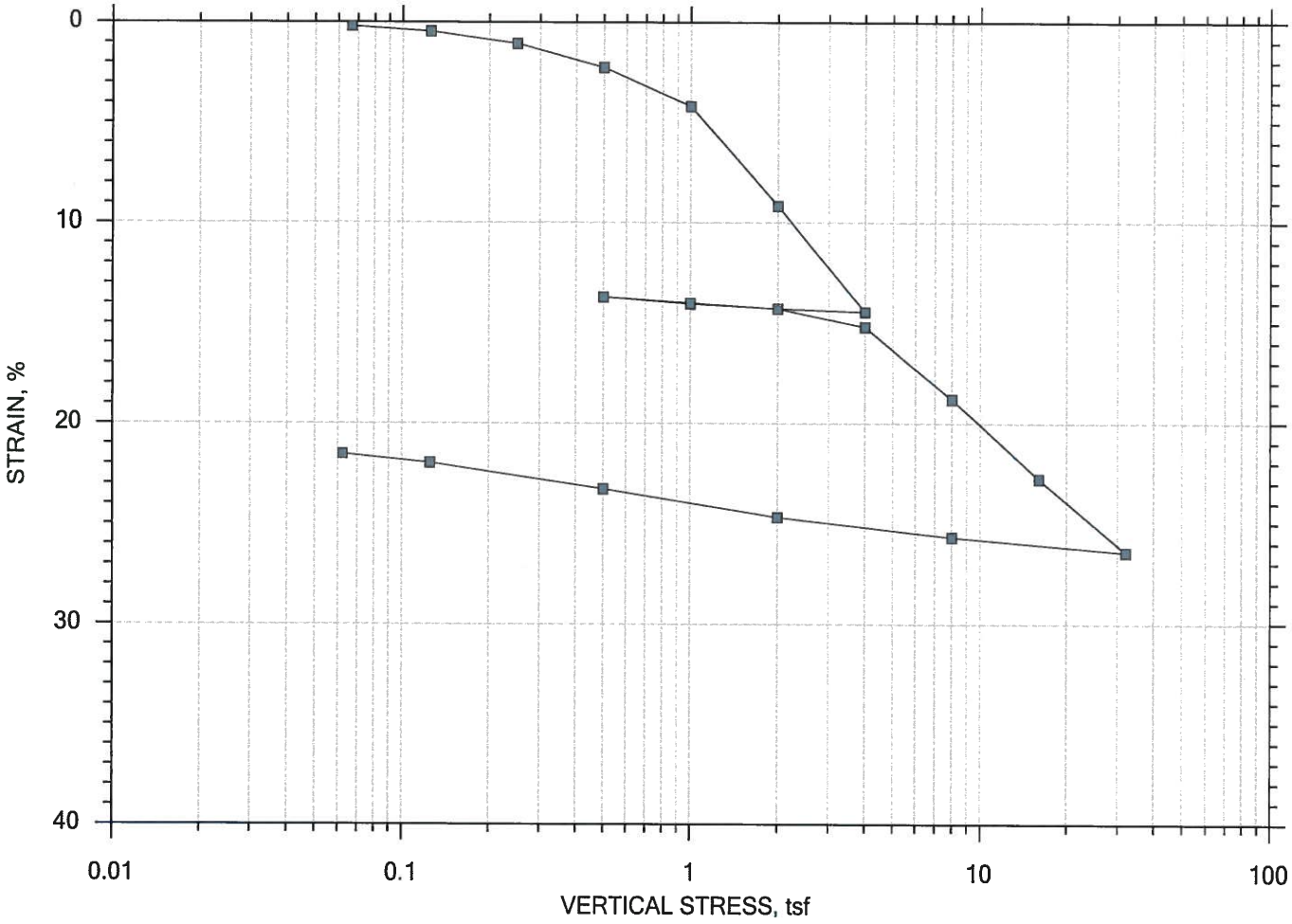
				Before Test	After Test
Current Vertical Effective Stress: —			Water Content, %	35.20	20.04
Preconsolidation Stress: —			Dry Unit Weight, pcf	86.016	110.28
Compression Ratio: —			Saturation, %	97.78	100.00
Diameter: 2.5 in		Height: 1 in	Void Ratio	0.98	0.55
LL: —	PL: —	PI: —	GS: 2.73		

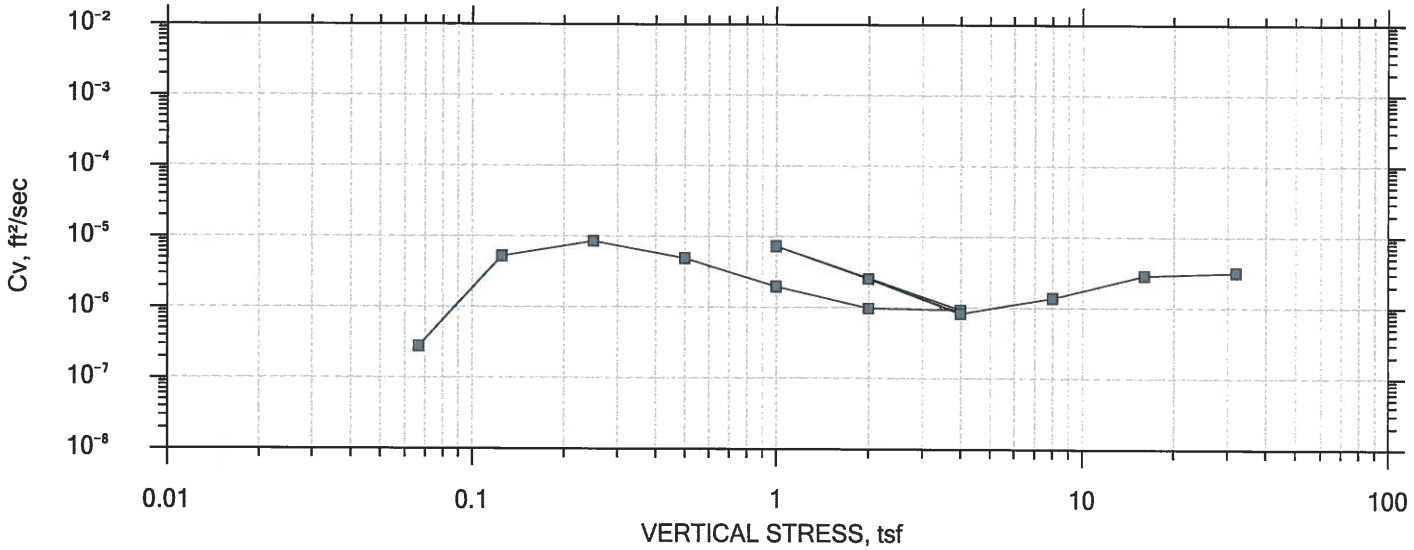
GeoTesting EXPRESS

Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
Boring No.: BB-MAR-202	Tested By: md	Checked By: njh
Sample No.: U1	Test Date: 10/16/17	Test No.: IP-2
Depth: 35-37 ft	Sample Type: intact	Elevation: —
Description: Moist, gray silty clay		
Remarks: System Y, Swell Pressure = 0.0666 tsf		
Displacement at End of Increment		

One-Dimensional Consolidation by ASTM D2435 - Method B

SUMMARY REPORT





GeoTesting EXPRESS	Project: Marsh Bridge	Location: Tremont/Southwest Harbor, ME	Project No.: GTX-306503
	Boring No.: BB-MAR-202	Tested By: md	Checked By: njh
	Sample No.: U1	Test Date: 10/16/17	Test No.: IP-2
	Depth: 35-37 ft	Sample Type: intact	Elevation: ---
	Description: Moist, gray silty clay		
	Remarks: System Y, Swell Pressure = 0.0666 tsf		
	Displacement at End of Increment		

APPENDIX E.2
EARTH PRESSURES

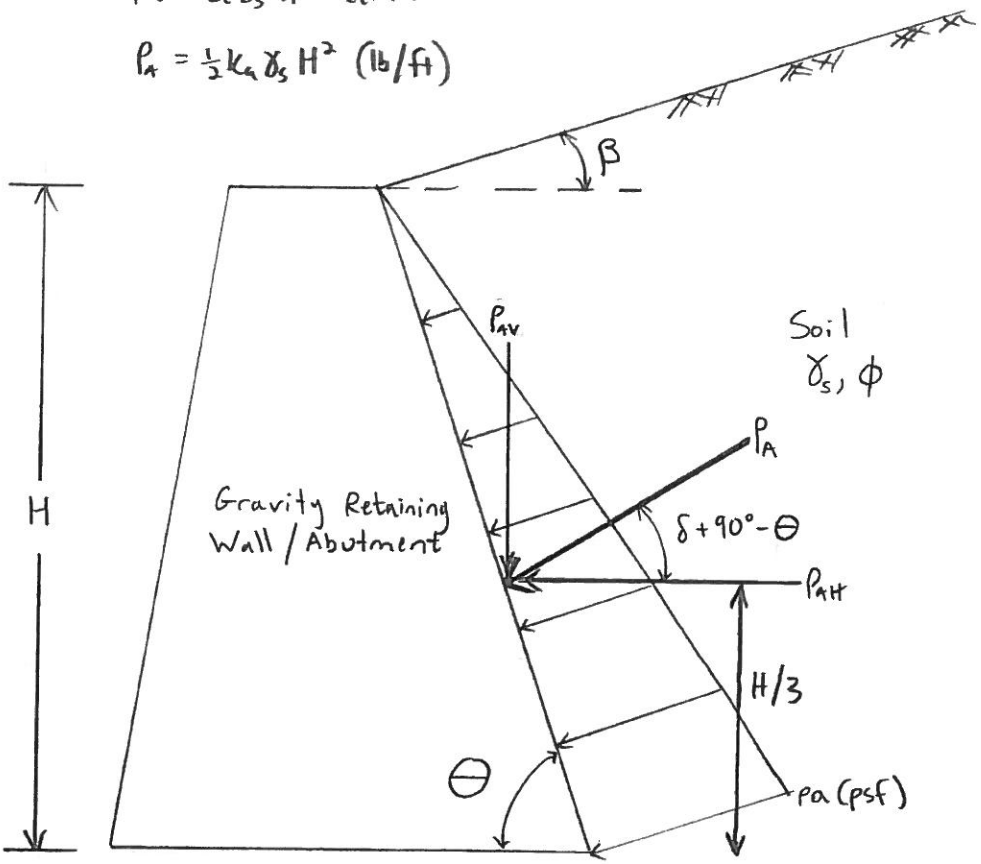
Golder Associates Manchester, New Hampshire	SUBJECT Active Earth Nomenclature		
Job No. 1668741	Made By: CJS	Date: 6/26/2017	
Ref.	Checked: mcm	Sheet 1 of 16	
	Reviewed: MSP		

Coulomb Active Earth Pressure Theory Nomenclature

ϕ = effective friction angle of soil
δ = friction angle between fill and wall
β = angle of fill to the horizontal
θ = angle of back face of wall to the horizontal
γ_s = unit weight of soil
K_a = Coulomb Active Earth Pressure Coefficient

$p_a = K_a \gamma_s H$ (psf)

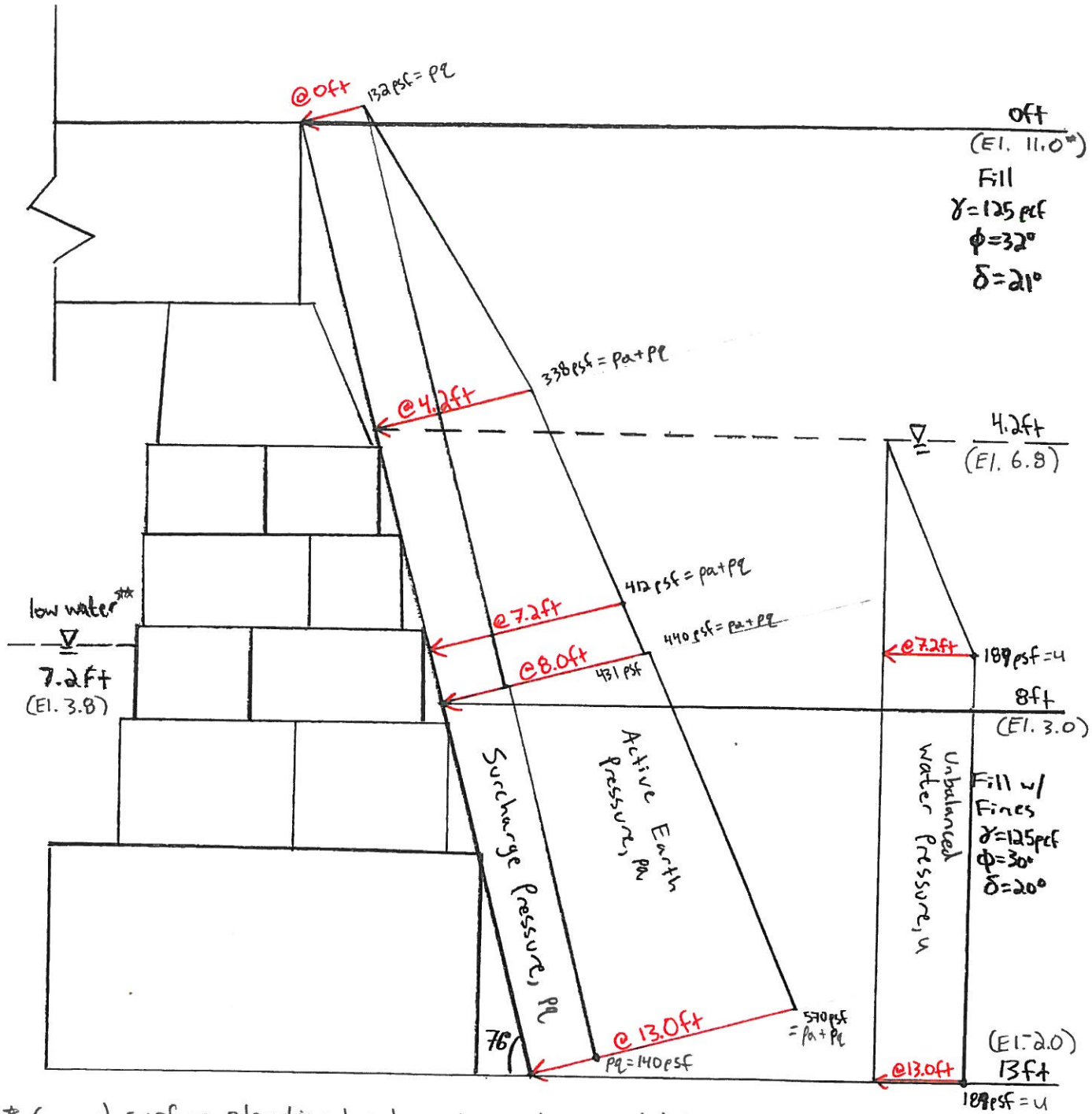
$P_A = \frac{1}{2} K_a \gamma_s H^2$ (lb/ft)



Where: p_a = active earth pressure (psf)
P_A = active earth pressure resultant (lb/ft)
P_{AH} = horizontal component of active earth Pressure resultant
$P_{AH} = P_A \cos(\delta + 90 - \theta)$

P_{AV} = vertical component of active earth pressure resultant
$P_{AV} = P_A \sin(\delta + 90 - \theta)$

Golder Associates Manchester, New Hampshire	SUBJECT East Abutment - Active Earth Pressures		
	Job No. 1668741	Made By: CJS	Date: 7/5/17
	Ref.	Checked: mcm	Sheet 2 of 16
		Reviewed: MSP	

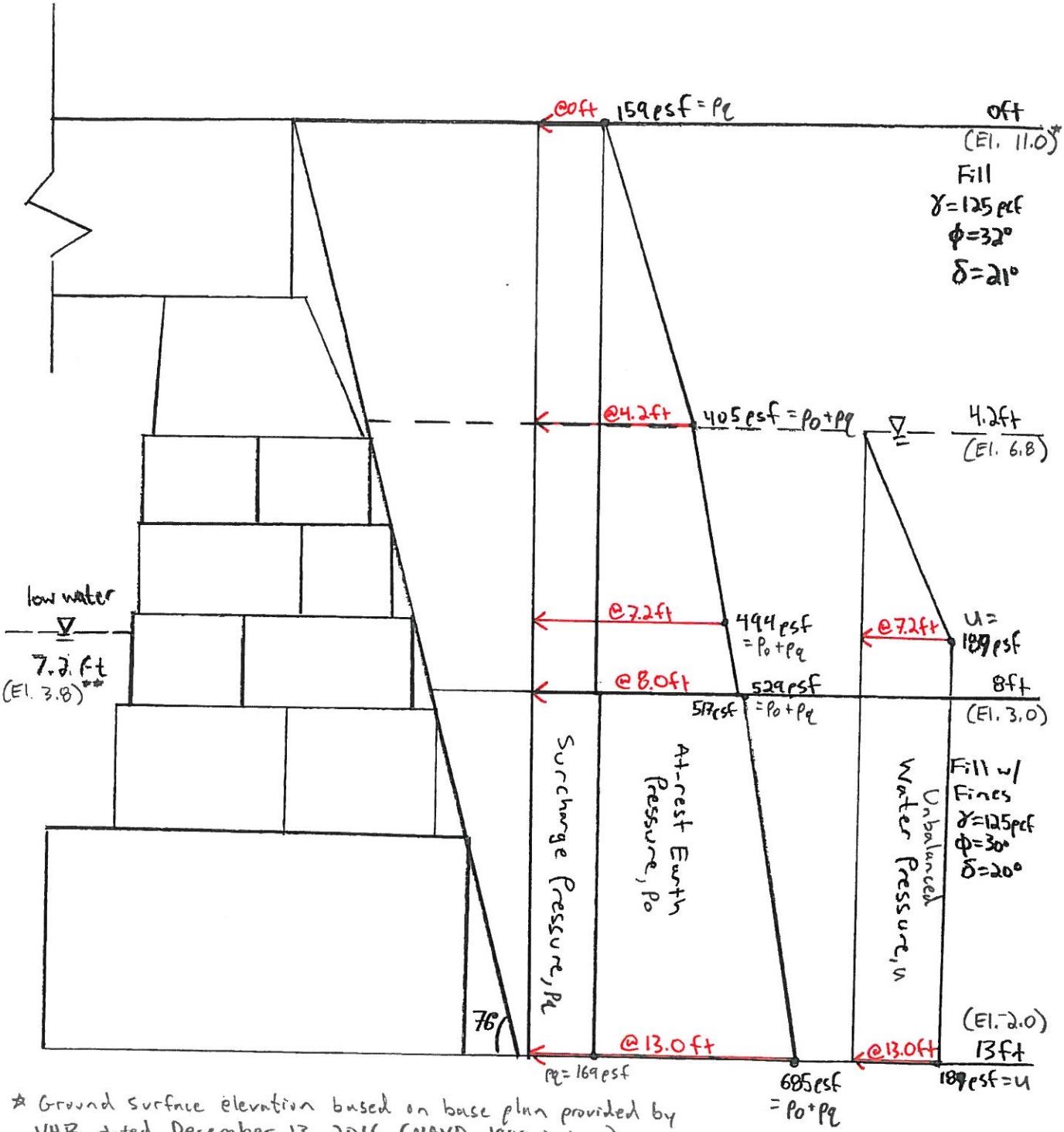


* Ground surface elevation based on base plan provided by VHB dated December 13, 2016 (NAVD 1988 Datum).

** "Low water" as described on 1931 design plans. If current extreme low water elevation is lower, it should be used for design.

Golder Associates
Manchester, New Hampshire

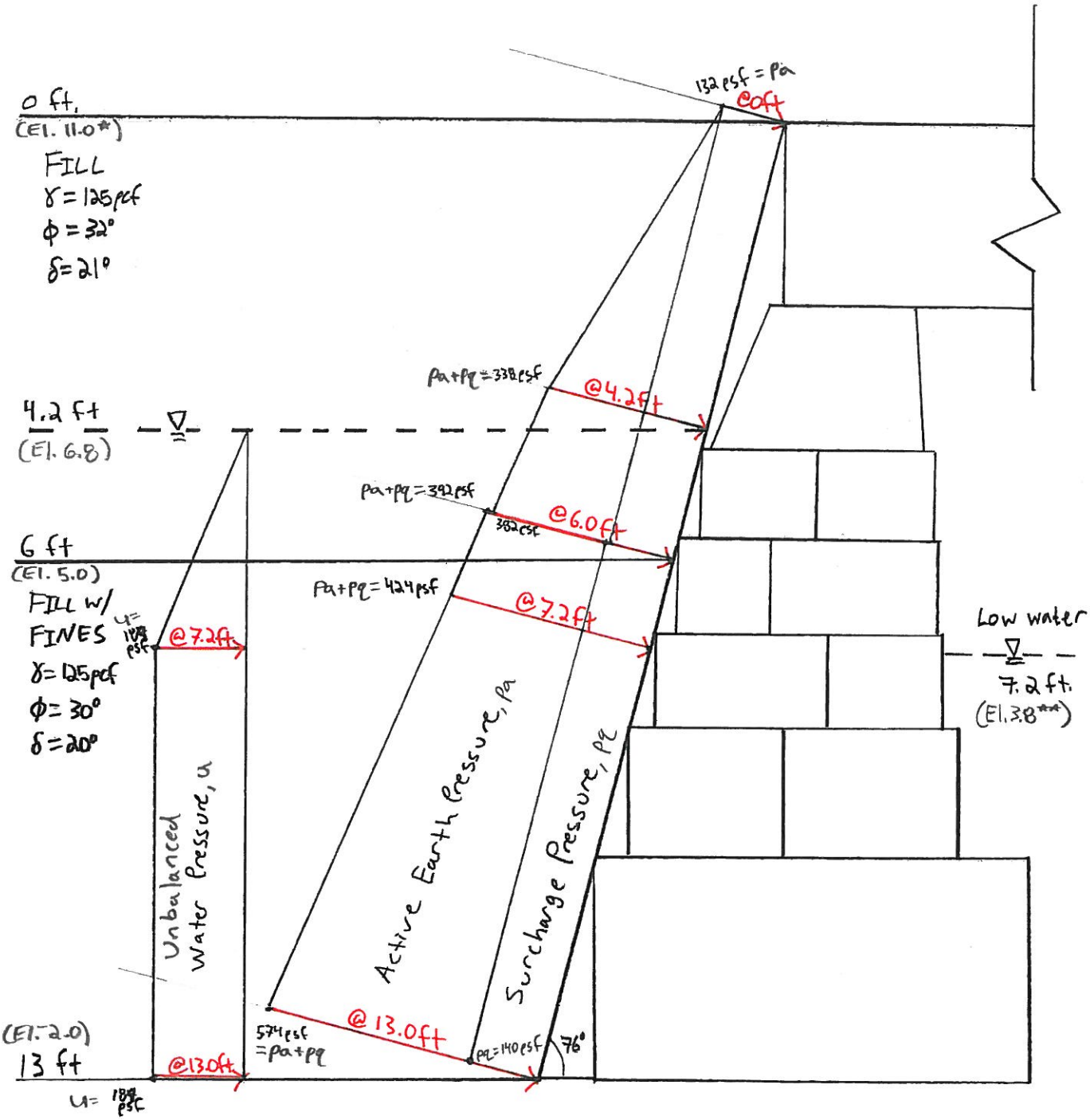
SUBJECT	East Abutment - At-Rest Earth Pressures		
Job No. 1668741	Made By: CJS	Date: 7/5/17	
Ref.	Checked: MCM	Sheet 3 of 16	
	Reviewed: WSP		



* Ground surface elevation based on base plan provided by VHB dated December 13, 2016 (NAVD 1988 Datum).

** "Low water" as described on 1931 design plans. If current extreme low water elevation is lower, it should be used for design

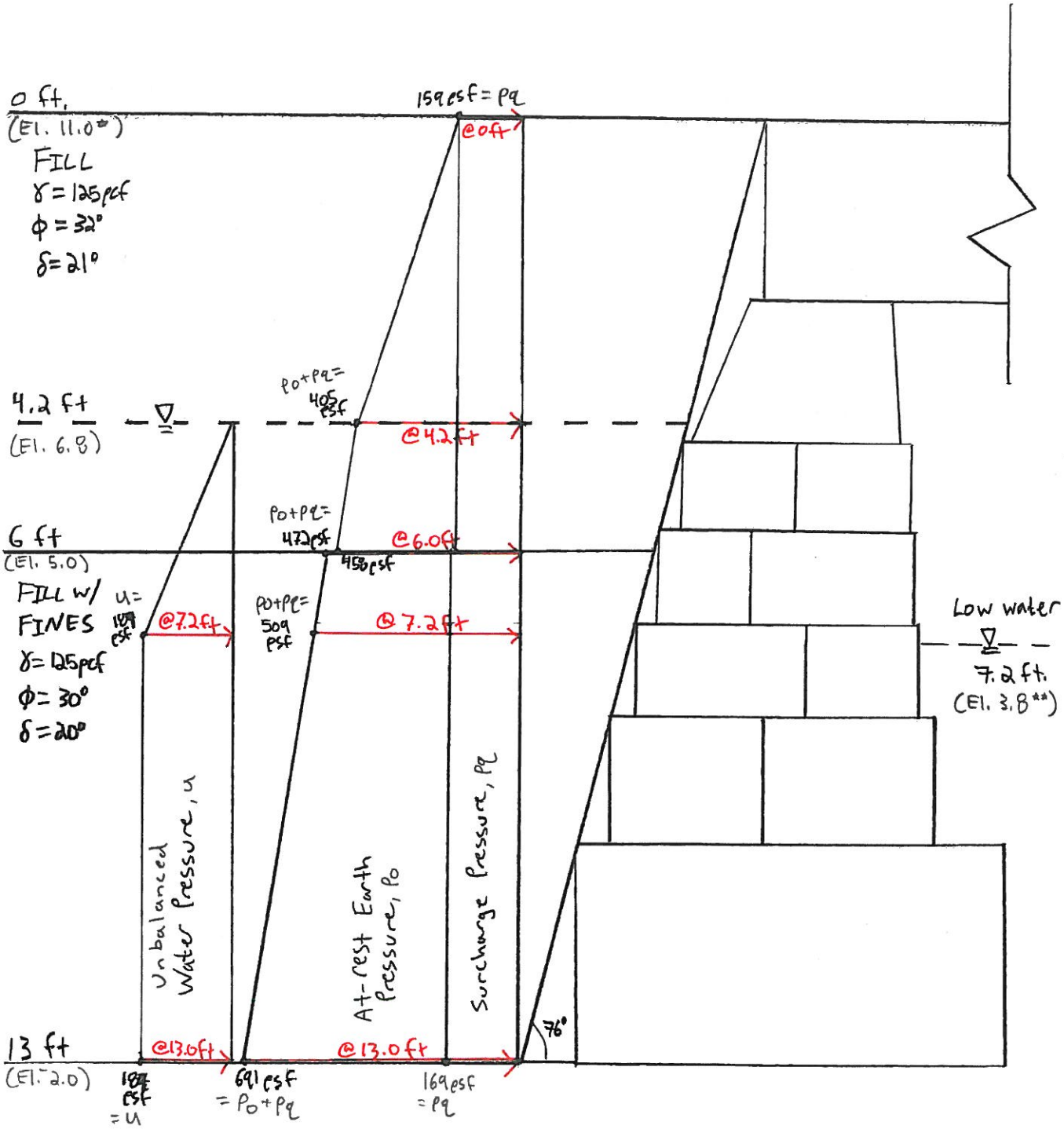
Golder Associates, Manchester, New Hampshire	SUBJECT: West Abutment – Active Earth Pressures		
Job No. 166741	Made By: CJS	Date: 7/5/17	
Ref.	Checked: MCV	Sheet 4 of 16	
	Reviewed: MSP		



* Ground surface elevation based on base plan provided by VHB dated December 13, 2016. (NAVD 1988 Datum)

** "Low water" as described on 1931 design Plans. If current extreme low water elevation is lower, it should be used for design.

SUBJECT	West Abutment At-Rest Earth Pressures		
Job No. 1668741	Made By: CJS	Date: 7/5/17	
Ref.	Checked: mcm	Sheet 5 of 16	
	Reviewed: MSP		



* Ground surface elevation based on base plan provided by VHB dated December 13, 2016 (NAVD 1988 Datum).

** "Low water" as described on 1931 design plans. If current extreme low water elevation is lower, it should be used for design.

Golder Associates

CALCULATIONS

Date:	7/6/2017	**Made by:**	CJS
Project No.:	1668741	**Checked by:**	TRM / MCM
Subject:	Earth Pressures on Existing Abutments	**Reviewed by:**	MSP
Project Short Title:	VHB/Marsh Bridge Replacement		

1.0 PURPOSE

To provide recommended earth pressures on the existing abutment walls for the proposed Marsh Bridge replacement in the towns of Southwest Harbor and Tremont, ME.

2.0 APPROACH

Use LRFD methods to calculate earth pressures acting on the existing abutment walls.

3.0 REFERENCES

1. AASHTO LRFD Bridge Design Specifications, 7th Ed, (2014).
2. Maine Department of Transportation (2003 with updates to 2014). "Bridge Design Guide," Augusta, ME.
3. 1931 Maine Highway Commision Bridge Division design plans for Marsh Bridge over Marshall Brook between the towns of Southwest Harbor and Tremont, Hancock County.

4.0 ATTACHMENTS

1. Design profiles

5.0 ASSUMPTIONS

1. Both Abutments are 13 feet tall (i.e. road surface to the bottom of the pile cap, assuming no passive resistance in front of the wall.
2. Wall is installed with a back batter of 3"/foot (θ = 76 degrees from the horizontal).
3. Assume the fill has a friction angle of 32 degrees with a unit weight of 125pcf based on MaineDOT Granular Underwater Backfill (Table 3-3, Reference 2).
4. Assume that the abutments were backfilled with controlled fill material throughout the entire depth of the wall, but have since been "contaminated" with migration of fines. Account for this by lowering the friction angle and use: friction angle of 30 degrees with a unit weight of 125pcf (i.e. lower strenght Granular Underwater Backfill) for soils adjacent layers containing appreciable fines.
5. Soil-wall friction angles are as follows:

Fill: δ =	21	deg	(assume 2/3ϕ)
Fill w/ Fines: δ =	20	deg	(assume 2/3ϕ)

6. Based on AASHTO 3.11.6.4, a 13ft wall requires a 2.7ft equivalent live load soil surcharge applied to the road surface. (2.7ft of soil with a 125pcf unit weight = 338 psf)
7. Marshall Brook is at 7.2ft below the road surface based on Reference 3 (Ground surface at El. 85.04 and "reported low" water at El. 77.85). The current "extreme low" water elevation is unknown. If the results fo H&H studies by Northstar Hydro, Inc. (for VHB) indicate the extreme low water elevation is lower than 7.2 ft bgs, the assumed water level in Marshall Brook should be lowered and these earth pressure calculations should be revised. Based on the Bridge Design Guide (Reference 2), a 3 ft unbalanced water load should be accounted for at walls located along a river. Therefore the water table level behind the wall should be 4.2ft below the road surface.
8. This calcuation evaluates both active and at rest earth pressures acting against both the east and west abutments because the abutment connection to the proposed bridge structure is currently unknown. Active pressure coefficients (k_a) are calculated using Coulomb Earth Pressure Theory per AASHTO 3.11.5.3 and at-rest coefficients (k_o) are calculated using AASHOT 3.11.5.2-2.

6.0 CALCULATION

1) Calculate Active Earth and Surcharge Pressures

Determine the active pressures acting against the wall.

$$p_a = \gamma_s' H k_a \quad \text{(psf)} \qquad P_a = \tfrac{1}{2} k_a \times \gamma_s' \times H^2 = \tfrac{1}{2} p_a \times H \quad \text{(lb/ft)}$$

where:

γ_s' = effective unit weight of soil (pcf)
H = soil layer thickness (ft)
k_a = active earth pressure coefficient (Coulomb)

$$k_a = \frac{\sin^2(\theta + \phi_f')}{\Gamma[\sin^2(\theta)\sin(\theta - \delta)]} \qquad \text{(Ref. 1, Eq. 3.11.5.3-1)}$$

$$\Gamma = \left[1 + \sqrt{\frac{\sin(\phi_f' + \delta)\sin(\phi_f' - \beta)}{\sin(\theta - \delta)\sin(\theta + \beta)}}\right]^2 \qquad \text{(Ref. 1, Eq. 3.11.5.3-2)}$$

where:

δ = soil-wall friction angle (deg)
β = soil backslope angle (deg)
θ = wall inclination (deg)
ϕ_f' = effective friction angle of soil layer (deg)

Determine the surcharge pressures acting against the wall

$$p_q = k_a q \quad \text{(psf)} \qquad P_q = k_a \times q \times H = p_q \times H \quad \text{(lb/ft)}$$

where:

k_a = active earth pressure coefficient (Coulomb)
q = 338 surcharge pressure psf, 2.7ft x γ_s per AASHTO 3.11.6.4

Calculate the total combined active and surcharge pressure with depth

$$p_{a+q} = p_a + p_q \quad \text{(psf)}$$

Section 11. In no case shall highly plastic clay be used for backfill.

For walls retaining cohesive materials, the effects of soil creep should be taken into consideration in estimating the design earth pressures. Evaluation of soil creep is complex and requires duplication in the laboratory of the stress conditions in the field as discussed by Mitchell (1976).

Under stress conditions close to the minimum active or maximum passive earth pressures, cohesive soils indicated in Table C3.11.1-1 creep continually, and the movements shown produce active or passive pressures only temporarily. If there is no further movement, active pressures will increase with time, approaching the at-rest pressure, and passive pressures will decrease with time, approaching values on the order of 40 percent of the maximum short-term value. A conservative assumption to account for unknowns would be to use the at-rest pressure based on the residual strength of the soil.

3.11.2—Compaction

Where activity by mechanical compaction equipment is anticipated within a distance of one-half the height of the wall, taken as the difference in elevation between the point where finished grade intersects the back of the wall and the base of the wall, the effect of additional earth pressure that may be induced by compaction shall be taken into account.

C3.11.2

Compaction-induced earth pressures may be estimated using the procedures described by Clough and Duncan (1991). The heavier the equipment used to compact the backfill, and the closer it operates to the wall, the larger are the compaction-induced pressures. The magnitude of the earth pressures exerted on a wall by compacted backfill can be minimized by using only small rollers or hand compactors within a distance of one-half wall height from the back of the wall. For MSE structures, compaction stresses are already included in the design model and specified compaction procedures.

3.11.3—Presence of Water

If the retained earth is not allowed to drain, the effect of hydrostatic water pressure shall be added to that of earth pressure.

In cases where water is expected to pond behind a wall, the wall shall be designed to withstand the hydrostatic water pressure plus the earth pressure.

Submerged unit weights of the soil shall be used to determine the lateral earth pressure below the groundwater table.

C3.11.3

The effect of additional pressure caused by groundwater is shown in Figure C3.11.3-1.

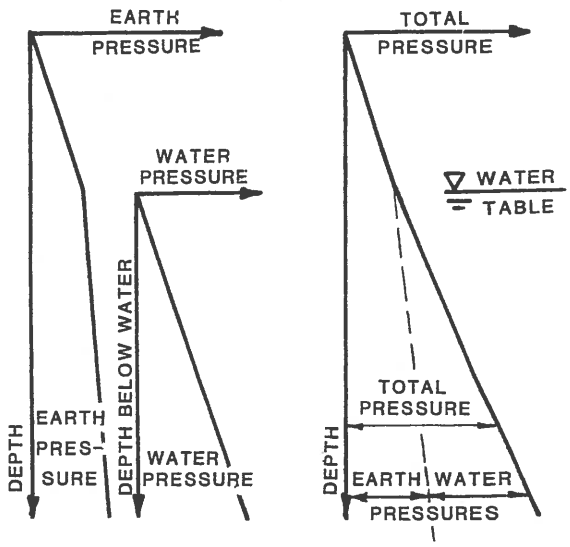


Figure C3.11.3-1—Effect of Groundwater Table

If the groundwater levels differ on opposite sides of the wall, the effects of seepage on wall stability and the potential for piping shall be considered. Pore water pressures shall be added to the effective horizontal stresses in determining total lateral earth pressures on the wall.

The development of hydrostatic water pressure on walls should be eliminated through use of crushed rock, pipe drains, gravel drains, perforated drains or geosynthetic drains.

Pore water pressures behind the wall may be approximated by flow net procedures or various analytical methods.

3.11.4—Effect of Earthquake

The effects of wall inertia and probable amplification of active earth pressure and/or mobilization of passive earth masses by earthquake shall be considered.

C3.11.4

The Mononobe-Okabe method for determining equivalent static fluid pressures for seismic loads on gravity and semigravity retaining walls is presented in Appendix A11.

The Mononobe-Okabe analysis is based, in part, on the assumption that the backfill soils are unsaturated and thus, not susceptible to liquefaction.

Where soils are subject to both saturation and seismic or other cyclic/instantaneous loads, special consideration should be given to address the possibility of soil liquefaction.

3.11.5—Earth Pressure: *EH*

3.11.5.1—Lateral Earth Pressure

Lateral earth pressure shall be assumed to be linearly proportional to the depth of earth and taken as:

$$p = k\gamma_s z \qquad (3.11.5.1\text{-}1)$$

where:

p = lateral earth pressure (ksf)
k = coefficient of lateral earth pressure taken as k_o, specified in Article 3.11.5.2, for walls that do not deflect or move, k_a, specified in Articles 3.11.5.3, 3.11.5.6 and 3.11.5.7, for walls that deflect or move sufficiently to reach minimum active conditions, or k_p, specified in Article 3.11.5.4, for walls that deflect or move sufficiently to reach a passive condition
γ_s = unit weight of soil (kcf)
z = depth below the surface of earth (ft)

The resultant lateral earth load due to the weight of the backfill shall be assumed to act at a height of $H/3$ above the base of the wall, where H is the total wall height, measured from the surface of the ground at the back of the wall to the bottom of the footing or the top of the leveling pad (for MSE walls).

C3.11.5.1

Although previous versions of these Specifications have required design of conventional gravity walls for a resultant earth pressure located $0.4H$ above the wall base, the current specifications require design for a resultant located $H/3$ above the base. This requirement is consisten with historical practice and with calibrated resistance factors in Section 11. The resultant lateral load due to the

The resultant lateral earth load due to the weight of the backfill shall be assumed to act at a height of $H/3$ above the base of the wall, where H is the total wall height, measured from the surface of the ground at the back of the wall to the bottom of the footing or the top of the leveling pad (for MSE walls).

earth pressure may act as high as $0.4H$ above the base of the wall for a mass concrete gravity retaining wall, where H is the total wall height measured from the top of the backfill to the base of the footing, where the wall deflects laterally, i.e., translates, in response to lateral earth loading. For such structures, the backfill behind the wall must slide down along the back of the wall for the retained soil mass to achieve the active state of stress. Experimental results indicate that the backfill arches against the upper portion of the wall as the wall translates, causing an upward shift in the location at which the resultant of the lateral earth load is transferred to the wall (Terzaghi, 1934; Clausen and Johansen et al., 1972; Sherif et al., 1982). Such walls are not representative of typical gravity walls used in highway applications.

For most gravity walls which are representative of those used in highway construction, nongravity cantilever retaining walls or other flexible walls which tilt or deform laterally in response to lateral loading, e.g., MSE walls, as well as walls which cannot translate or tilt, e.g., integral abutment walls, significant arching of the backfill against the wall does not occur, and the resultant lateral load due to earth pressure acts at a height of $H/3$ above the base of the wall. Furthermore, where wall friction is not considered in the analysis, it is sufficiently conservative to use a resultant location of $H/3$ even if the wall can translate.

3.11.5.2—At-Rest Lateral Earth Pressure Coefficient, k_o

For normally consolidated soils, vertical wall, and level ground, the coefficient of at-rest lateral earth pressure may be taken as:

$$k_o = 1 - \sin \phi'_f \qquad (3.11.5.2\text{-}1)$$

where:

ϕ'_f = effective friction angle of soil
k_o = coefficient of at-rest lateral earth pressure

For overconsolidated soils, the coefficient of at-rest lateral earth pressure may be assumed to vary as a function of the overconsolidation ratio or stress history, and may be taken as:

$$k_o = (1 - \sin \phi'_f)\,(OCR)^{\sin \phi'_f} \qquad (3.11.5.2\text{-}2)$$

where:

OCR = overconsolidation ratio

Silt and lean clay shall not be used for backfill unless suitable design procedures are followed and construction control measures are incorporated in the construction documents to account for their presence. Consideration must be given for the development of pore water pressure

C3.11.5.2

For typical cantilevered walls over 5.0 ft high with structural grade backfill, calculations indicate that the horizontal movement of the top of the wall due to a combination of structural deformation of the stem and rotation of the foundation is sufficient to develop active conditions.

In many instances, the OCR may not be known with enough accuracy to calculate k_o using Eq. 3.11.5.2-2. Based on information on this issue provided by Holtz and Kovacs (1981), in general, for lightly overconsolidated sands (OCR = 1 to 2), k_o is in the range of 0.4 to 0.6. For highly overconsolidated sand, k_o can be on the order of 1.0.

The evaluation of the stress induced by cohesive soils is highly uncertain due to their sensitivity to shrink-swell, wet-dry and degree of saturation. Tension cracks can form, which considerably alter the assumptions for the estimation of stress. Extreme caution is advised in the determination

within the soil mass in accordance with Article 3.11.3. Appropriate drainage provisions shall be provided to prevent hydrostatic and seepage forces from developing behind the wall in accordance with the provisions of Section 11. In no case shall highly plastic clay be used for backfill.

3.11.5.3—Active Lateral Earth Pressure Coefficient, k_a

Values for the coefficient of active lateral earth pressure may be taken as:

$$k_a = \frac{\sin^2(\theta + \phi'_f)}{\Gamma[\sin^2\theta \sin(\theta - \delta)]} \qquad (3.11.5.3\text{-}1)$$

in which:

$$\Gamma = \left[1 + \sqrt{\frac{\sin(\phi'_f + \delta)\sin(\phi'_f - \beta)}{\sin(\theta - \delta)\sin(\theta + \beta)}}\right]^2 \qquad (3.11.5.3\text{-}2)$$

where:

δ = friction angle between fill and wall taken as specified in Table 3.11.5.3-1 (degrees)
β = angle of fill to the horizontal as shown in Figure 3.11.5.3-1 (degrees)
θ = angle of back face of wall to the horizontal as shown in Figure 3.11.5.3-1 (degrees)
ϕ'_f = effective angle of internal friction (degrees)

For conditions that deviate from those described in Figure 3.11.5.3-1, the active pressure may be calculated by using a trial procedure based on wedge theory using the Culmann method (e.g., see Terzaghi et al., 1996).

of lateral earth pressures assuming the most unfavorable conditions. See Article C3.11.1 for additional guidance on estimating earth pressures in fine-grained soils. If possible, cohesive or other fine-grained soils should be avoided as backfill.

C3.11.5.3

The values of k_a by Eq. 3.11.5.3-1 are based on the Coulomb earth pressure theories. The Coulomb theory is necessary for design of retaining walls for which the back face of the wall interferes with the development of the full sliding surfaces in the backfill soil assumed in Rankine theory (Figure C3.11.5.3-1 and Article C3.11.5.8). Either Coulomb or Rankine wedge theory may be used for long heeled cantilever walls shown in Figure C3.11.5.3-1a. In general, Coulomb wedge theory applies for gravity, semigravity and prefabricated modular walls with relatively steep back faces, and concrete cantilever walls with short heels.

For the cantilever wall in Figure C3.11.5.3-1b, the earth pressure is applied to a plane extending vertically up from the heel of the wall base, and the weight of soil to the left of the vertical plane is considered as part of the wall weight.

The differences between the Coulomb theory currently specified, and the Rankine theory specified in the past is illustrated in Figure C3.11.5.3-1. The Rankine theory is the basis of the equivalent fluid method of Article 3.11.5.5.

Silt and lean clay should not be used for backfill where free-draining granular materials are available. When using poorly draining silts or cohesive soils, extreme caution is advised in the determination of lateral earth pressures assuming the most unfavorable conditions. Consideration must be given for the development of pore water pressure within the soil mass in accordance with Article 3.11.3. Appropriate drainage provisions should be provided to prevent hydrostatic and seepage forces from developing behind the wall in accordance with the provisions in Section 11. In no case should highly plastic clay be used for backfill.

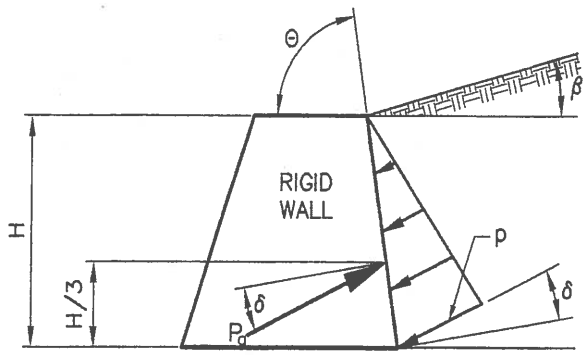


Figure 3.11.5.3-1—Notation for Coulomb Active Earth Pressure

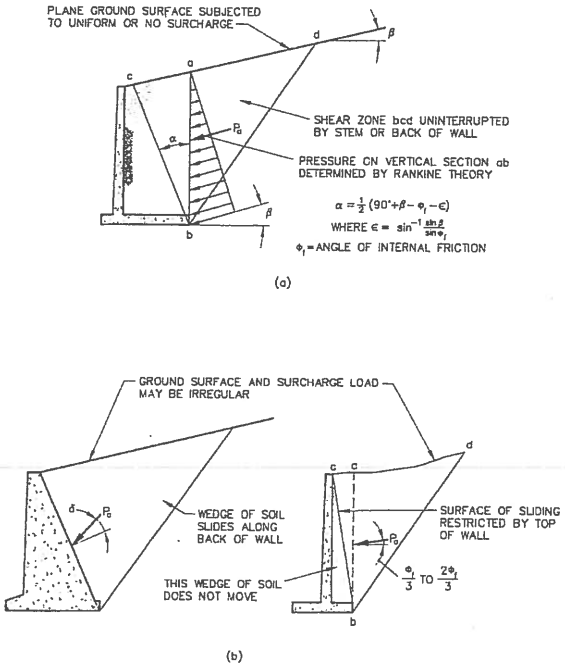


Figure C3.11.5.3-1—Application of (a) Rankine and (b) Coulomb Earth Pressure Theories in Retaining Wall Design

Table 3.11.5.3-1—Friction Angle for Dissimilar Materials (U.S. Department of the Navy, 1982a)

Interface Materials	Friction Angle, δ (degrees)	Coefficient of Friction, tan δ (dim.)
Mass concrete on the following foundation materials:		
• Clean sound rock	35	0.70
• Clean gravel, gravel-sand mixtures, coarse sand	29 to 31	0.55 to 0.60
• Clean fine to medium sand, silty medium to coarse sand, silty or clayey gravel	24 to 29	0.45 to 0.55
• Clean fine sand, silty or clayey fine to medium sand	19 to 24	0.34 to 0.45
• Fine sandy silt, nonplastic silt	17 to 19	0.31 to 0.34
• Very stiff and hard residual or preconsolidated clay	22 to 26	0.40 to 0.49
• Medium stiff and stiff clay and silty clay	17 to 19	0.31 to 0.34
Masonry on foundation materials has same friction factors.		
Steel sheet piles against the following soils:		
• Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls	22	0.40
• Clean sand, silty sand-gravel mixture, single-size hard rock fill	17	0.31
• Silty sand, gravel or sand mixed with silt or clay	14	0.25
• Fine sandy silt, nonplastic silt	11	0.19
Formed or precast concrete or concrete sheet piling against the following soils:		
• Clean gravel, gravel-sand mixture, well-graded rock fill with spalls	22 to 26	0.40 to 0.49
• Clean sand, silty sand-gravel mixture, single-size hard rock fill	17 to 22	0.31 to 0.40
• Silty sand, gravel or sand mixed with silt or clay	17	0.31
• Fine sandy silt, nonplastic silt	14	0.25
Various structural materials:		
• Masonry on masonry, igneous and metamorphic rocks:		
o dressed soft rock on dressed soft rock	35	0.70
o dressed hard rock on dressed soft rock	33	0.65
o dressed hard rock on dressed hard rock	29	0.55
• Masonry on wood in direction of cross grain	26	0.49
• Steel on steel at sheet pile interlocks	17	0.31

3.11.5.4—Passive Lateral Earth Pressure Coefficient, k_p

For noncohesive soils, values of the coefficient of passive lateral earth pressure may be taken from Figure 3.11.5.4-1 for the case of a sloping or vertical wall with a horizontal backfill or from Figure 3.11.5.4-2 for the case of a vertical wall and sloping backfill. For conditions that deviate from those described in Figures 3.11.5.4-1 and 3.11.5.4-2, the passive pressure may be calculated by using a trial procedure based on wedge theory, e.g., see Terzaghi et al. (1996). When wedge theory is used, the limiting value of the wall friction angle should not be taken larger than one-half the angle of internal friction, ϕ_f.

For cohesive soils, passive pressures may be estimated by:

C3.11.5.4

The movement required to mobilize passive pressure is approximately 10.0 times as large as the movement needed to induce earth pressure to the active values. The movement required to mobilize full passive pressure in loose sand is approximately five percent of the height of the face on which the passive pressure acts. For dense sand, the movement required to mobilize full passive pressure is smaller than five percent of the height of the face on which the passive pressure acts, and five percent represents a conservative estimate of the movement required to mobilize the full passive pressure. For poorly compacted cohesive soils, the movement required to mobilize full passive pressure is larger than five percent of the height of the face on which the pressure acts.

$$\Delta_p = k\gamma_s h_{eq} \qquad (3.11.6.4\text{-}1)$$

where:

Δ_p = constant horizontal earth pressure due to live load surcharge (ksf)
γ_s = total unit weight of soil (kcf)
k = coefficient of lateral earth pressure
h_{eq} = equivalent height of soil for vehicular load (ft)

Equivalent heights of soil, h_{eq}, for highway loadings on abutments and retaining walls may be taken from Tables 3.11.6.4-1 and 3.11.6.4-2. Linear interpolation shall be used for intermediate wall heights.

The wall height shall be taken as the distance between the surface of the backfill and the bottom of the footing along the pressure surface being considered.

Table 3.11.6.4-1—Equivalent Height of Soil for Vehicular Loading on Abutments Perpendicular to Traffic

Abutment Height (ft)	h_{eq} (ft)
5.0	4.0
10.0	3.0
≥20.0	2.0

Table 3.11.6.4-2—Equivalent Height of Soil for Vehicular Loading on Retaining Walls Parallel to Traffic

Retaining Wall Height (ft)	h_{eq} (ft) Distance from wall backface to edge of traffic	
	0.0 ft	1.0 ft or Further
5.0	5.0	2.0
10.0	3.5	2.0
≥20.0	2.0	2.0

The load factor for both vertical and horizontal components of live load surcharge shall be taken as specified in Table 3.4.1-1 for live load surcharge.

3.11.6.5—Reduction of Surcharge

If the vehicular loading is transmitted through a structural slab, which is also supported by means other than earth, a corresponding reduction in the surcharge loads may be permitted.

- Poisson's ratio for the pavement and subgrade materials are 0.2 and 0.4, respectively
- Wheel loads were modeled as a finite number of point loads distributed across the tire area to produce an equivalent tire contact stress
- The process for equating wall moments resulting from the elastic solution with the equivalent surcharge method used a wall height increment of 0.25 ft.

The value of the coefficient of lateral earth pressure k is taken as k_o, specified in Article 3.11.5.2, for walls that do not deflect or move, or k_a, specified in Articles 3.11.5.3, 3.11.5.6 and 3.11.5.7, for walls that deflect or move sufficiently to reach minimum active conditions.

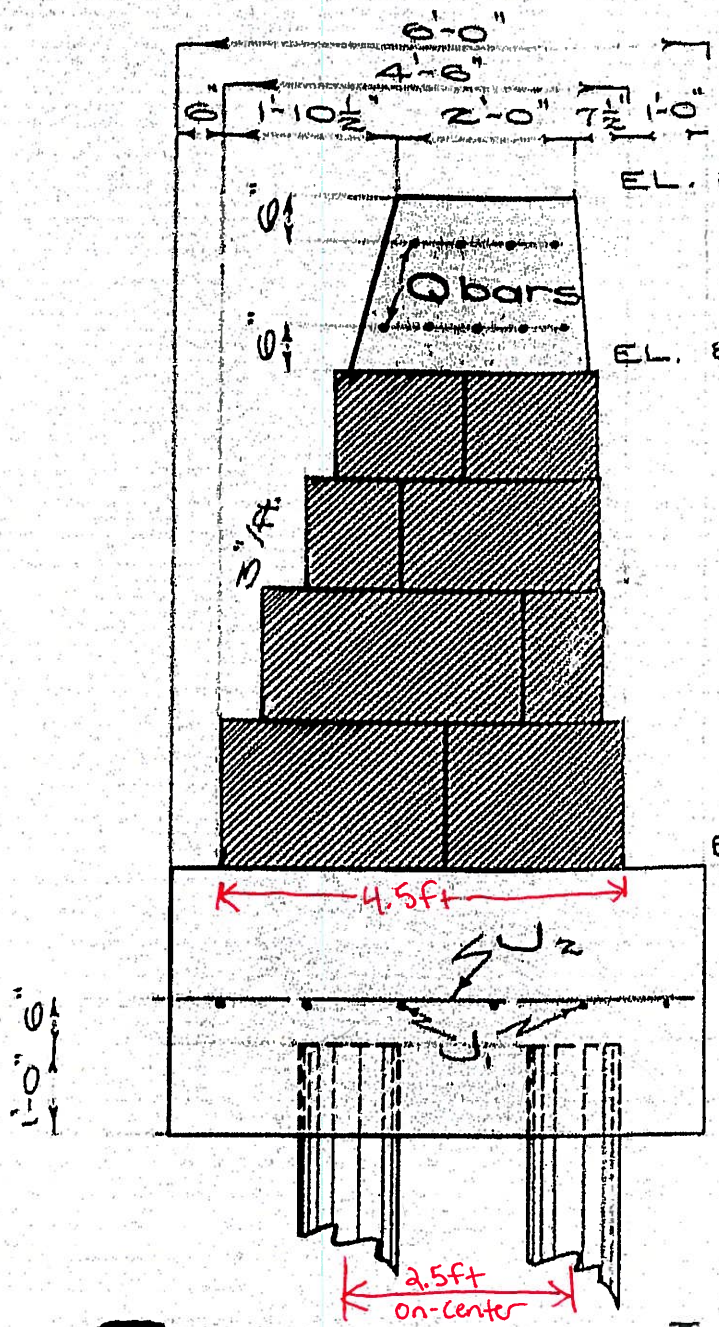
The analyses used to develop Tables 3.11.6.4-1 and 3.11.6.4-2 are presented in Kim and Barker (1998).

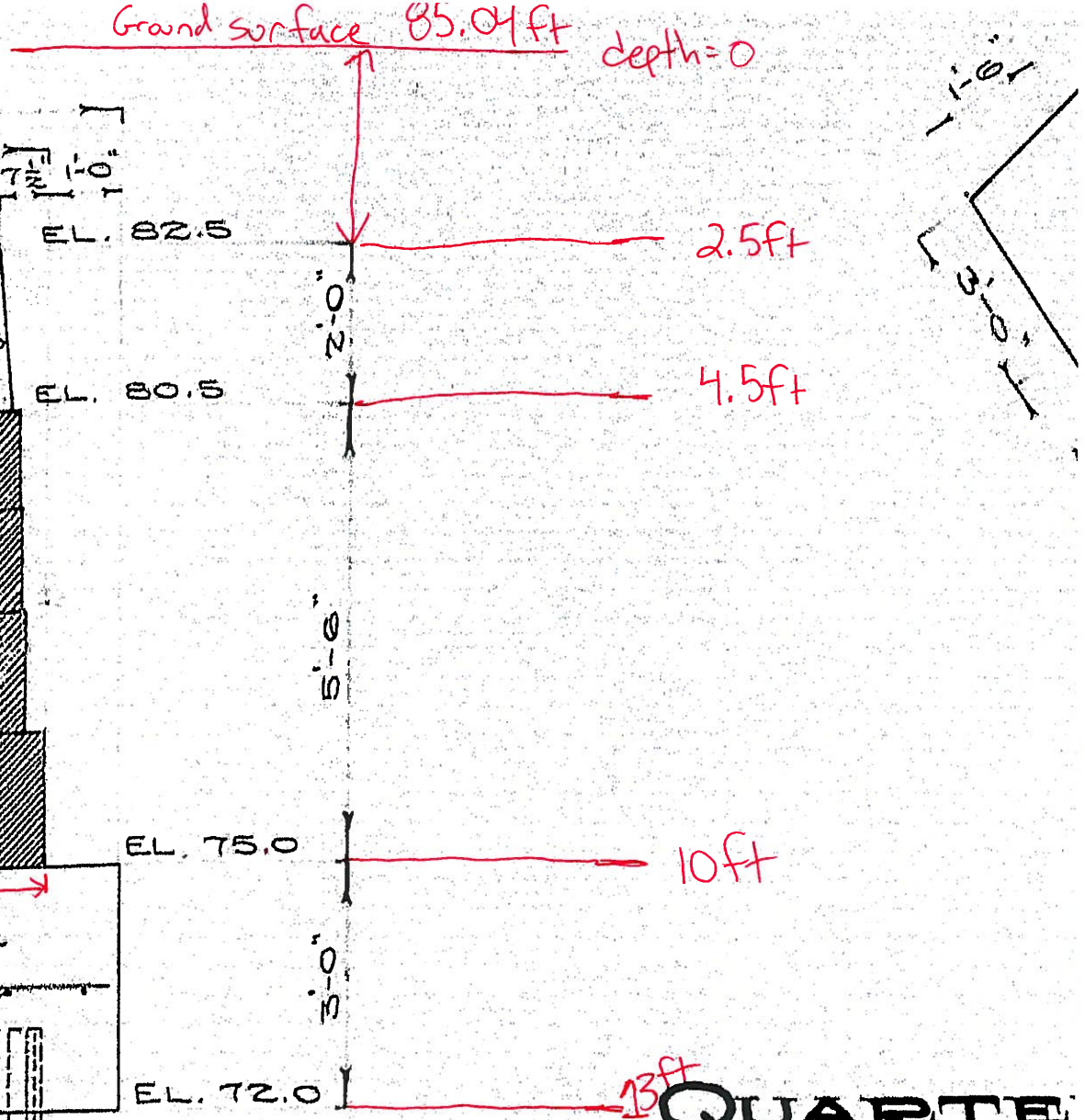
The values for h_{eq} given in Tables 3.11.6.4-1 and 3.11.6.4-2 are generally greater than the traditional 2.0 ft of earth load historically used in the AASHTO specifications, but less than those prescribed in previous editions (i.e., before 1998) of this specification. The traditional value corresponds to a 20.0-kip single unit truck formerly known as an H10 truck, Peck et al. (1974). This partially explains the increase in h_{eq} in previous editions of this specification. Subsequent analyses, i.e., Kim and Barker (1998) show the importance of the direction of traffic, i.e., parallel for a wall and perpendicular for an abutment on the magnitude of h_{eq}. The magnitude of h_{eq} is greater for an abutment than for a wall due to the proximity and closer spacing of wheel loads to the back of an abutment compared to a wall.

The backface of the wall should be taken as the pressure surface being considered. Refer to Article C11.5.5 for application of surcharge pressures on retaining walls.

C3.11.6.5

This Article relates primarily to approach slabs which are supported at one edge by the backwall of an abutment, thus transmitting load directly thereto.





SECTION A-A

QUARTE
PILE PL
(48.~12 ton piles
(Both Abuts.)

Note — Q-bars to be placed 6" below the bridge seat and 6" above the masonry spaced 6" on centers. They

APPENDIX E.3

FB-MULTIPLIER LATERAL PILE ANALYSIS FOR EXISTING TIMBER PILES

CALCULATIONS

Date:	11/28/2017	**Made by:**	CJS
Project No.:	1668741	**Checked by:**	CCB
Subject:	FB-Multipier Analysis of Existing Piles	**Reviewed by:**	CCB
Project Short Title:	VHB Marsh Bridge Replacement		

1.0 OBJECTIVE

Check the maximum deflection, moments and axial loads for the existing timber piles under the proposed pile cap loads provided by VHB.

2.0 METHOD

1. Use FB-Multipier to analyze pile response to the loads provided by VHB .

3.0 REFERENCES

1. Email communication from Carl Ayers of VHB. Subject line, "Marsh Bridge revised pile cap loads," received by Golder Associates on November 16, 2017 at 5:56PM.
2. FB-MultiPier software version 5.1.1.
3. Maine Highway Commission Bridge Division bridge design plans titled, "Marsh Bridge over Marshall Brook Between the Towns of Southwest Harbor and Tremont," (1931).
4. ASD Manual for Engineered Wood Construction (2003), "Timber Poles and Piles," page PP-13.
5. Chellis, Robert D., (1961), "Pile Foundations," 2ed, McGraw-Hill Book Company, New York.
6. MnDOT (1992), "USFS Timber Bridge Manual," pp 3-18.
7. FB-MultiPier Soil Parameter Table (US Customary Unit) (2013), Bridge Software Institute, University of Florida.
8. Geotesting Express laboratory data dated 6/8/2017.

4.0 ATTACHMENTS

1. Interpreted subsurface profile.
2. Program Input Summary Table
3. FB-MultiPier output summary table
4. FB-Multipier program output

5.0 ASSUMPTIONS

1. The installed size of the existing piles is unknown. Typical MEDOT construction practices of the era indicate that tapered piles with a butt diameter of 10-inches and a tip diameter of 8-inches were common. Golder's model assumes that the entire pile has a diameter of 8-inches which accounts for some section loss at the top of pile where tidal effects are more significant. The pile spacing is as shown in Reference 3 and conservatively does not account for resistance provided by the "winged" portions of the pile caps.
2. Loads provided VHB as shown in the calculation below. Based on Golder's discussions with VHB, loads are for at-rest earth pressures with no differential water pressure. Golder's understanding is that VHB has assumed the tidal changes are sufficiently slow as to not cause build up of differential head at the wall.
3. Subsurface layering is as shown on Attachment 1. Assume that the top of pile cap is embedded 2.5ft below the bottom of the stream channel.
4. Water elevation is assumed to be at the bottom of streambed elevation (i.e. 2.5ft above the pile cap as described in Assumption #3).
5. The pile spacing is variable, but it was assumed that p-y multipliers of 0.95 and 0.57 would be applied for the first and second rows respectively (based on LRFD recommendations).

Date:	11/28/2017	**Made by:**	CJS
Project No.:	1668741	**Checked by:**	CCB
Subject:	FB-Multipier Analysis of Existing Piles	**Reviewed by:**	CCB
Project Short Title:	VHB Marsh Bridge Replacement		

6.0 INPUT PARAMETERS

Pile Parameters:

Diameter	8	inches
Young's Modulus (E)	1250	ksi, (Red-Oak, Ref. 4)
Unit Weight	64	pcf, (Ref. 5)
Poisson's Ratio (v)	0.47	(Largest Value Table 3.5 [Sitka Spruce], Ref. 6)

Shear Modulus

$A = \pi r^2$ A = 50.3 in^2

Shear modulus (G) = 100.0 ksi (0.08*E, Ref. 6)

Moments of Inertia

$I_2 = \frac{\pi}{4} r^4$ I_2 = 201.1 in^4

$I_3 = \frac{\pi}{4} r^4$ I_3 = 201.1 in^4

Polar Moment of Inertia

$J = \frac{\pi}{32} D^4$ J = 402.1 in^4

Soil Parameters Parameters:

The soil properties used in the FB-Multipier model are based on recommended properties from Reference 7 and laboratory test results in Reference 8. These input parameters are summarized in Attachment 2. Shear strength of the clay layers are based on the results of field vane testing. Shear modulus of rock is calculated below:

Rock information

E	11500	ksi, [lab data BB-MAR103B, R1]
v	0.21	ksi, [lab data BB-MAR103B, R1]
UCS	4032000	psf, [lab data BB-MAR103B, R1]

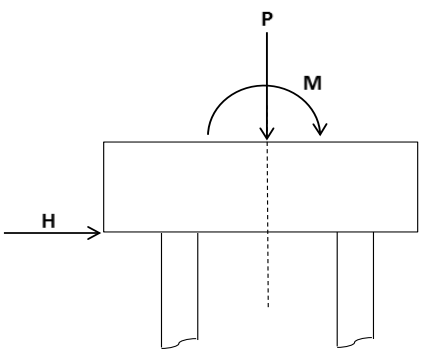
Shear Modulus (rock)

$G_s = \frac{E}{2(1+v)}$ 4752.1 ksi

Load Application:

The loads provided by VHB (Ref. 1) were applied as specified by VHB in the diagram below. Please note that the loads are applied in the program as a line load, but the software resolves the line load to each node over the tributary area of the node.

Date:	11/28/2017	**Made by:**	CJS
Project No.:	1668741	**Checked by:**	CCB
Subject:	FB-Multipier Analysis of Existing Piles	**Reviewed by:**	CCB
Project Short Title:	VHB Marsh Bridge Replacement		



VHB Pile Cap Loads (Ref. 1)

PILE CAP LOADS		P (k/ft)	H (k/ft)	M (k-ft/ft)
At-Rest	STR	18.8	4.8	15.0
	SERV	12.8	3.3	9.9

**Note: loads are provided for a 27.7ft long footing with no unbalanced water force.*

7.0 Analyses

Run an analysis on the existing 8-inch timber piles at the east abutment (worst case based on Golder's preliminary analysis completed in August 2017) using at-rest earth pressures and the loads provided by VHB.

8.0 Conclusion

The full results of the FB-MultiPier Analysis are Summarized in Attachment 3. Program output files are provided as Attachments 4.

A

West

A'

East

BB-MAR-101
7' LT

BB-MAR-201B
8' LT

BB-MAR-201A
9' LT

BB-MAR-102
7' RT

BB-MAR-103A/B
6.5' LT

BB-MAR-203A
32.0' LT

BB-MAR-205
30.0' RT

BB-MAR-203B
32.0' LT

BB-MAR-104
6.5' LT

BB-MAR-204
32' LT

BB-MAR-202
6.5' RT

EXISTING BRIDGE

ABUTMENT 1

ABUTMENT 2

(EDGE OF WATERWAY ELEVATION)

8" OF ASPHALT

8" OF ASPHALT

FILL - BROWN, DAMP, LOOSE TO MEDIUM DENSE, FINE SAND, SOME GRAVEL, SOME SILT

GRAY, MOIST, SOFT TO VERY STIFF, SILTY CLAY, TRACE FINE SAND, LOW PLASTICITY, PRESUMPSCOT FORMATION

PALE BROWN TO LIGHT BLUISH GRAY TO DARK BLUISH GRAY, FINE TO MEDIUM GRAINED, FRESH TO SLIGHTLY WEATHERED, STRONG, GRANITE, CASTINE FORMATION

PALE RED, FINE TO MEDIUM GRAINED, FRESH TO SLIGHTLY WEATHERED, STRONG, GRANITE, CASTINE FORMATION

EXISTING GRANITE BLOCK ABUTMENTS (REF 4)

EXISTING CONCRETE PILE CAP (REF 4)

EXISTING TIMBER PILE FOUNDATIONS ASSUMED DRIVEN TO BEDROCK REFUSAL (REF 4)

BEDROCK

FILL - BROWN, DAMP, LOOSE TO MEDIUM DENSE, FINE SAND, SOME GRAVEL, SOME SILT

DARK BROWN MOIST TO WET, SOFT TO MEDIUM STIFF, ORGANIC SILTY CLAY AND PEAT WITH FINE SAND

GRAY, MOIST TO WET, VERY SOFT TO SOFT, SILTY CLAY, TRACE FINE SAND, LOW-MEDIUM PLASTICITY, PRESUMPSCOT FORMATION

TILL - GRAY, MOIST TO WET, MEDIUM DENSE, GRAVEL, LITTLE SILT, TRACE CLAY

DARK GRAY TO LIGHT BLUISH GRAY, FINE TO MEDIUM GRAINED, FRESH TO SLIGHTLY WEATHERED, STRONG, GRANITE, CASTINE FORMATION

RUN1 - 42%
RUN2 - 70%
EOB 18.1 FT BGS

EOB 15.0 FT BGS

RUN1 - 35%
RUN2 - 57%
EOB 18.0 FT BGS

EOB 29.6 FT BGS

RUN1 - 87.5%
RUN2 - 55%
EOB 42.4 FT BGS

EOB 29 FT BGS

EOB 38.3 FT BGS

EOB 18 FT BGS

EOB 43.3 FT BGS

EOB 16 FT BGS

RUN1 - 0%
RUN2 - 100%
RUN3 - 100%
EOB 60.5 FT BGS

ELEVATION (FT)

STATION (FT)

14+50 15+00 15+50 16+00 16+50 17+00

50 40 30 20 10 0 -10 -20 -30 -40 -50 -60

LEGEND

- BB-MAR-101 / 7FT RT — BORING LOCATION I.D. / OFFSET FROM CENTERLINE
- WOH / WOR — WEIGHT OF HAMMER / WEIGHT OF RODS
- 10 — SPT : N - VALUE
- 687 / 632 — FIELD VANE SHEAR TEST LOCATION AND UNDRAINED SHEAR STRENGTH (PSF)
- GROUNDWATER ELEVATION (FT)
- RUN1 - 42% — ROCK CORE RUN NUMBER AND RQD
- EOB 18.1 FT BGS — END OF BORING
- ASPHALT
- FILL
- ORGANIC SILTY CLAYS AND PEAT
- PRESUMPSCOT FORMATION
- GLACIAL TILL
- GRANITE, CASTINE FORMATION
- ESTIMATED BEDROCK SURFACE FROM SEISMIC SURVEY HAGER-RICHTER MAY 2017
- ESTIMATED LOCATION OF TIMBER CRIB FROM HAGER-RICHTER SURVEY MAY 2017 & 1931 HISTORICAL DRAWINGS

NOTES

1. THE WATER TABLE IS TIDALLY INFLUENCED. THE WATER TABLE SHOWN IN THIS FIGURE IS APPROXIMATE AND ACTUAL FIELD CONDITIONS WILL VARY.
2. THE 100-SERIES BORING LOCATIONS SHOWN IN THIS FIGURE WERE SURVEYED BY MAINEDOT AND RECEIVED BY GOLDER ON MAY 19, 2017. GOLDER HAS NOT RECEIVED SURVEY DATA FOR THE 200-SERIES BORINGS AT THE WRITING OF THIS REPORT. THE 200-SERIES BORINGS ARE LOCATED BASED ON APPROXIMATE FIELD MEASUREMENTS.
3. SEE BORING LOGS FOR DETAILED LITHOLOGIC DESCRIPTIONS.
4. THIS GENERALIZED SUBSURFACE PROFILE IS INTENDED TO CONVEY TRENDS IN SUBSURFACE CONDITIONS. THE BOUNDARIES BETWEEN STRATA ARE APPROXIMATE AND IDEALIZED, AND HAVE BEEN DEVELOPED BY INTERPRETATIONS OF WIDELY SPACED EXPLORATIONS OF SAMPLES. ACTUAL SOIL AND ROCK TRANSITIONS MAY VARY AND ARE PROBABLY MORE ERRATIC. FOR MORE SPECIFIC INFORMATION REFER TO EXPLORATION LOGS.
5. SEISMIC SURVEY WAS COMPLETED BY HAGER-RICHTER ON MAY 4, 2017 AND WAS SUMMARIZED IN A REPORT TO GOLDER ASSOCIATES ON JUNE 7, 2017.

REFERENCES

1. BASEMAP ELEMENTS FROM VHB DRAWING TITLED "3DTopo_13Dec2016.dgn" DATED ON DECEMBER 13, 2016. RECEIVED BY GOLDER ON APRIL 27, 2017.
2. BORINGS WERE LOCATED AND OBSERVED BY GOLDER AND DRILLED BY NEW ENGLAND BORING CONTRACTORS OF HERMON, MAINE FROM MAY 8-11, 2017 (100-SERIES BORINGS) AND SEPTEMBER 25-29M 2017 (200-SERIES BORINGS).
3. SEISMIC SURVEY DATA PROVIDED IN DRAWING TITLED "Figure 4.dwg" BY HAGER-RICHTER DATED JUNE 7, 2017.
4. EXISTING STRUCTURES ARE APPROXIMATE AND TAKEN FROM MAINE HIGHWAY COMMISSION, MARSH BRIDGE OVER MARSHALL BROOK, SOUTHWEST HARBOR & TREMONT, FEBRUARY 1931.

DRAFT

0 10 20
1" = 20' FEET

CLIENT
VANASSE HANGEN BRUSTLIN, INC.
500 SOUTHBOROUGH DRIVE, SUITE 105B
SOUTH PORTLAND, MAINE 04106

CONSULTANT
Golder Associates

YYYY-MM-DD	2017-05-31
DESIGNED	LLM
PREPARED	RWC
REVIEWED	
APPROVED	

PROJECT
MARSH BRIDGE REPLACEMENT
TREMONT AND SOUTHWEST HARBOR, MAINE
MAINEDOT WIN 21702.00

TITLE
INTERPRETED SUBSURFACE PROFILE A-A'

PROJECT NO.	SUBTITLE	REV.	FIGURE
166-8741	A	0	3

IF THIS MEASUREMENT DOES NOT MATCH WHAT IS SHOWN, THE SHEET SIZE HAS BEEN MODIFIED FROM: ANSI B

0 1 in

Path: \\manchester\cadd\Drawings\2016\166874\1 VHB Marsh Bridge - Tremont, ME\PRODUCTION\ | File Name: 166874A006.dwg | Last Edited By: rclark Date: 2017-11-20 Time:10:01:45 AM | Printed By: rclark Date: 2017-11-20 Time:10:04:23 AM

Attachment 2: Summary of Soil Parameters for FB MultiPier Analysis - Existing Timber Piles
Preliminary Geotechnical Design Report
Marsh Bridge Replacement WIN 21702.00
Tremont & Southwest Harbor, Maine
MaineDOT WIN 21702.00

Layer Description	Top of Layer (Depth BGS, ft)	Recommended Lateral Layer Model	Recommended Axial Layer Model	Recommended Torsional Layer Model	Total Unit Weight (lb/ft^3)	Friction Angle, ϕ	Modulus of Subgrade Reaction, k_{hi} (lb/in^3)	Shear Modulus, G (kip/in^2)	Torsional Shear Stress, τ (kip/ft^2)	Uniaxial Compressive Strength (lb/ft^2)	Undrained Shear Strength (lb/ft^2)	ε_{50}	Poisson's Ratio, v
Soft, Silty CLAY	13	Clay (Soft<Water) Clay(API)	Driven Pile	Hyperbolic	0.115	n/a	n/a	Top: 1.65	0.1	n/a	Top: 900	0.01	Top: 0.45
								Bottom: 0.6			Bottom: 400	0.02	Bottom: 0.4
Soft, Silty CLAY	25.5	Clay (Soft<Water) Clay(API)	Driven Pile	Hyperbolic	0.115	n/a	n/a	0.60	0.1	n/a	400	0.02	0.4
Soft, Silty CLAY	32.5	Clay (Soft<Water) Clay(API)	Driven Pile	Hyperbolic	0.115	n/a	n/a	0.60	0.1	n/a	400	0.02	0.4
GRANITE	34.5	Limestone (McVay)	Driven Pile	Hyperbolic	165	34	n/a	4750	0.7	4032000	n/a	n/a	0.21

Attachment 3 (pg 2 of 2)

FB-MULTIPIER ANALYSIS RESULTS -11/28/2017

(East Abutment - At-Rest Case)

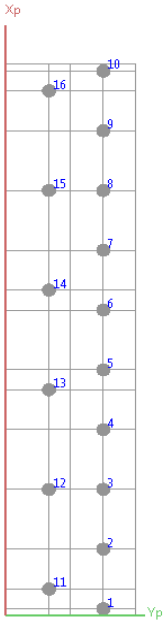
	VHB Loads for 27.7ft Footing		
	zp	yp	mx
8in Pile East Abutment	P(k/ft)	H(k/ft)	M (k-ft/ft)
Load Case 1: At-Rest Strength	18.8	4.8	15.0
Load Case 2: At-Rest Service	12.8	3.3	9.9

Notes:

1) VHB's provided loads for a 27.7ft footing size in the FB Multipier Model

2) The abutment foundation model assumes the timber pile behavior is purely linear elastic. No P-Delta effects are modeled.

3) The timber piles are modeled with an 8-inch diameter for the full length of the piles.



Pile ID	Fixed Connection		
	Load Case 1: At-Rest Strength		Load Case 2: At-Rest Service
	Max Moment(kip-ft)	Max Axial (kip)	Deflection (in)
1	16.9	-37.6	0.397
2	16.9	-38.6	0.397
3	16.9	-39.4	0.397
4	16.9	-39.9	0.398
5	16.9	-40.2	0.398
6	16.9	-40.2	0.398
7	16.9	-39.9	0.398
8	16.9	-39.4	0.397
9	16.9	-38.6	0.397
10	16.9	-37.6	0.397
11	12.9	-11.1	0.397
12	12.9	-12.6	0.397
13	12.9	-13.4	0.398
14	12.9	-13.4	0.398
15	12.9	-12.6	0.397
16	12.9	-11.1	0.397

APPENDIX E.4
GLOBAL STABILITY

Golder Associates

CALCULATIONS

Date:	11/22/2017	**Made by:**	CJS
Project No.:	1668741	**Checked by:**	MCM
Subject:	Global Stability Analysis	**Reviewed by:**	MSP
Project Short Title:	VHB Marsh Bridge Replacement		

1.0 Purpose

Calculate global factor of safety for the proposed roadway widening associated with the Marsh Bridge replacement. Analyze cases for two proposed slope stabilization designs (1.5:1 Rip Rap Slope and reinforced earth retaining wall).

2.0 Method

Use Slide slope stability analysis software to analyze global stability.

3.0 References

1) Bowles, Josepth E., (1982), "Foundation Analysis and Design," McGraw-Hill, Inc., pp 100.
2) Golder Associates boring logs, Report Appendix A.
3) Rocscience Slide Software Package version 7.029, dated October 16, 2017.
4) VHB Preliminary PDR drawing package dated August 17, 2017.
5) 3. Holtz, Robert D. and Kovacs, William D., "An Introduction to Geotechnical Engineering," Prentice-Hall, Inc., Upper Saddle River, New Jersey, 1981.
6) North Star Hydro, " Preliminary Design Hydrology and Hydraulics Report: Southwest Harbor and Tremont - Marsh Bridge #2511," dated August 2017.
7) Golder Associates calculation titled, " Organic and Clay Design Parameters Summary," dated 12/5/2017.

4.0 Attachments

1) Slide Output Figures
2) VHB Preliminary Design Plan View Layout

5.0 Assumptions

1) The load applied by the road and traffic is modeled at 250 psf.
2) Cohesive layers are assumed to exhibit fully undrained behavior ($\phi' = 0$)
3) Rip rap sections are 1.5H:1V, MaineDOT 703.26 Plain Riprap, 2ft thick, placed over the existing sideslope and organic layer, and are not embedded at the toe.
4) Based on field observations and the Preliminary Design Hydrology and Hydraulics Report in Reference 6, design for a water level at EL. 4 in the embankment fill materials and EL. 2.6 at the toe. This corresponds to MLW at the toe and an average water level observed within the embankment. MLW is the critical design case, however, the phreatic surface will remain higher within the embankment.
5) No work may be completed outside of the existing right-of-way as indicated in VHB's design drawings.
6) Based on several preliminary analyses, failure surfaces are generally circular in nature using both circular and non-circular failure search methods. Use Bishop Method for circular failures for the analyses discussed in this calculation.
7) The proposed wall system used in these equations is referred to as a modular block wall, however, these results are generic and are applicable to other systems which can provide the same strengths at the dimensions shown in the models. For all walls, we assumed that the modular block facing and reinforced zone would extend below frost (i.e., 4ft) measured from the top of the riprap slope in front of the wall and that the reinforced zone would extend 0.7H behind the facing blocks. The modular blocks and reinforced zones should be founded on a minimum 2 foot thick layer of granular structural fill that extends down to the clay.
8) Proposed geometry is based on VHB's preliminary design package (Reference 4).

6.0 Calculation

Determine input parameters to build the soil model in Slide. Use field N values, in situ field vane shear test data, lab direct simple shear test data, and local engineering experience to develop these parameters. The field N values and vane shear test data are shown on the boring logs (Reference 2). The material parameters selected for use in the Slide models are shown in Figure E-1.

Use the soil layer parameters summarized in Figure E-1 to analyze the following scenarios:

1) Proposed conditions at the locations with the most anticipated fill and/or weakest subsurface conditions (ST 15+50 Right, ST 16+00 Left, ST 16+15 Right, ST 17+50 Left and Right).

2) Proposed wall systems tallest wall height (ST 15+50 Left & a composite section representing the Southeast Abutment Corner)

Golder Associates

CALCULATIONS

Date: 11/22/2017 **Made by:** CJS

Project No.: 1668741 **Checked by:** MCM

Subject: Global Stability Analysis **Reviewed by:** MSP

Project Short Title: VHB Marsh Bridge Replacement

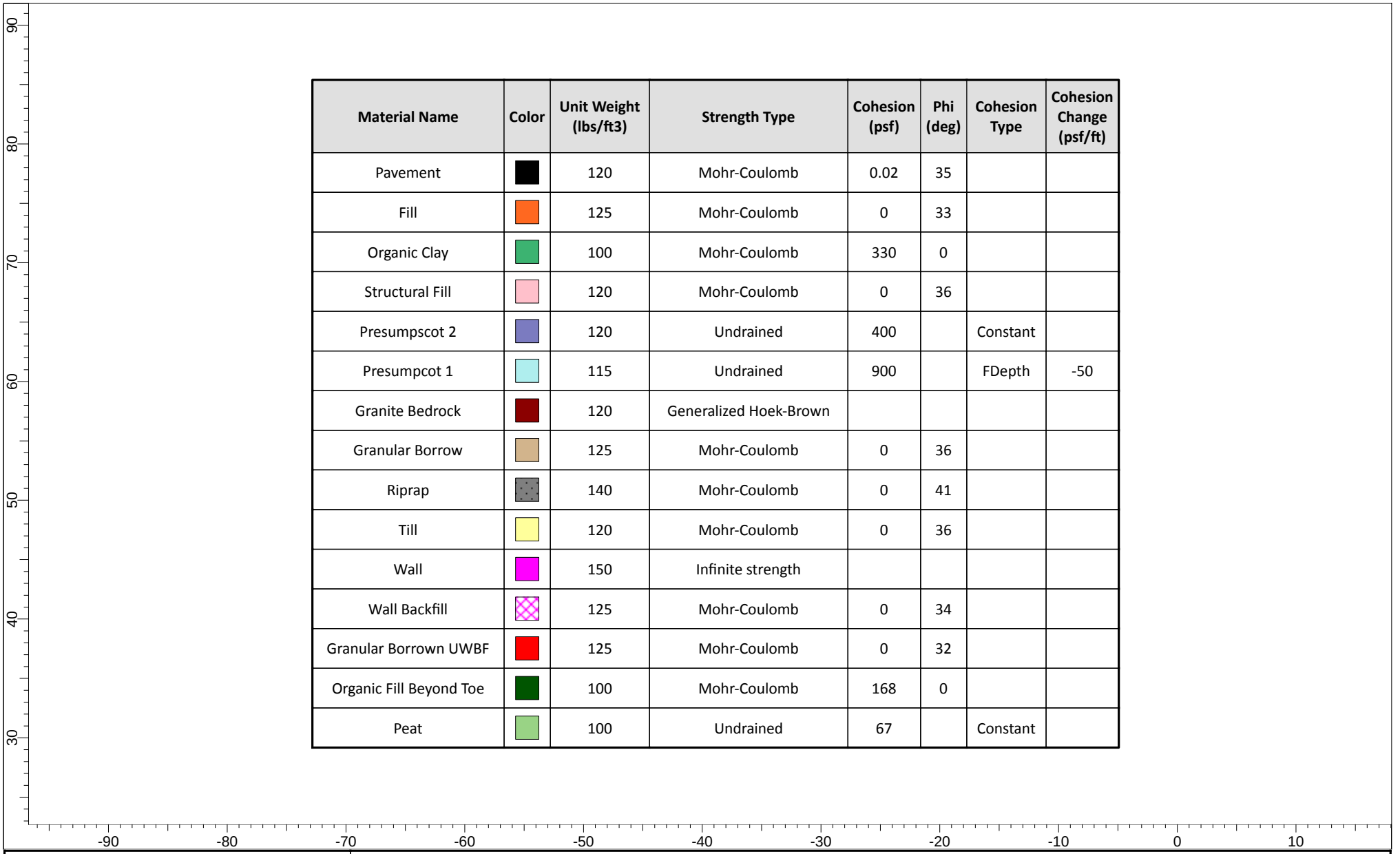
7.0 Results & Conclusions

Table 7.1: Proposed Conditions Slope Stability Results

Run Description	Figure No.	Global Factor of Safety	Offset From ROW (ft)	Representative Slope Length	Additional Toe Resistance Needed?	Comments
ST 15+50 Left	**E-2**	1.4	4.2	~ST15+25 to ST15+60 (~50ft long wall NW approach; ~10ft long wall SW approach)	No	Reinforced Modular Block Wall w/ 2ft thick 1.5H:1V Riprap Slope in Front
ST 15+50 Right	**E-3**	1.5	8.4	~ST14+75 to ST15+50 (without wall)	No	2ft thick 1.5H:1V Riprap Slope
ST 16+00 Left	**E-4**	1.3	4.3	~ST16+00 (without wall) to ST17+50	No	2ft thick 1.5H:1V Riprap Slope
ST 16+15 Right	**E-5**	1.3	Toe of slope: 5 Edge of Gabion: 1.1	~ST16+00 (without wall) to ST17+50	Yes. Equivalent to 350psf surcharge.	2ft thick 1.5H:1V Riprap Slope
Abutment Corner Wall	**E-6**	1.5	NA	~10ft long walls NE & SE approach	No	SE Abutment Corner, Reinforced Modular Block Wall w/ 2ft thick 1.5H:1V Riprap Slope in Front
ST 17+50 Left	**E-7**	1.7	9.0	~ST17+50 to ST18+50	No	2ft thick 1.5H:1V Riprap Slope
ST 17+50 Right	**E-8**	1.6	NA	~ST17+50 to ST18+50	No	3H:1V Soil Slope

Conclusion:

- Proposed widened embankment slopes and retaining walls analyzed are stable against global stability failures and indicate FS>1.3 as detailed.
- All proposed geometries maintain embankment widening activities within the Right of Way.
- Gabion baskets or other surcharge weight equivalent to 350psf (i.e. 2.5ft tall x 5ft wide gabion baskets) need to be placed along the toe of the southern embankment slope from ~16+15 to 16+75 to obtain a factor of safety greater than 1.3.
- Embankment retaining walls, founded at least 4ft below the top of riprap slops and bearing on compacted granular fill pads extending down to the native clay layer, with improved granular backfill extending at least 0.7H behind the wall will be required at the four corners of the bridge abutments.



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	Cohesion (psf)	Phi (deg)	Cohesion Type	Cohesion Change (psf/ft)
Pavement		120	Mohr-Coulomb	0.02	35		
Fill		125	Mohr-Coulomb	0	33		
Organic Clay		100	Mohr-Coulomb	330	0		
Structural Fill		120	Mohr-Coulomb	0	36		
Presumpscot 2		120	Undrained	400		Constant	
Presumpcot 1		115	Undrained	900		FDepth	-50
Granite Bedrock		120	Generalized Hoek-Brown				
Granular Borrow		125	Mohr-Coulomb	0	36		
Riprap		140	Mohr-Coulomb	0	41		
Till		120	Mohr-Coulomb	0	36		
Wall		150	Infinite strength				
Wall Backfill		125	Mohr-Coulomb	0	34		
Granular Borrown UWBF		125	Mohr-Coulomb	0	32		
Organic Fill Beyond Toe		100	Mohr-Coulomb	168	0		
Peat		100	Undrained	67		Constant	

Golder Associates

SLIDE 7.029

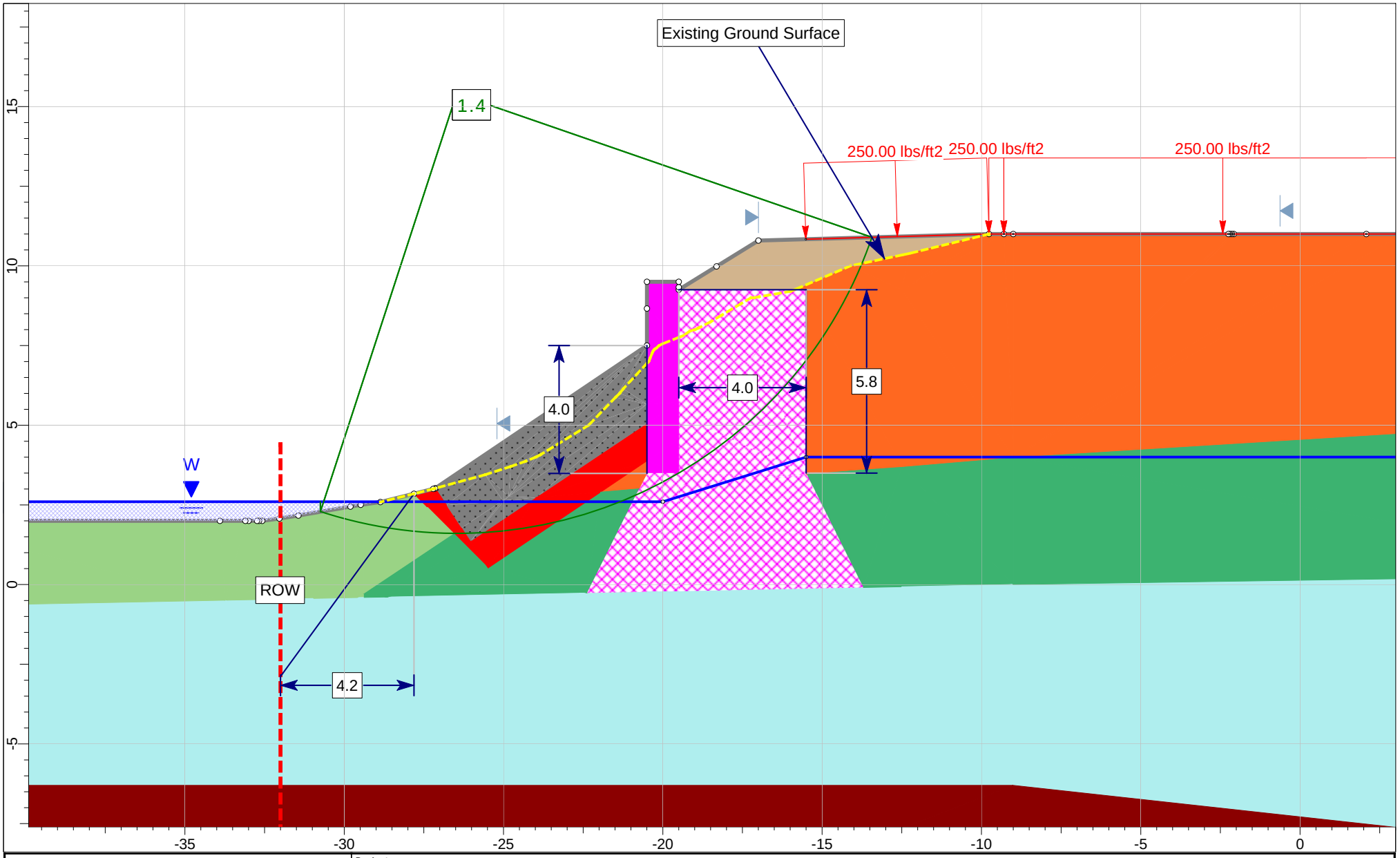
Project VHB Marsh Bridge Final Design

Analysis Description Material Properties Summary

Drawn By	*Checked by:*	*Reviewed by:*	*Scale*
CJS	MCM	MSP	1:134

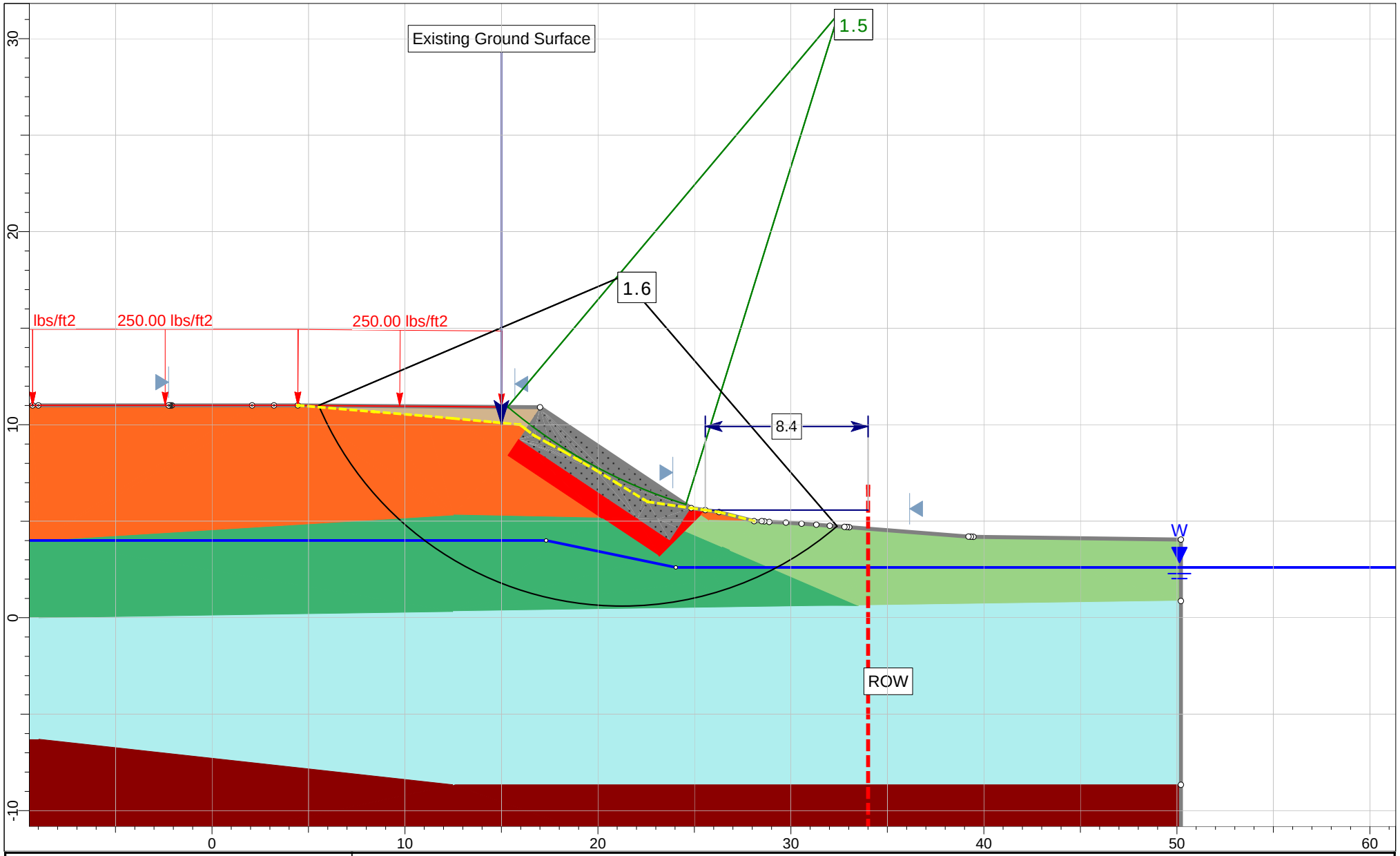
Date 11/22/2017

Figure E-1



Golder Associates	Project: VHB Marsh Bridge Final Design			
	Analysis Description: STA 15+50 Left Section, Reinforced Modular Block Wall w/ 2ft 1.5H:1V Riprap Slope In Front			
	Drawn By: CJS	Checked by: MCM	Reviewed by: MSP	Scale: 1:50
	Date: 11/17/2017			Figure E-2

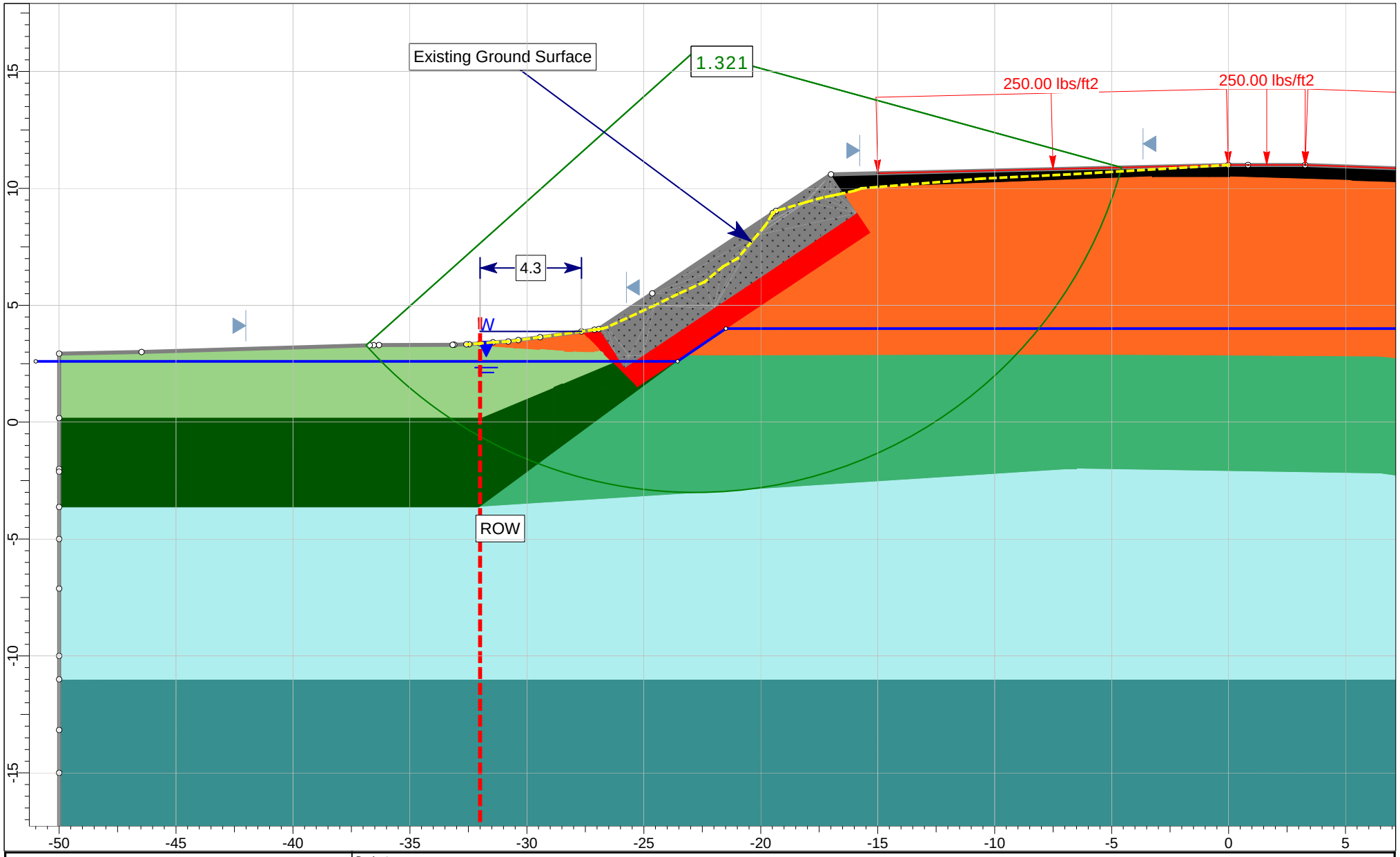
SLIDEINTERPRET 7.029



Golder Associates	Project: VHB Marsh Bridge Final Design			
	Analysis Description: STA 15+50 Right, Section 2ft 1.5H:1V riprap slope			
	Drawn By: CJS	Checked by: MCM	Reviewed by: MSP	Scale: 1:82
	Date: 11/17/2017			

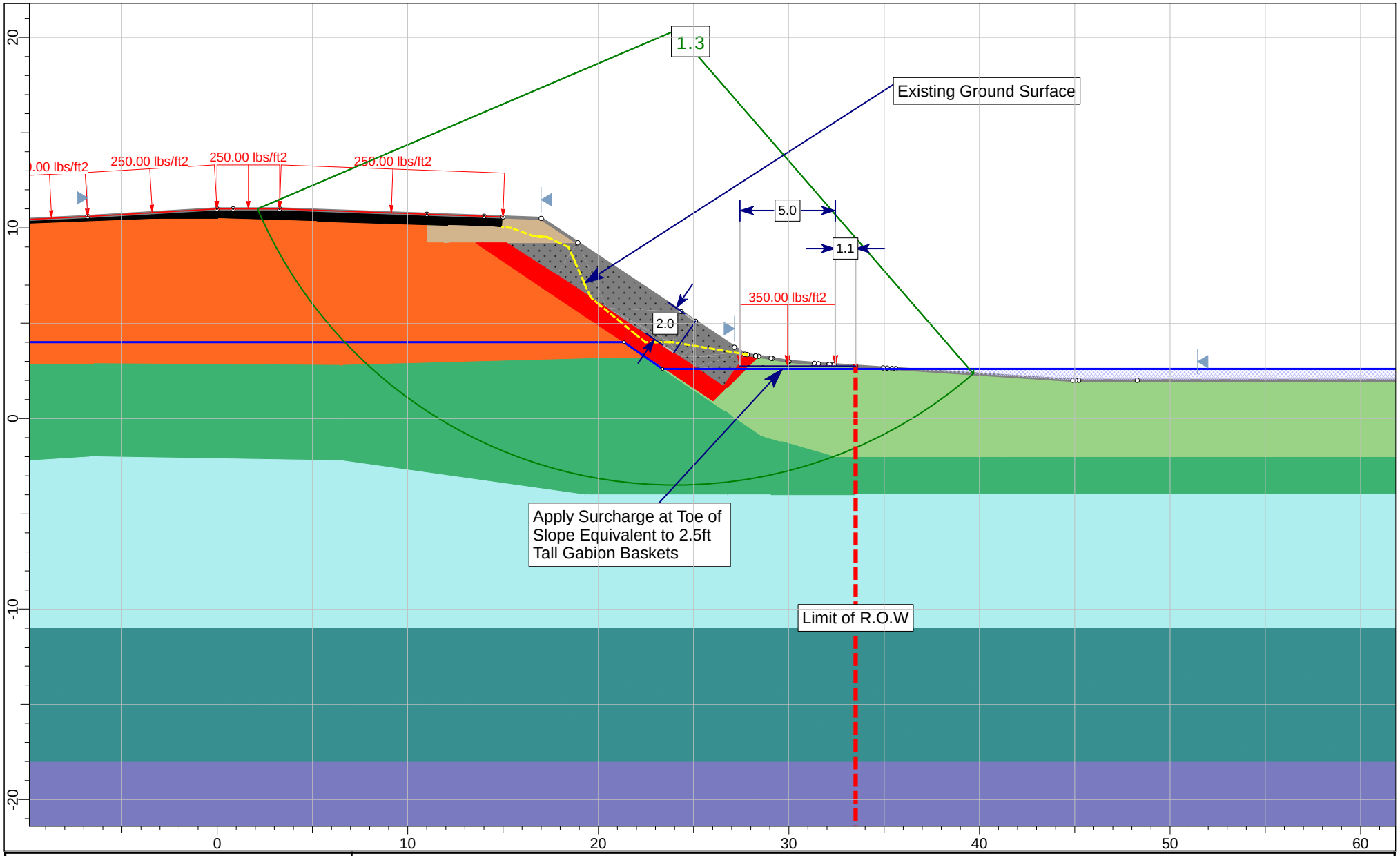
SLIDEINTERPRET 7.029

Figure E-3



Golder Associates	Project: VHB Marsh Bridge Final Design			
	Analysis Description: 16+00 Left Proposed, 2ft 1.5H:1V riprap slope			
	Drawn By: CJS	Checked by: MCM	Reviewed by: MSP	Scale: 1:68
	Date: 11/17/2017			Figure E-4

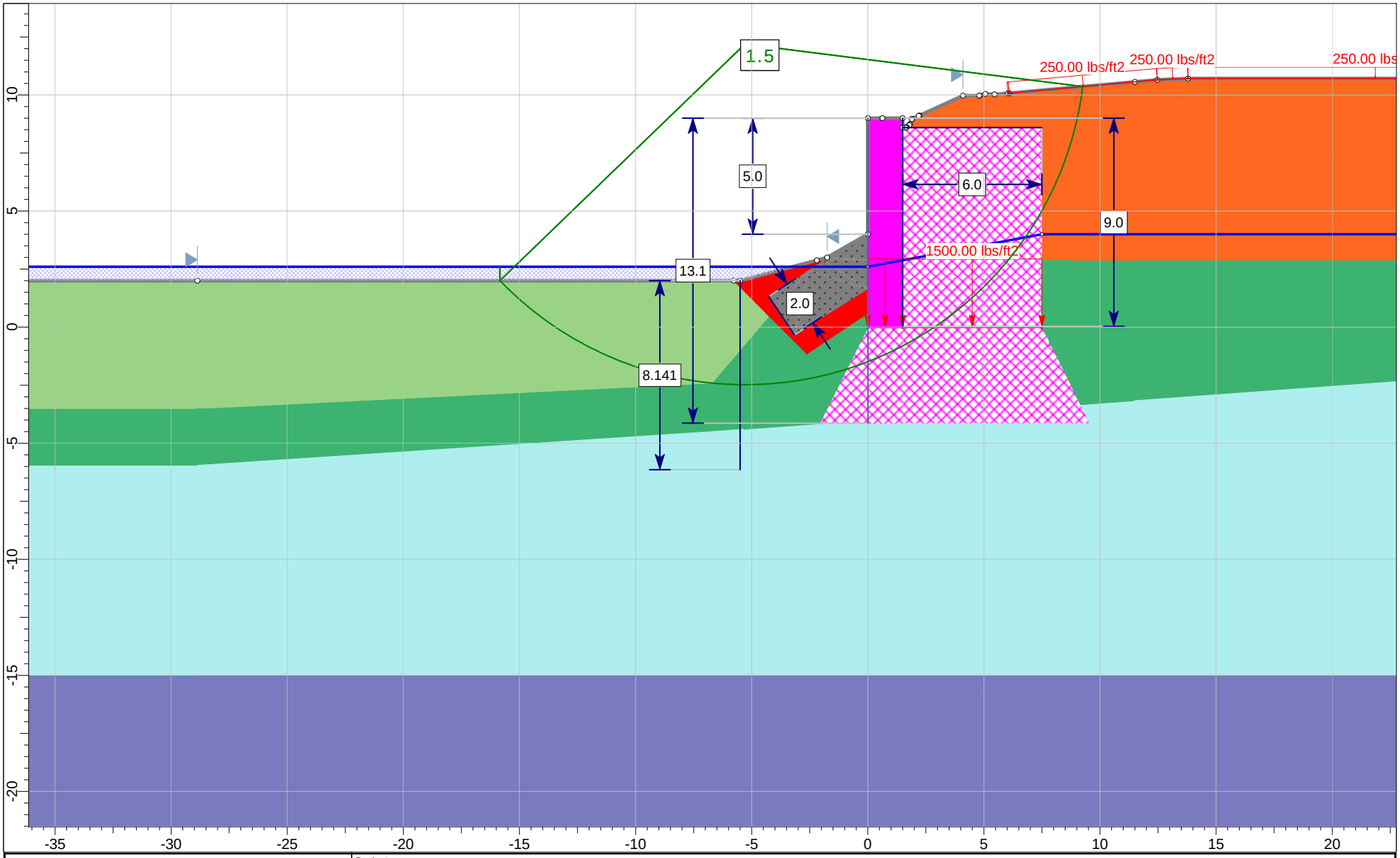
SLIDEINTERPRET 7.029



Golder Associates	Project: VHB Marsh Bridge Final Design			
	Analysis Description: 16+15 Right Proposed, 2ft 1.5H:1V riprap slope			
	Drawn By: CJS	Checked by: MCM	Reviewed by: MSP	Scale: 1:83
	Date: 11/17/2017			

SLIDEINTERPRET 7.029

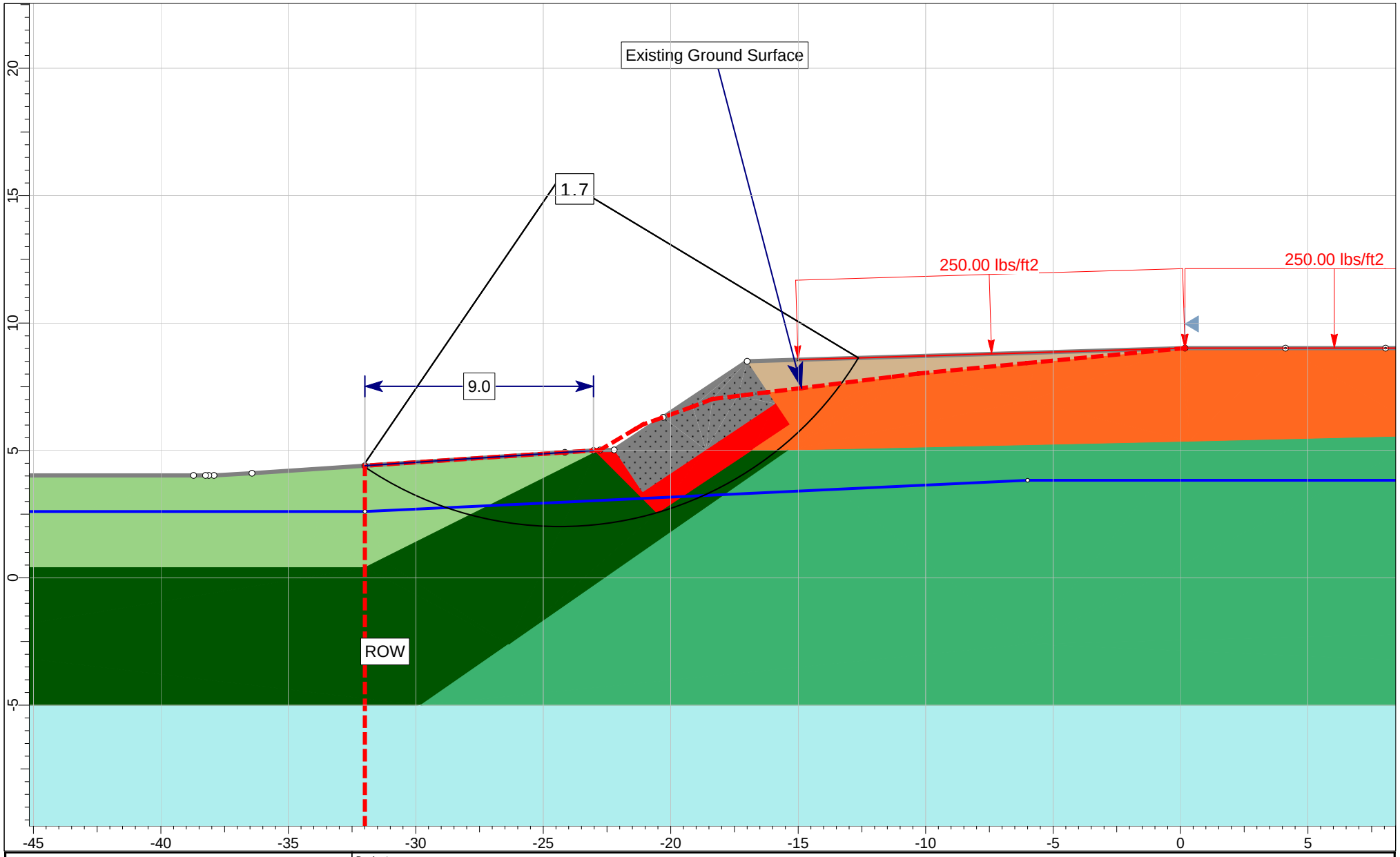
Figure E-5



Golder Associates

SLIDEINTERPRET 7.029

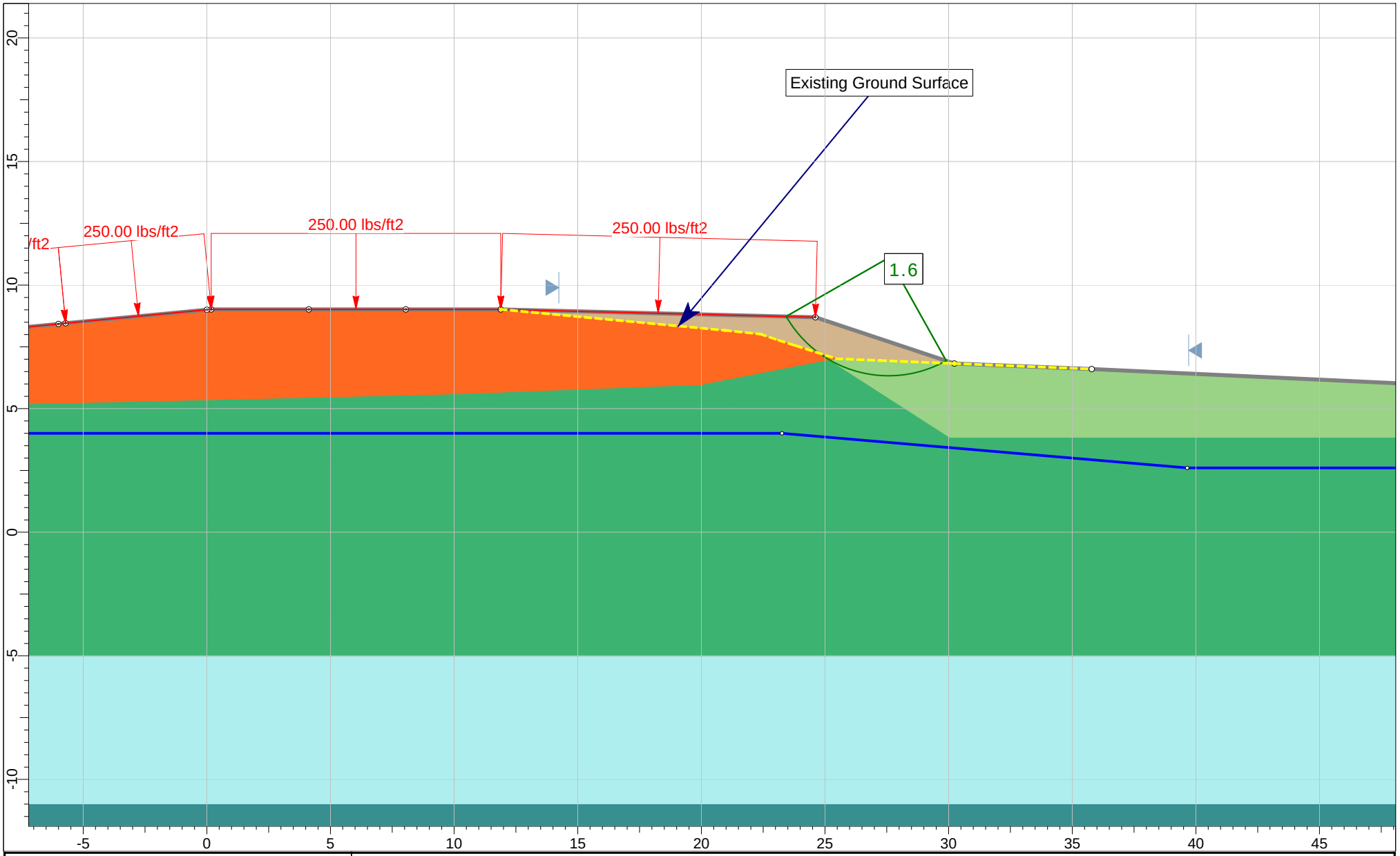
Project	VHB Marsh Bridge Final Design			
Analysis Description	Short Abutment Walls (Bearing Stress)- 16+00 Right, SE Abutment, Reinforced Modular Block Wall w/ 2ft 1.5H:1V RipRap Slope In Front			
Drawn By: CJS	Checked by: MCM	Reviewed by: MSP	Scale: 1:69	Figure E-6
Date: 11/17/2017				



Golder Associates

SLIDEINTERPRET 7.029

Project	VHB Marsh Bridge Final Design			
Analysis Description	Sta. 17+50, Left, 2ft 1.5H:1V Riprap Slope			
Drawn By CJS	*Checked by:* MCM	*Reviewed by:* MSP	*Scale* 1:62	Figure E-7
Date 12/1/2017				



Golder Associates	Project: VHB Marsh Bridge Final Design			
	Analysis Description: Sta. 17+50, Right, 3H:1V Soil Slope			
	Drawn By: CJS	Checked by: MCM	Reviewed by: MSP	Scale: 1:64
	Date: 12/4/2017			

SLIDEINTERPRET 7.029

Figure E-8

Filename: ...\MSTA\001_GeneralPlan1.dgn Division: BRIDGE Username: PDonovan Date:8/16/2017

CURVE DATA #2
PI = 19+33.79
D = 8°00'48.2"
Δ = 32°22'12.2" Lt.
R = 715.00'
L = 403.95'
T = 207.52'
E = 29.51'

POR #7
Map 3, Lot 18
PETER EARL

POR #8
Map 3, Lot 17
UNITED STATES OF AMERICA

POR #5
Map 15, Lot 8
UNITED STATES OF AMERICA

PC = STA. 17+26.27

℄ Brg..
Abut. No. 1
Sta. 15+66.31

℄ Brg..
Abut. No. 2
Sta. 15+88.31

22'-0"

SOUTHWEST HARBOR
TREMONT

Limits of
Parking Area

Sta. 12+75.00
Begin Mill and Fill
Begin Limit of Work

Sta. 14+75.00
End Transition
Begin Full
Construction

Sta. 18+25.00
End Transition
Begin Mill and Fill

Sta. 19+67.20
End Shoulder
Reconstruction
End Limit of Work

Heavy Riprap
Typ.

Plain Riprap
Typ.

Stability Analysis Section Shown
on Figure E-6

Sta. 18+50.00
End Mill and Fill
Begin Shoulder
Reconstruction

Sta. 18+00.00
End Full Construction
Begin Transition

18" x 50' Opt I

Sta. 14+50.00
End Mill and Fill
Begin Transition

Highest Annual Tide
Elevation (HAT)

CURVE DATA #1
PI = 12+20.50
D = 6°37'25.6"
Δ = 27°22'00.2" Rt.
R = 865.00'
L = 413.16'
T = 210.60'
E = 25.27'

PT = STA. 14+23.06

POR #6
Map 3, Lot (none)
ACADIA NATIONAL PARK

POR #1
Map 20, Lot 21
ACADIA NATIONAL PARK

POR #3
Map 20, Lot 3
ACADIA NATIONAL

POR #4
Map 20, Lot 20
PHEOBE BARNES

MHW (Typ.)

MLW (Typ.)

N66°29'31.15"E

PRELIMINARY
NOT FOR
CONSTRUCTION
08/17/2017

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2170(200)

WIN 21702.00

BRIDGE NO. 2511

BRIDGE PLANS

PRELIMINARY
NOT FOR
CONSTRUCTION
08/17/2017

	BY	DATE
PROJ. MANAGER	L. TIMBERLAKE	
DESIGN-DETAILED	TJC	DPD
CHECKED-REVIEWED	DME	RSBLUNT
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

MARSH BRIDGE
BASS HARBOR ROAD OVER MARSHALL BROOK
TREMONT/SOUTHWEST HARBOR HANCOCK COUNTY

SHEET NUMBER
1
OF 12

APPENDIX E.5
BEARING CAPACITY OF RETAINING WALL SYSTEM

Golder Associates	**SUBJECT:** Bearing Capacity of Proposed Approach Embankment Wall System **Project Number:** 1668741 **Project Name:** VHB Marsh Bridge - Final Design **Prepared by:** CJS **Checked by:** LLM **Date:** 11/15/2017 **Reviewed by:** MCM

Objective:

To calculate the bearing resistance of the proposed modular block wall system for the strength limit state.

Method:

Use the method described in the AASHTO LRFD Bridge Design Specifications, 7th Ed, 2014. Calculate the factored bearing resistance for both the strength limit state.

References:

1. AASHTO LRFD Bridge Design Specifications, 7th Ed, 2014 (LRFD)
2. GEOTECHNICAL ENGINEERING CIRCULAR NO. 6: Shallow Foundations. FHWA: September 2002 (FHWA).
3. MEDOT. (2003). "Bridge Design Guide", includes updates through (2014).
4. VHB Preliminary PDR drawing package dated August 17, 2017.
5. Golder slope stability calculation package dated 12/4/2017.
6. Golder Associates calculation titled, " Organic and Clay Design Parameters Summary," dated 12/5/2017.

Assumptions:

1. Ground water table is at top of organic layer. D_w = 0 ft
2. Ground surface (i.e. depth = 0ft) is at top of organic layer.
3. Assume wall system is 6ft wide and 50ft long. The shorter walls (10ft should have a higher bearing capacity).

Calculations:

A. Determine the bearing resistance at the strength limit state. Use Article 10.6.3, Strength Limit Design, from the AASHTO LRFD Bridge Design Specifications. This calculation is a typical calculation based on a 6ft wide by 50ft long wall system. Other wall sizes were analyzed using a similar calculation.

Assume fill below footing will be compacted Structural Fill material such as Underdrain Backfill Material Type C (703.22). Use properties from MEDOT Bridge Design Guide. Assume second layer is presumpscot clay with strength parameters obtained from in situ vane shear testing. Selected values are summarized below:

ϕ_f = internal friction angle, deg
c = cohesion for Presumpscot material, psf

	Internal Friction Angle, ϕ_f (^{o})	Cohesion (ksf)	Unit weight, γ (kcf)
Compacted Fill Below Footing	36	0	0.135
Organic Clay	0	0.900	0.115

H = thickness of upper soil layer below footing (compacted fill), ft H = 2.0 ft.

According to LRFD Article 10.6.3.1.2d, when the soil profile contains a second soil layer of different properties within a distance below the footing less than H_{crit}, then the bearing resistance needs to be calculated for a two-layered soil system.

$$H_{crit} = \frac{(3 * B_f)\ln\left(\frac{q_1}{q_2}\right)}{2\left(1+\frac{B_f}{L_f}\right)} \qquad \text{Eq 10.6.3.1.2d-1}$$

where:

q_1 = nominal bearing resistance of a footing supported by the upper soil assuming the layer is infinitely thick
q_2 = nominal bearing resistance of a fictitious footing supported on the surface of the second layer

Layer 1 - Structural Fill Below Footing

Calculate q_1:

$$q_1 = \gamma_1 D_{f1} N_{qm1} C_{wq1} + \frac{1}{2}\gamma_1 B_f N_{\gamma m1} C_{w\gamma}$$

where:

		Fill	
B_f =	footing width, ft	B_f = 6	ft
γ_1 =	total unit weight, kcf	γ_1 = 0.135	kcf
ϕ^*_{f1} =	effective friction angle (reduced for failure mode)	ϕ^*_{f1} = 36	o
D_f =	footing embedment depth, ft	D_f = 4.0	ft
L_f =	footing length, ft	L_f = 50	ft

$$N_{qm1} = N_{q1} s_q d_q$$

N_{qm1} = 41.0

N_{q1} = surcharge factor, Table 10.6.3.1.2a-1 — N_{q1} = 37.8

$$s_q = 1 + \left(\frac{B_f}{L_f}\tan\phi_{f1}\right)$$

s_q = 1.09

d_q = depth correction factor, Table 10.6.3.1.2a-4 — d_q = 1.00

$$N_{\gamma m1} = N_{\gamma q1} s_\gamma$$

$N_{\gamma m1}$ = 53.6

$N_{\gamma 1}$ = unit weight factor, Figure 10.6.3.1.2a-1 — $N_{\gamma 1}$ = 56.3

$$s_\gamma = 1 - 0.4\left(\frac{B_f}{L_f}\right)$$

s_γ = 0.95

D_w = depth to water, ft — D_w = 0

C_{wq} = coefficient of groundwater, FHWA eq 5-9

$$C_{wq} = 0.5 + 0.5\left(\frac{D_w}{D_f}\right) \le 1.0$$

C_{wq} = 0.50

$C_{w\gamma}$ = coefficient of groundwater, FHWA eq 5-10

$$C_{w\gamma} = 0.5 + 0.5\left(\frac{D_w}{1.5B_f + D_f}\right) \le 1.0$$

$C_{w\gamma}$ = 0.50

q_1 = 21.9 ksf

Layer 2 - Presumpscot Clay

Calculate q_2:

$$q_2 = cN_{cm2} + \gamma_2 D_{f2} N_{qm2} C_{wq2} + \frac{1}{2}\gamma_2 B_f N_{\gamma m2} C_{w\gamma}$$

where:

		Presumpscot	
B_f =	footing width, ft	B_f = 6	ft
γ_2 =	total unit weight, kcf	γ_2 = 0.115	kcf
ϕ^*_{f2} =	effective friction angle (reduced for failure mode)	ϕ^*_{f2} = 0	o
c* =	effective cohesion (reduced for failure mode), ksf	c* = 0.900	ksf
D_f =	footing embedment depth = $D_{f(1)}$+H, ft	D_{f2} = 6.0	ft
L_f =	footing length, ft	L_{f2} = 50	ft

$$N_{qm2} = N_{q2} s_q d_q$$

N_{qm2} = 1.0

N_{q2} = surcharge factor, Table 10.6.3.1.2a-1 — N_{q2} = 1.0

$$s_q = 1 + \left(\frac{B_f}{L_f}\tan\phi_{f2}\right)$$

s_q = 1.00

d_q = depth correction factor, Table 10.6.3.1.2a-4 — d_q = 1.00

$$N_{\gamma m2} = N_{\gamma 2} s_\gamma$$ — $N_{\gamma m2}$ = 0.0

$N_{\gamma 2}$ = unit weight factor, Figure 10.6.3.1.2a-1 — $N_{\gamma 2}$ = 0.0

$$s_\gamma = 1 - 0.4\left(\frac{B_f}{L_f}\right)$$ — s_γ = 1.00

D_w = depth to water, ft — D_w = 0 ft

C_{wq} = coefficient of groundwater, FHWA eq 5-9

$$C_{wq} = 0.5 + 0.5\left(\frac{D_w}{D_f}\right) \le 1.0$$ — C_{wq2} = 0.50

$C_{w\gamma}$ = coefficient of groundwater, FHWA eq 5-10

$$C_{w\gamma} = 0.5 + 0.5\left(\frac{D_w}{1.5B_f + D_f}\right) \le 1.0$$ — $C_{w\gamma}$ = 0.50

$$N_{cm2} = N_c s_c$$ — N_{cm2} = 5.3

N_{c2} = cohesion factor, Table 10.6.3.1.2a-1 — N_{c2} = 5.1

$$s_c = 1 + \left(\frac{B}{5L}\right)$$ — s_c = 1.02

q_2 = 5.1 ksf

Calculate H_{crit}

$$H_{crit} = \frac{(3 * B_f)\ln\left(\frac{q_1}{q_2}\right)}{2\left(1 + \frac{B_f}{L_f}\right)}$$ Eq 10.6.3.1.2d-1

H_{crit} = 11.8 ft

Is H_{crit} less than H? No

If H_{crit} is less than H, then a two-layer system does not apply.

Calculate q_n for a two layer system

$$q_n = q_2 e^{2\left[1+\left(\frac{B_f}{L_f}\right)\right]K \tan\phi_{f1}\left(\frac{H}{B}\right)}$$ Eq 10.6.3.1.2f-1

where:

$$K = \frac{1 - \sin^2\phi_{f1}}{1 + \sin^2\phi_{f1}}$$ Eq 10.6.3.1.2f-2 — K = 0.5

q_n = 6.6 ksf

q_n for two-layer system: use $q_n = q_1$ if H_{crit} is less than H, otherwise use q_n calculated for two-layers.

q_n = 6.6 ksf

Apply resistance factor:

$$q_R = \varphi_b q_n$$

φ_b = resistance factor specified in Article 11.5.7 = — φ_b = 0.55
(Table 11.5.7-1: Bearing: Gravity/Semi-Gravity Walls)
(NOTE: φ_b = 0.65 is recommended for reinforced MSE Walls)

$q_{R\text{-}2}$ (ksf) = 3.64

Conclusions:

For an approach embankment wall system 6ft wide x50ft long, the strength limit bearing capacity is 3.64 ksf. The strength limit bearing capacity for other footing sizes at both the east and west approaches are summarized in the table below:

Footing Size	East or West Approach	Strength Limit Bearing Capacity
50ft x 6ft	West Approach	3.64
50ft x 8ft	West Approach	3.46
10ft x 6ft	East Approach	4.43
10ft x 8ft	East Approach	4.32
10ft x 6ft	West Approach	4.43
10ft x 8ft	West Approach	4.32

APPENDIX E.6
RETAINING WALL ESTIMATED SETTLEMENT

Golder Associates	**SUBJECT:** Settlement Proposed Wall - West Approach **Project Number:** 1668741 **Project Name:** Marsh Bridge **Prepared by:** CJS **Checked by:** LLM **Date:** 11/14/2017 **Reviewed by:** MCM

Objective:

A) To estimate the service limit state bearing resistance for up to 3 inches of settlement of the proposed wall system, along the western approach, for the Marsh Bridge replacement.
B) To estimate the time rate of settlement through the clay layer.

Method:

Use the methods described in the AASHTO LRFD Bridge Design Specifications, 7th Ed, 2014 and Holtz & Kovacs to estimate settlement under the proposed approach embankment retaining wall.

References:

1. Holtz, R. D., & Kovacs, W. D. (1981). *An Introduction to Geotechnical Engineering* . Upper Saddle River: Prentice-Hall, Inc.
2. Golder Associates calculation titled, " Organic and Clay Design Parameters Summary," dated 12/5/2017.
3. VHB Preliminary PDR drawing package dated August 17, 2017.

Attachments:

1. Figure - Soil Profile for west walls

Assumptions:

1. Ground water table is at the ground surface at toe of slope. D_w = 0 *ft*
2. Clay consolidation parameters (for Presumpscot Clay) Ccε = 0.14, Crε = 0.02 are based on laboratory data. Consolidation parameters for the organic layer are assumed to be Ccε = 0.37, Crε = 0.12 based on laboratory data. c_v = 5x10^{-6} ft^2/s (for Presumpscot Clay). cv = 4x10^{-7} ft^2/s (for Organic Layer). Refer to Reference 2.
3. Assume organic layer at toe starts at ~ EL. 0.

Calculations:

Based on review of the VHB drawings (Reference 3) and discussions with retaining wall vendors we evaluated 4 wall options for the west approach: 50ft x 6ft, 50ft x 8ft, 10ft x 6ft, and 10ft x 8ft. The results of these options are included at the end of this calculation. The following example calculation is for a 6ft wide footing with 1-inch of settlement. Each option was evaluated for up to 3-inches of settlement.

A) Use consolidation theory to estimate settlement of the clay layers.

1. Subdivide the subsurface soils into layers no larger than 10 ft thick and to a depth two times the footing width. Assume embankment stress distribution method (Fig 8.23, Ref 1).

$$\frac{\sigma_z}{q_o} = I$$ (Holtz & Kovacs, Figure 8.23)

where:

q_o = Unit load of embankment
b = Width of flat part of embankment
Z = depth to mid point of layer, ft

0.7H assuming H=9ft
b = 6 *ft*

Refer to attached Figure, Soil Profile for West Walls. Assume that the organic clay layer is on average 5 ft thick. During construction, this organic layer will be need to be removed fully (i.e. to the top of clay layer, D=5ft, assuming that D=0 is the top of the organic clay for the purposes of this calculation). The wall will then be constructed by placing 2 ft of structural fill for the wall to bear upon (i.e. structural fill from D=3ft to D=5ft)

				Influence factor for embankment load		
Layer		Depth Below Top of Organic Clay Layer(ft)	Thickness (ft)	z, below footing (ft)	b/z	I ($\Delta\sigma_z$)/q_o
Organic Clay	1a	0-3	Load Bearing on Structural Fill Pad			
Structural Fill Pad	1b	3-5	2	1	6.0	0.986
Presumpscot	2	5-11	6	5	1.2	0.624

As stated above, D=0 is assumed to be the top of the organic clay layer. This calculation assumes that a wall will be constructed through the side of the embankment slope. Typically, there is approximately 6ft of embankment fill above the organic layer at the proposed wall locations. Assume all embankment fill is above the water table for the purpose of this calculation. In the calculation below, the effective stress is calculated from the top of the Organic Layer in Column A. The effective stress is then increased for the 6ft of embankment stress in Column B. Note that the structural fill pad is part of the proposed construction, so for the existing effective stress calculate stress for the full 5ft Organic Clay Layer and neglect the Structural Fill Pad.

Unit Weight of Embankment Fill	0.125	kcf
Embankment Fill Thickness Above Organic Clay Layer	6	ft
Additional Stress From Embankment Fill Above the Organic Clay Area	0.75	kcf

					Column A	Column B
Layer	Depth Below Top of Organic Clay Layer(ft)	Layer Thickness (ft)	Z (ft)	Unit Weight of Layer (ksf)	σ'_{v0} - From Top of Organic Layer (ksf)	σ'_{v0} - Including 6ft of Embankment Fill Above the Organic Layer (ksf)
1. Organic Clay	0 - 5	5	2.5	0.080	0.04	0.79
2. Presumpscot	5 - 11	6	8	0.115	0.25	1.00

3. Calculate the total settlement of the clays using the consolidation theory:

$$\Delta H_i = C_{r\varepsilon} H_c \log\frac{\sigma'_p}{\sigma'_{v0}} + C_{c\varepsilon} H_c \log\frac{\sigma'_{v0} + \Delta\sigma_v}{\sigma'_p}$$

for overconsolidated clay, $\sigma'_{v0}+\Delta\sigma_v>\sigma'_p$

$$\Delta H_i = C_{r\varepsilon} H_c \log\frac{\sigma'_{v0} + \Delta\sigma_v}{\sigma'_{v0}}$$

for overconsolidated clay, $\sigma'_{v0}+\Delta\sigma_v<=\sigma'_p$

$$\Delta H_i = C_{c\varepsilon} H_c \log\frac{\sigma'_{v0} + \Delta\sigma_v}{\sigma'_{v0}}$$

for normally consolidated clay

$$\Delta\sigma_v = \frac{\Delta\sigma_v}{q} \times q$$

Where:

H_c = initial height (thickness) of layer *i* , ft
$\Delta\sigma_v$ = surcharge load, psf
σ'_{v0} = in situ vertical effective stress, psf
$\sigma'_{v0} + \Delta\sigma_v$ = (check against σ'_p to select equation)
σ'_p = preconsolidated stress, psf
$C_{r\varepsilon}$ = modified recompression index
$C_{c\varepsilon}$ = modified compression index
ΔH_i = Total settlement

	Clays	Organic Clays
$C_{c\varepsilon}$ =	0.14	0.37
$C_{r\varepsilon}$ =	0.02	

	Layer 1a Organic Clay	Layer 1b Structural Fill Pad	Layer 2 Clay
H_c (ft)	Layer Removed During Construction	Settlement Assumed to Occur Near Instantaneously During Construction	6.0
OCR			3.6
$\Delta\sigma_v$ (psf)			2789
σ'_{v0} (psf)			996
$\sigma'_{v0} + \Delta\sigma_v$ (psf)			3785
σ'_p (psf)			3585
			OC
ΔH_i (ft)			0.083
ΔH_i (in)			1.00

Limit allowable bearing stress to limit total estimated settlements to 1-inch.

	Layer	Settlement ΔH_i (in)
1a. Organic Clay	1a	Removed During Construction
1b. Structural Fill Pad	1b	Negligible
2. Presumpscot	2	1.00
	Total Settlement: (in)	1.00
	Bearing stress, q (ksf)	4.47

Conclusions A):

A service limit state bearing stress of 4.26 may be used to limit settlements to 1-inch, when wall footings over the western approach are founded on a minimum 2ft thick structural fill pad bearing on native clay soils. Differential settlements can be expected to be up to half tot total estimated settlement.

West Approach Wall Results

Footing Size	East or West Approach	Service Limit Bearing Capacity (ksf)		
		1" Settlement	2" Settlement	3" Settlement
50ft x 6ft	West Approach	4.47	6.02	7.98
50ft x 8ft	West Approach	3.79	5.11	6.77
10ft x 6ft	West Approach	4.47	6.02	7.98
10ft x 8ft	West Approach	3.79	5.11	6.77

Calculations B):

Calculate Time Rate of Consolidation (Clay). Look at an average clay thickness of 6ft for the western approach. Assume layer is single drained:

$$t_{90} = \frac{T_{90} H_{dr}^2}{c_v}$$ ref 1, eq. 9-5 $$t_{50} = \frac{T_{50} H_{dr}^2}{c_v}$$

Where:

$H_{dr\ -1}$	6	thickness of drainage layer, (ft)
T_{50}	0.197	time factor for 50% consolidation
T_{90}	0.848	time factor for 90% consolidation
c_v	5.00E-06	coefficient of consolidation from laboratory data, (ft2/sec)
$t_{50\text{-}1}$	1.42E+06	time to reach 50% consolidation, (sec)
$t_{90\text{-}1}$	6.11E+06	time to reach 90% consolidation, (sec)

t_{50}	**0.04**	**years**
t_{50}	**0.5**	**months**
t_{90}	**0.2**	**years**
t_{90}	**2.3**	**months**

Conclusions B):

Estimated it will take 0.2 years (2.3 months) to reach 90% consolidation and 0.04 years (0.5 months) to reach 50% consolidation within the approximately 6ft thick layer of clay present beneath the western approach retaining walls.

Golder Associates	**SUBJECT:** Settlement Proposed Wall - East Approach **Project Number:** 1668741 **Project Name:** Marsh Bridge **Prepared by:** CJS **Checked by:** LLM **Date:** 11/14/2017 **Reviewed by:** MCM

Objective:

A) To estimate the service limit state bearing resistance for up to 3 inches of settlement of the proposed wall system, along the eastern approach, for the Marsh Bridge replacement.

B) To estimate the time rate of settlement through the clay layer.

Method:

Use the methods described in the AASHTO LRFD Bridge Design Specifications, 7th Ed, 2014 and Holtz & Kovacs to estimate settlement under the proposed approach embankment retaining wall.

References:

1. Holtz, R. D., & Kovacs, W. D. (1981). *An Introduction to Geotechnical Engineering* . Upper Saddle River: Prentice-Hall, Inc.
2. Golder Associates calculation titled, " Organic and Clay Design Parameters Summary," dated 12/5/2017.
3. VHB Preliminary PDR drawing package dated August 17, 2017.

Attachments:

1. Figure - Soil Profile for east walls

Assumptions:

1. Ground water table is at the ground surface at toe of slope. D_w = 0 *ft*
2. Clay consolidation parameters (for Presumpscot Clay) Ccε = 0.14, Crε = 0.02 are based on laboratory data. Consolidation parameters for the organic layer are assumed to be Ccε = 0.37, Crε = 0.12 based on laboratory data. $c_v = 5x10^{-6}$ ft^2/s (for Presumpscot Clay). $cv = 4x10^{-7}$ ft^2/s (for Organic Layer). Refer to Reference 2.
3. Assume organic layer at toe starts at ~ EL. 0.

Calculations:

Based on review of the VHB drawings (Reference 3) and discussions with retaining wall vendors we evaluated 2 wall options for the east approach: 10ft x 6ft and 10ft x 8ft. The following example calculation is for a 6ft wide footing with 1-inch of settlement. Each option was evaluated for up to 3-inches of settlement.

A) Use consolidation theory to estimate settlement of the clay layers.

1. Subdivide the subsurface soils into layers no larger than 10 ft thick and to a depth two times the footing width. Assume embankment stress distribution method (Fig 8.23, Ref 1).

$$\frac{\sigma_z}{q_o} = I$$ (Holtz & Kovacs, Figure 8.23)

where:

q_o = Unit load of embankment

b = Width of flat part of embankment — 0.7H assuming H=9ft; b = 6 *ft*

Z = depth to mid point of layer, ft

Refer to attached Figure, Soil Profile for East Walls. Assume that the organic clay layer is on average 5 ft thick. During construction, this organic layer will be need to be removed fully (i.e. to the top of clay layer, D=5ft, assuming that D=0 is the top of the organic clay for the purposes of this calculation). The wall will then be constructed by placing 2 ft of structural fill for the wall to bear upon (i.e. structural fill from D=3ft to D=5ft)

Influence factor for embankment load

Layer		Depth Below Top of Organic Clay Layer(ft)	Thickness (ft)	z, below footing (ft)	b/z	I $(\Delta\sigma_z)/q_o$
Organic Clay	1a	0-3	Load Bearing on Structural Fill Pad			
Structural Fill Pad	1b	3-5	2	1	6.0	0.986
Presumpscot	2	5-12	7	5.5	1.1	0.587
Presumpscot	3	12-15	3	10.5	0.6	0.358
Presumpscot	4	15-25	10	17	0.4	0.246
Presumpscot	5	25-29	4	24	0.3	0.187

As stated above, D=0 is assumed to be the top of the organic clay layer. This calculation assumes that a wall will be constructed through the side of the embankment slope. Typically, there is approximately 6ft of embankment fill above the organic layer at the proposed wall locations. Assume all embankment fill is above the water table for the purpose of this calculation. In the calculation below, the effective stress is calculated from the top of the Organic Layer in Column A. The effective stress is then increased for the 6ft of embankment stress in Column B. Note that the structural fill pad is part of the proposed construction, so for the existing effective stress calculate stress for the full 5ft Organic Clay Layer and neglect the Structural Fill Pad.

Unit Weight of Embankment Fill	0.125	kcf
Embankment Fill Thickness Above Organic Clay Layer	6	ft
Additional Stress From Embankment Fill Above the Organic Clay Area	0.75	kcf

					Column A	Column B
Layer	Depth Below Top of Organic Clay Layer(ft)	Layer Thickness (ft)	Z (ft)	Unit Weight of Layer (ksf)	σ'_{v0} - From Top of Organic Layer (ksf)	σ'_{v0} - Including 6ft of Embankment Fill Above the Organic Layer (ksf)
1. Organic Clay	0 - 5	5	2.5	0.080	0.04	0.79
2. Presumpscot	5 - 12	7	8.5	0.115	0.27	1.02
3. Presumpscot	12 - 15	3	13.5	0.115	0.54	1.29
4. Presumpscot	15 - 25	10	20	0.115	0.88	1.63
5. Presumpscot	25-29	4	27	0.115	1.25	2.00

3. Calculate the total settlement of the clays using the consolidation theory:

$$\Delta H_i = C_{r\varepsilon} H_c \log\frac{\sigma'_p}{\sigma'_{v0}} + C_{c\varepsilon} H_c \log\frac{\sigma'_{v0} + \Delta\sigma_v}{\sigma'_p}$$

for overconsolidated clay, $\sigma'_{v0}+\Delta\sigma_v>\sigma'_p$

$$\Delta H_i = C_{r\varepsilon} H_c \log\frac{\sigma'_{v0} + \Delta\sigma_v}{\sigma'_{v0}}$$

for overconsolidated clay, $\sigma'_{v0}+\Delta\sigma_v<=\sigma'_p$

$$\Delta H_i = C_{c\varepsilon} H_c \log\frac{\sigma'_{v0} + \Delta\sigma_v}{\sigma'_{v0}}$$

for normally consolidated clay

$$\Delta\sigma_v = \frac{\Delta\sigma_v}{q} \times q$$

Where:

H_c = initial height (thickness) of layer i, ft
$\Delta\sigma_v$ = surcharge load, psf
σ'_{v0} = in situ vertical effective stress, psf
$\sigma'_{v0}+\Delta\sigma_v$ = (check against σ'_p to select equation)
σ'_p = preconsolidated stress, psf
$C_{r\varepsilon}$ = modified recompression index
$C_{c\varepsilon}$ = modified compression index
ΔH_i = Total settlement

	Clays	Organic Clays
$C_{c\varepsilon}$ =	0.14	0.37
$C_{r\varepsilon}$ =	0.02	

	Layer 1a Organic Clay	Layer 1b Structural Fill Pad	Layer 2 Clay	Layer 3 Clay	Layer 4 Clay	Layer 5 Clay
H_c (ft)	Layer Removed During Construction	Settlement Assumed to Occur Near Instantaneously During Construction	7.0	3.0	10.0	4.0
OCR			3.6	1.8	1.2	1.2
$\Delta\sigma_v$ (psf)			916	558	384	292
σ'_{v0} (psf)			1022	1285	1627	1995
$\sigma'_{v0}+\Delta\sigma_v$ (psf)			1938	1844	2011	2287
σ'_p (psf)			3680	2313	1952	2394
			OC	OC	OC	OC
ΔH_i (ft)			0.037	0.009	0.033	0.005
ΔH_i (in)			0.44	0.11	0.40	0.05

Limit allowable bearing stress to limit total estimated settlements to 1-inch.

	Layer	Settlement ΔH_i (in)
1a. Organic Clay	1a	Removed During Construction
1b. Structural Fill Pad	1b	Negligible
2. Presumpscot	2	0.44
3. Presumpscot	3	0.11
4. Presumpscot	4	0.40
5. Presumpscot	5	0.05
	Total Settlement: (in)	1.00
	Bearing stress, q (ksf)	1.56

Conclusions A):

A service limit state bearing stress of 1.52 may be used to limit settlements to 1-inch, when wall footings over the eastern approach are founded on a minimum 2ft thick structural fill pad bearing on native clay soils. Differential settlements can be expected to be up to half tot total estimated settlement.

East Approach Wall Results

Footing Size	East or West Approach	Service Limit Bearing Capacity (ksf)		
		1" Settlement	2" Settlement	3" Settlement
10ft x 6ft	East Approach	1.56	2.40	3.17
10ft x 8ft	East Approach	1.26	1.99	2.61

Calculations B):

Calculate Time Rate of Consolidation (Clay). Look at an average clay thickness of 22ft for the eastern approach. Assume layer is single drained:

$$t_{90} = \frac{T_{90} H_{dr}^2}{c_v}$$ ref 1, eq. 9-5 $$t_{50} = \frac{T_{50} H_{dr}^2}{c_v}$$

Where:

$H_{dr\ -1}$	22	thickness of drainage layer, (ft)
T_{50}	0.197	time factor for 50% consolidation
T_{90}	0.848	time factor for 90% consolidation
c_v	5.00E-06	coefficient of consolidation from laboratory data, (ft2/sec)
$t_{50\text{-}1}$	1.91E+07	time to reach 50% consolidation, (sec)
$t_{90\text{-}1}$	8.21E+07	time to reach 90% consolidation, (sec)

t_{50}	**0.6**	**years**
t_{50}	**7**	**months**
t_{90}	**2.6**	**years**
t_{90}	**31**	**months**

Conclusions B):

Estimated it will take 2.6 years (31 months) to reach 90% consolidation and 0.6 years (7 months) to reach 50% consolidation within the approximately 22ft thick layer of clay present beneath the eastern approach retaining walls.

Golder Associates	**SUBJECT:** Bearing Capacity & Settlement Proposed Embankment Fill **Project Number:** 1668741 **Project Name:** Marsh Bridge **Prepared by:** LLM/CJS **Checked by:** LLM **Date:** 11/14/2017 **Reviewed by:** MCM

Objective:

To estimate the settlement of the proposed embankment fill for the Marsh Bridge replacement. Assume a 2ft thick triangular rip rap blanket is placed on top of the embankment and organic layer.

Method:

Use the methods described in the AASHTO LRFD Bridge Design Specifications, 7th Ed, 2014 and Holtz & Kovacs to estimate settlement under the proposed fill slope.

References:

1. Holtz, R. D., & Kovacs, W. D. (1981). *An Introduction to Geotechnical Engineering* . Upper Saddle River: Prentice-Hall, Inc.
2. Golder Associates calculation titled, " Organic and Clay Design Parameters Summary," dated 12/5/2017.
3. VHB Preliminary PDR drawing package dated August 17, 2017.

Attachments:

1. Figure - Soil Profile for Proposed Embankment Fill

Assumptions:

1. Ground water table is at the ground surface at top of organic layer. D_w = 0 *ft*
2. Clay consolidation parameters (for Presumpscot Clay) Ccε = 0.14, Crε = 0.02 are based on laboratory data. Consolidation parameters for the organic layer are assumed to be Ccε = 0.37, Crε = 0.12 based on laboratory data. c_v = 5x10^{-6} ft^2/s (for Presumpscot Clay). cv = 4x10^{-7} ft^2/s (for Organic Layer). Refer to Reference 2.
3. Assume organic layer at toe starts at ~ EL. 0.

Calculations A):

Use consolidation theory to estimate settlement of the organic and clay layers. For this case (riprap blanket), the organic layer will remain in place.

1. Subdivide the subsurface soils into layers no larger than 10 ft thick and to a depth two times the footing width. Assume embankment stress distribution method (Fig 8.23, Ref 1). Based on the geometry of the triangular embankment and placement of the riprap blanket over the exterior sideslope (refer to sketch). Use stress distribution for a 2ft tall triangular embankment load.

$$\frac{\sigma_z}{q_o} = I$$ (Holtz & Kovacs, Figure 8.23)

where:

q_o = Unit load of embankment
a = Width of sloped part of embankment — a = 15.4 *ft*
b = Width of flat part of embankment — b = 0 *ft*
Z = depth to mid point of layer, ft

Layer		Depth Below Ground Surface (ft)	Thickness (ft)	Influence factor for embankment load: z (ft)	a/z	I $(\Delta\sigma_z)/q_o$
Organic Clay	1	0 - 5	5	2.5	6.2	0.48
Presumpscot	2	5 - 12	7	8.5	1.8	0.34
Presumpscot	3	12 - 15	3	13.5	1.1	0.26
Presumpscot	4	15 - 25	10	20	0.8	0.22
Presumpscot	5	25-35	10	30	0.5	0.14

Layer	Depth Below Ground Surface (ft)	Layer Thickness (ft)	Z (ft)	Unit Weight of Layer (ksf)	σ'_{v0} - Under the Toe (ksf)
1. Organic Clay	0 - 5	5	2.5	0.080	0.04
2. Presumpscot	5 - 12	7	8.5	0.115	0.27
3. Presumpscot	12 - 15	3	13.5	0.115	0.54
4. Presumpscot	15 - 25	10	20	0.115	0.88
5. Presumpscot	25-35	10	30	0.115	1.40

3. Calculate the total settlement of the organic and clays using the consolidation theory:

$$\Delta H_i = C_{r\varepsilon} H_c \log\frac{\sigma'_p}{\sigma'_{v0}} + C_{c\varepsilon} H_c \log\frac{\sigma'_{v0} + \Delta\sigma_v}{\sigma'_p}$$

for overconsolidated clay, $\sigma'_{v0}+\Delta\sigma_v>\sigma'_p$

$$\Delta H_i = C_{r\varepsilon} H_c \log\frac{\sigma'_{v0} + \Delta\sigma_v}{\sigma'_{v0}}$$

for overconsolidated clay, $\sigma'_{v0}+\Delta\sigma_v<=\sigma'_p$

$$\Delta H_i = C_{c\varepsilon} H_c \log\frac{\sigma'_{v0} + \Delta\sigma_v}{\sigma'_{v0}}$$

for normally consolidated clay

$$\Delta\sigma_v = \frac{\Delta\sigma_v}{q} \times q$$

Where:

H_c = initial height (thickness) of layer *i* , ft
$\Delta\sigma_v$ = surcharge load, psf
σ'_{v0} = in situ vertical effective stress, psf
$\sigma'_{v0}+\Delta\sigma_v$ = (check against σ'_p to select equation)
σ'_p = preconsolidated stress, psf
$C_{r\varepsilon}$ = modified recompression index
$C_{c\varepsilon}$ = modified compression index
ΔH_i = Total settlement

	Clays	Organic Clays
$C_{c\varepsilon}$ =	0.14	0.37
$C_{r\varepsilon}$ =	0.02	

	Layer 1 Organic Clay	Layer 2 Clay	Layer 3 Clay	Layer 4 Clay	Layer 5 Clay
H_c (ft)	5.0	7.0	3.0	10.0	10.00
OCR	1.0	3.6	1.8	1.2	1.20
$\Delta\sigma_v$ (psf)	134	95	71	62	40
σ'_{v0} (psf)	44	272	535	877	1403
$\sigma'_{v0} + \Delta\sigma_v$ (psf)	178	367	607	939	1443
σ'_p (psf)	44	980	963	1052	1684
	NC	OC	OC	OC	OC
ΔH_i (ft)	1.125	0.017	0.003	0.006	0.002
ΔH_i (in)	13.50	0.21	0.04	0.07	0.03

RIPRAP SLOPE	Layer	Settlement ΔH_i (in)		
1. Organic Clay	1	13.50		
2. Presumpscot	2	0.21		
3. Presumpscot	3	0.04		
4. Presumpscot	4	0.07	0.34	estimated settlement if over excavate organic layer = sum of clay layers. Assumed settlement in structural fill is negligible
5. Presumpscot	5	0.03		
	Total Settlement: (in)	13.84		
	Bearing stress, q (ksf)	0.28	0.140kcf *2ft	(load increase - 2ft RipRap fill)

Conclusions A):

A service limit state settlement of 13.8-inches is estimated for a 2ft thick riprap blanket on the embankment slopes and founded directly on the organic layer.

Calculations B):

Estimate Time Rate of Consolidation (Peat Only). Assume layer is singley drained:

$$t_{90} = \frac{T_{90} H_{dr}^2}{c_v}$$ ref 1, eq. 9-5

Where:

H_{dr}	5	thickness of drainage layer, (ft)
T_{50}	0.197	time factor for 50% consolidation
T_{90}	0.848	time factor for 90% consolidation
c_v	4.00E-07	coefficient of consolidation from laboratory data, (ft2/sec)
$t_{50\text{-}1}$	1.23E+07	time to reach 50% consolidation, (sec)
$t_{90\text{-}1}$	5.30E+07	time to reach 90% consolidation, (sec)

t_{50}	**0.4**	**years**
t_{50}	**5**	**months**
t_{90}	**1.7**	**years**
t_{90}	**20**	**months**

Conclusions B):

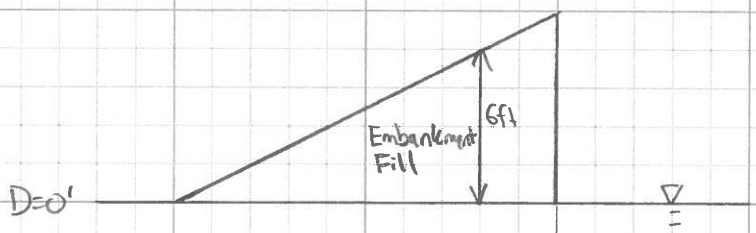
Estimated it will take 2.6 years (20 months) to reach 90% consolidation and 0.4 years (5 months) to reach 50% consolidation within the approximately 5ft thick layer of organic material beneath the proposed embankment fill.

Golder Associates

SUBJECT	Soil Profile for West Walls	
Job No. 1668741 Ref.	Made by CJS Checked Reviewed	Date 12/1/17 Sheet 1 of 1

not to scale

Existing Conditions
(use to calculate in situ effective stresses)



1) Organic Silts & Clays

$\gamma = 80$ pcf, OCR = 1

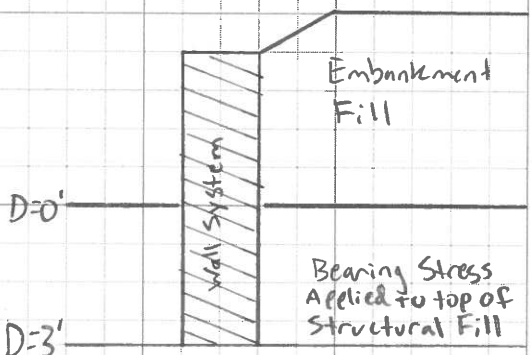
D=5'

2) Presumpscot

$\gamma = 115$ pcf, OCR = 3.6

D=11'

Proposed Conditions
(use to calculate settlement)



1b) Structural Fill PAD

D=5'

2) Presumpscot

$\gamma = 115$ pcf, OCR = 3.6

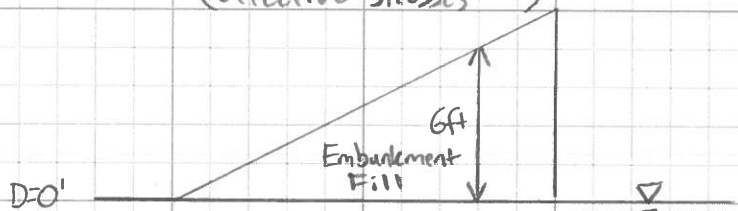
D=11'

Golder Associates

SUBJECT	Soil Profile for East Walls	
Job No. 1668741 Ref.	Made by CJS Checked Reviewed	Date 12/1/17 Sheet 1 of 1

not to scale

Existing Conditions
(use to calculate in situ effective stresses)



1) Organic Silts & Clays
$\gamma = 80$ pcf, OCR = 1

D=5'

2) Presumpscot
$\gamma = 115$ pcf, OCR = 3.6

D=12'

3) Presumpscot
$\gamma = 115$ pcf, OCR = 1.8

D=15'

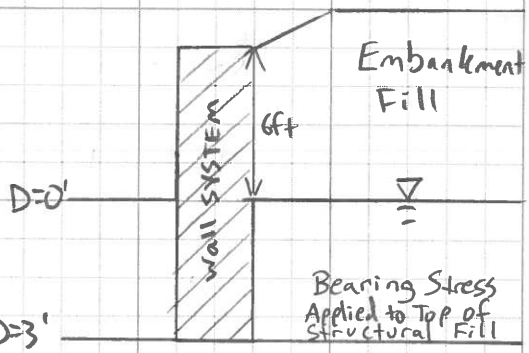
4) Presumpscot
$\gamma = 115$ pcf, OCR = 1.2

D=25'

5) Presumpscot
$\gamma = 115$ pcf, OCR = 1.2

D=29'

Proposed Conditions
(use to calculate settlement)



1b) Structural Fill PAD

D=5'

2) Presumpscot
$\gamma = 115$ pcf, OCR = 3.6

D=12'

3) Presumpscot
$\gamma = 115$ pcf, OCR = 1.8

D=15'

4) Presumpscot
$\gamma = 115$ pcf, OCR = 1.2

D=25'

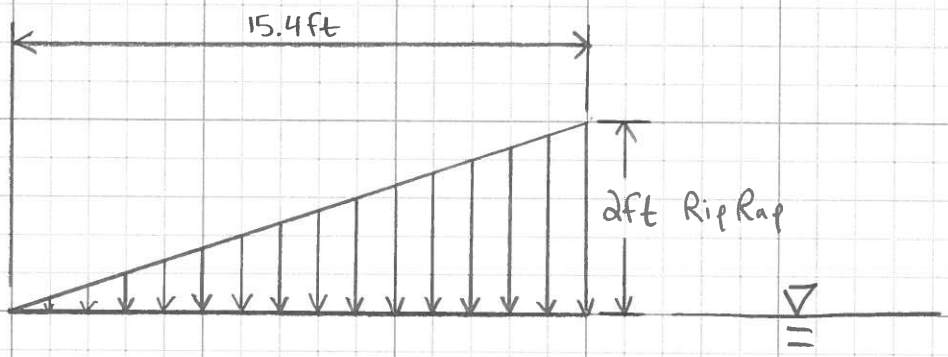
5) Presumpscot
$\gamma = 115$ pcf OCR = 1.2

D=29'

Golder Associates

SUBJECT	Soil Profile for Proposed Embankment Fill	
Job No. 1668741 Ref.	Made by CJS Checked Reviewed	Date 12/1/2017 Sheet 1 of 1

not to scale



D=0'

1) Organic Silts & Clays
γ = 80pcf, OCR = 1

D=5'

2) Presumpscot
γ = 115pcf, OCR = 3.6

D=12'

3) Presumpscot
γ = 115pcf, OCR = 1.8

D=15'

4) Presumpscot
γ = 115pcf, OCR = 1.2

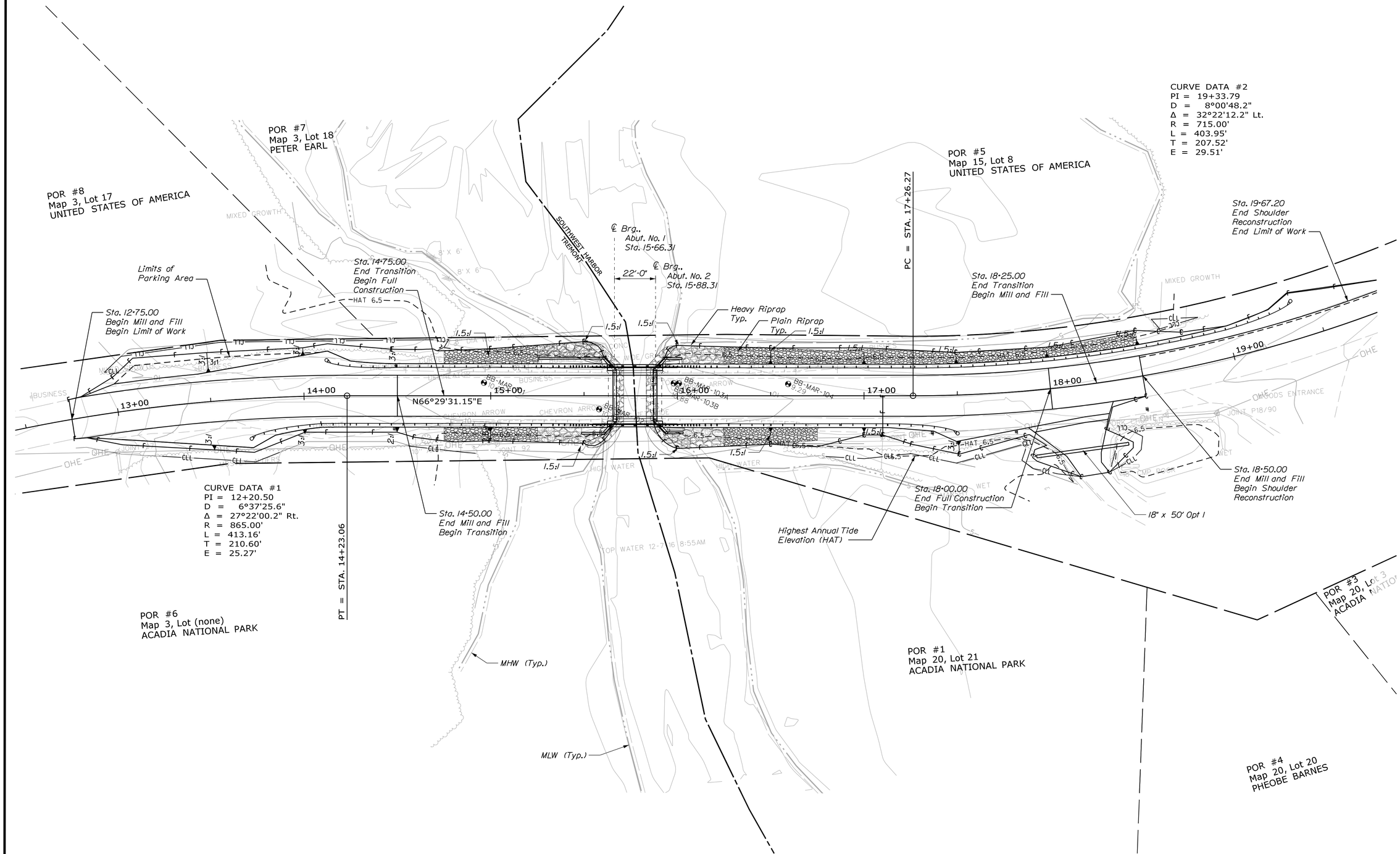
D=25'

5) Presumpscot
γ = 115pcf, OCR = 1.2

D=35'

APPENDIX F

VHB PRELIMINARY DESIGN DRAWINGS
8/17/17



PRELIMINARY
NOT FOR CONSTRUCTION
08/17/2017

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2170(200)

WIN 21702.00

BRIDGE NO. 2511

BRIDGE PLANS

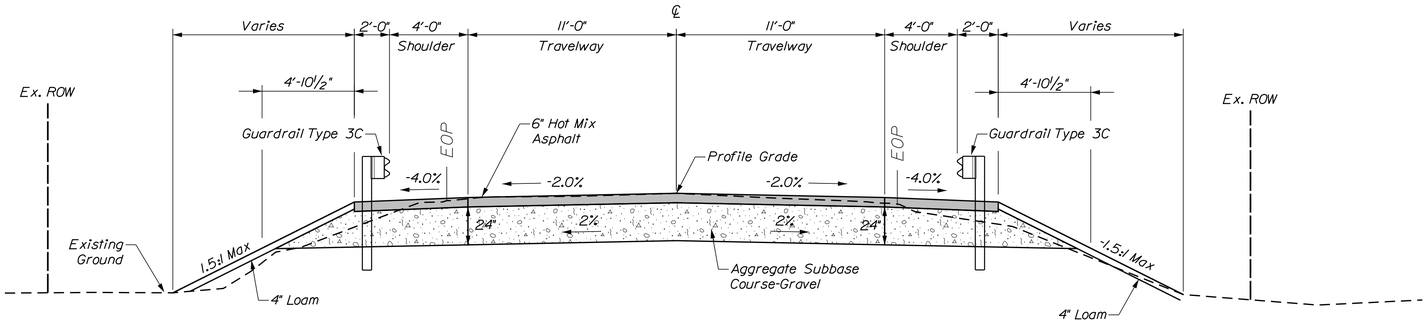
	BY	DATE
PROJ. MANAGER	L. TIMBERLAKE	
DESIGN-DETAILED	TJC	DPD
CHECKED-REVIEWED	OME	RSBLUNT
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

MARSH BRIDGE
BASS HARBOR ROAD OVER MARSHALL BROOK
TREMONT/SOUTHWEST HARBOR HANCOCK COUNTY

SHEET NUMBER
1
OF 12

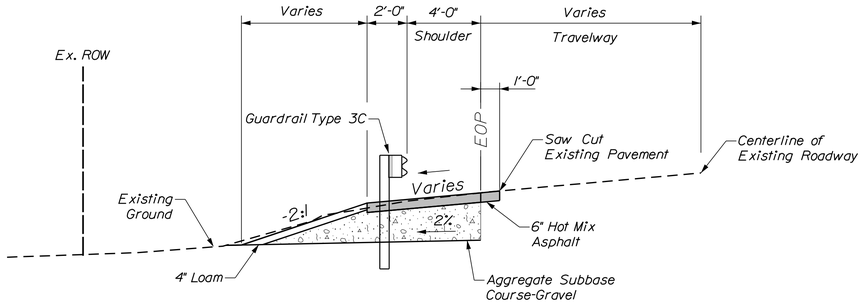
PRELIMINARY
NOT FOR CONSTRUCTION
08/17/2017

Filename: ...\MSTA\001_GeneralPlan1.dgn Username: PDonovan Date:8/16/2017 Division: BRIDGE



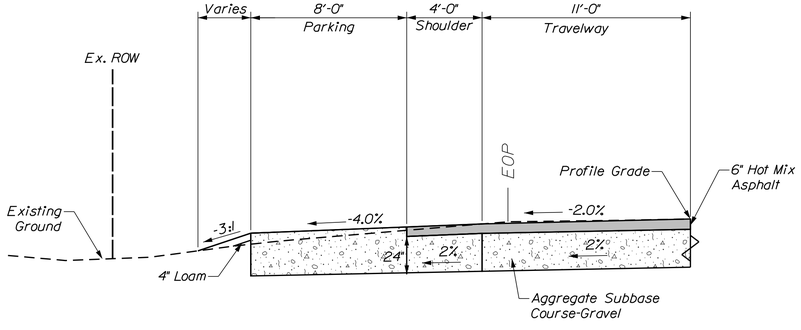
TYPICAL ROADWAY SECTION

AGGREGATE SUBBASE COURSE GRAVEL		
LEFT SHOULDER 58.6 CY/100 LF	11' TRAVEL LANES 81.5 CY/100LF	RIGHT SHOULDER 51.6 CY/100 LF
STATION TO STATION	STATION TO STATION	STATION TO STATION



TYPICAL GUARDRAIL/SHOULDER RECONSTRUCTION SECTION

SHOULDER W/ GUARDRAIL 24" A.S.C.G = 57.4 CY/100 LF
STATION TO STATION



TYPICAL ROADWAY SECTION AT PARKING AREA

PARKING AREA 24" A.S.C.G = 96.6 CY/100 LF
STATION TO STATION

TYPICAL SECTION NOTES

1. The pavement and subbase depths as shown on the plans are nominal
2. Crowns for all courses of subbase and pavement shall be straight
3. All necessary pavement cutting shall be saw cut and done in such a manner as to leave a clean vertical face
4. Bituminous tack coat is required between all lifts of pavement and on all existing paved or milled surfaces prior to placing proposed pavement or as directed by the resident
5. All guardrail shall be type 3c, single rail with 7' posts
6. EOP = Existing Edge of Pavement

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2170(200)

WIN 21702.00

BRIDGE NO. 2511

BRIDGE PLANS

PRELIMINARY
NOT FOR
CONSTRUCTION
08/17/2017

PROJ. MANAGER	L. TIMBERLAKE		
DESIGN-DETAILED	TJC	DPD	--------
CHECKED-REVIEWED	CME	RSBLUNT	--------
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

MARSH BRIDGE
BASS HARBOR ROAD OVER MARSHALL BROOK
TREMONT/SOUTHWEST HARBOR HANCOCK COUNTY

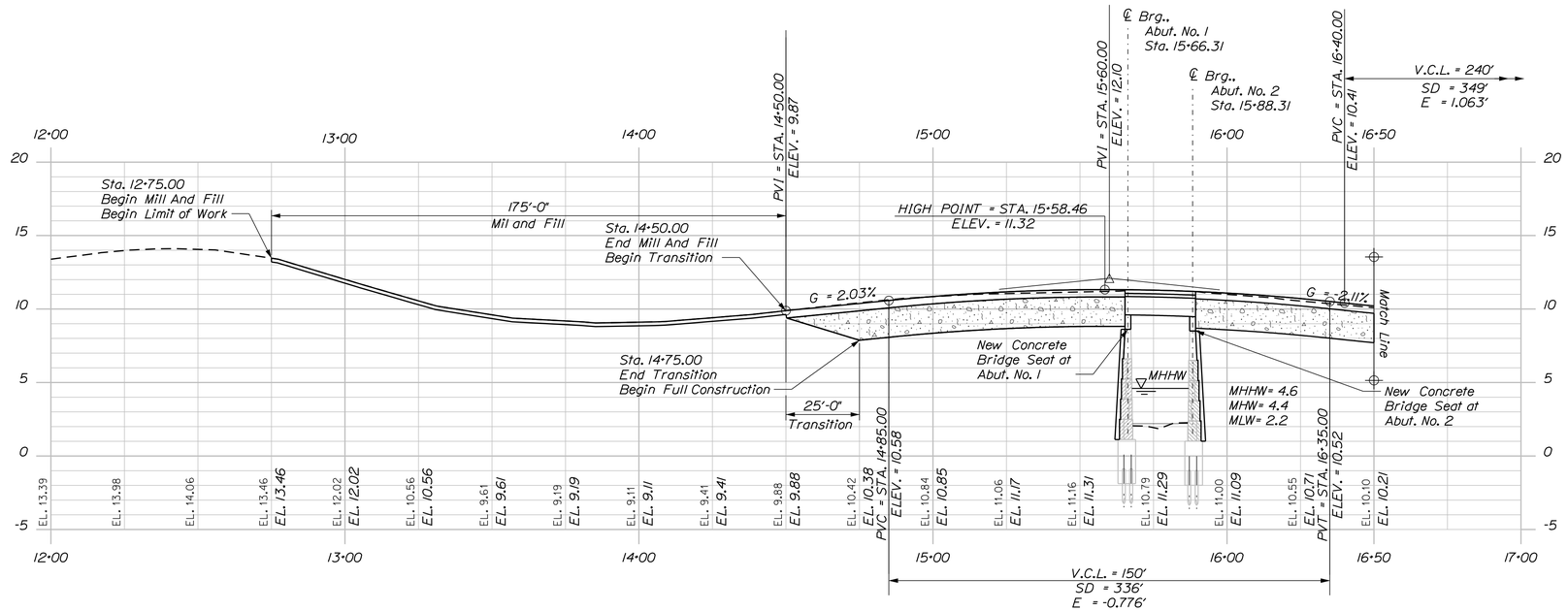
ROADWAY TYPICALS

SHEET NUMBER

2

OF 12

Filename: ...\BRIDGE\MSTA\002_Typical_ 1.dgn Division: BRIDGE Username: PDonovan Date:8/16/2017



MARSH BRIDGE OVER MARSH BROOK PROFILE

Horiz. 25 0 25 50

Vert. 5 0 5 10

Scale of Feet

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2170(200)

WIN 21702.00

BRIDGE NO. 2511 — BRIDGE PLANS

PRELIMINARY
NOT FOR
CONSTRUCTION
08/17/2017

	L. TIMBERLAKE	BY	DATE
PROJ. MANAGER			
DESIGN-DETAILED	TJC	DPD	--------
CHECKED-REVIEWED	GME	RSBLUNT	--------
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

MARSH BRIDGE
BASS HARBOR ROAD OVER MARSHALL BROOK
TREMONT/SOUTHWEST HARBOR HANCOCK COUNTY

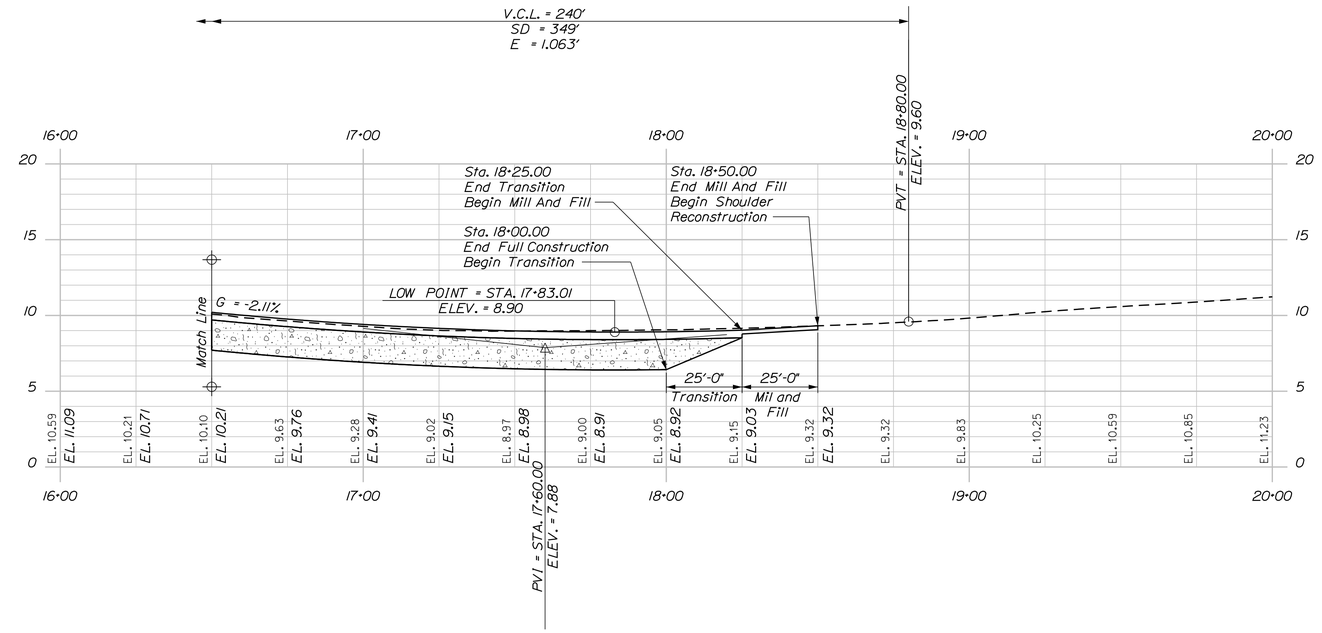
PROFILE (1 OF 2)

SHEET NUMBER

3

OF 12

Filename: ...\BRIDGE\MSTA\003_Profile_01.dgn Division: BRIDGE Username: PDonovan Date:8/16/2017



MARSH BRIDGE OVER MARSH BROOK PROFILE

Horiz. 25 0 25 50

Vert. 5 0 5 10

Scale of Feet

Username: PDonovan Date:8/16/2017

Filename: ...\BRIDGE\MSTA\004_Profile_02.dgn Division: BRIDGE

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2170(200)

WIN 21702.00

BRIDGE NO. 2511 BRIDGE PLANS

PRELIMINARY
NOT FOR
CONSTRUCTION
08/17/2017

PROJ. MANAGER	L. TIMBERLAKE	BY	DATE
DESIGN-DETAILED	TJC	DPD	-------
CHECKED-REVIEWED	OME	RSBLUNT	-------
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

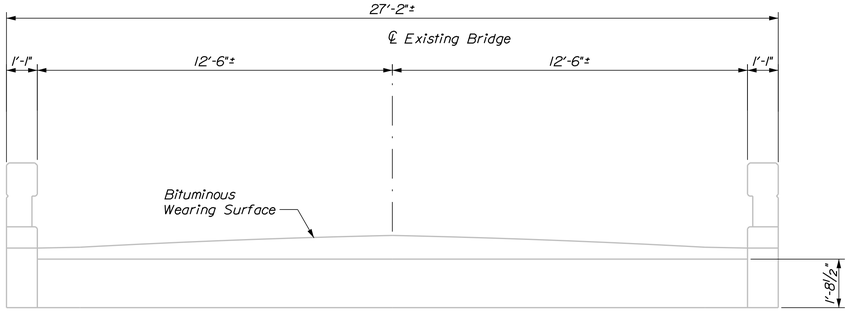
MARSH BRIDGE
BASS HARBOR ROAD OVER MARSHALL BROOK
TREMONT/SOUTHWEST HARBOR HANCOCK COUNTY

PROFILE (2 OF 2)

SHEET NUMBER

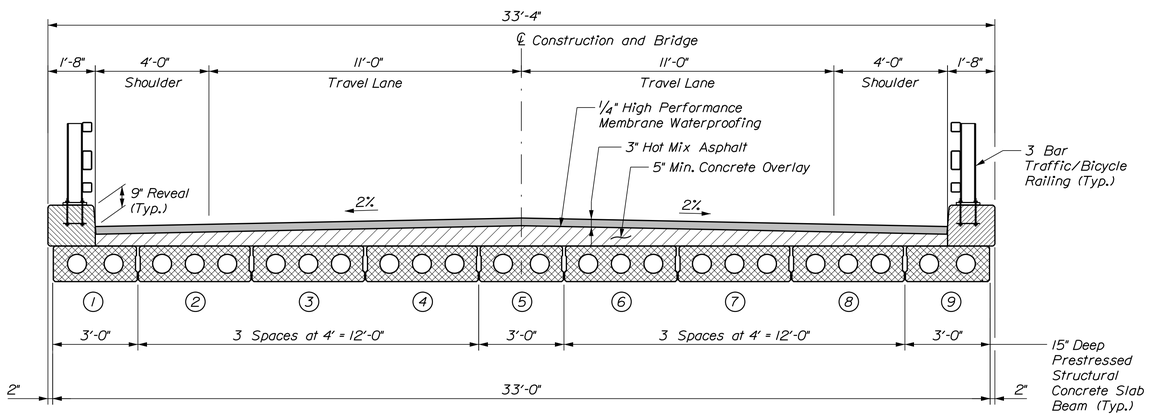
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OF 12



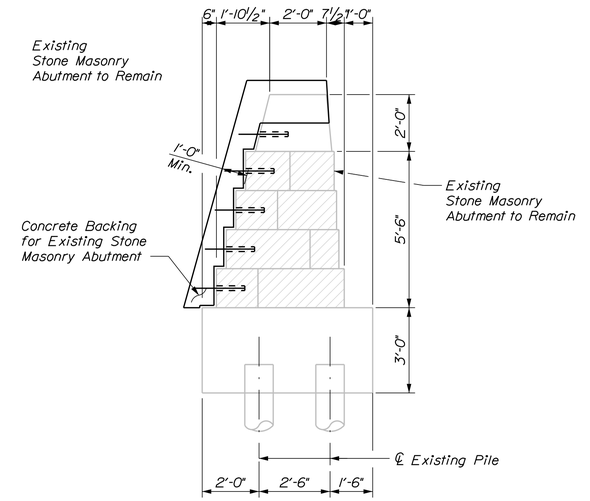
EXISTING SUPERSTRUCTURE SECTION

Scale: 3/8" = 1'-0"



PROPOSED SUPERSTRUCTURE SECTION

(Looking Upstation)
(Reinforcing not shown)
Scale: 3/8" = 1'-0"



PROPOSED ABUTMENT SECTION

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2170(200)

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08/17/2017

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CHECKED-REVIEWED	RSBLUNT	-------
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

MARSH BRIDGE
BASS HARBOR ROAD OVER MARSHALL BROOK
TREMONT/SOUTHWEST HARBOR HANCOCK COUNTY

SHEET NUMBER

6

OF 12

Filename: ...\MST\A\006_Btypical_01.dgn Division: BRIDGE Username: PDonovan Date:8/16/2017

VERTICAL DATUM NAVD88

Flood — Ebb

MARSHALL BROOK

℄ Exterior Beam Brg. Abut. No. 1

℄ Brg. Abut. No. 1 (Fixed)

℄ Exterior Beam Brg. Abut. No. 2

℄ Brg. Abut. No. 2 (Fixed)

Heavy Riprap Typ.

Plain Riprap Typ.

1.5:1

1'-5"

22'-0"

1'-8"

4'-0"

11'-0"

33'-4" Out to Out

Face of Curb

Face of Rail

90°0'0" (Typ.)

Bass Harbor Road

To Bass Harbor

15+00

16+00

℄ Construction

To Southwest Harbor

Proposed Retaining Wall (Typ.)

Remove Existing Block Retaining Wall (Typ.)

OHE

MARSHALL BROOK

PLAN

3/16" = 1'-0"

℄ Brg. Abut. No. 1 (Fixed)

3 Bar Traffic/Bicycle Railing (Typ.)

℄ Brg. Abut. No. 2 (Fixed)

Precast Concrete Roadway Transition Curb (Typ.)

Bridge Rail Transition (Typ.)

New Superstructure

1.5 : 1

4.6 MHHW

4.4 MHW

2.2 MLW

4' Riprap Along Abutments Typ.

Approxmate Channel Bed

Modified Abutment (Typ.)

Existing Block Wingwall Wall With Concrete Extension

ELEVATION

1 0 15

3/16" = 1'-0"

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2170(200)

WIN 21702.00

BRIDGE NO. 2511

BRIDGE PLANS

PRELIMINARY
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08/17/2017

	BY	DATE
PROJ. MANAGER L. TIMBERLAKE		
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CHECKED-REVIEWED	RSBLUNT	-------
DESIGN2-DETAILED2		
DESIGN3-DETAILED3		
REVISIONS 1		
REVISIONS 2		
REVISIONS 3		
REVISIONS 4		
FIELD CHANGES		

MARSH BRIDGE
BASS HARBOR ROAD OVER MARSHALL BROOK
TREMONT/SOUTHWEST HARBOR HANCOCK COUNTY

BRIDGE PLAN AND ELEVATION

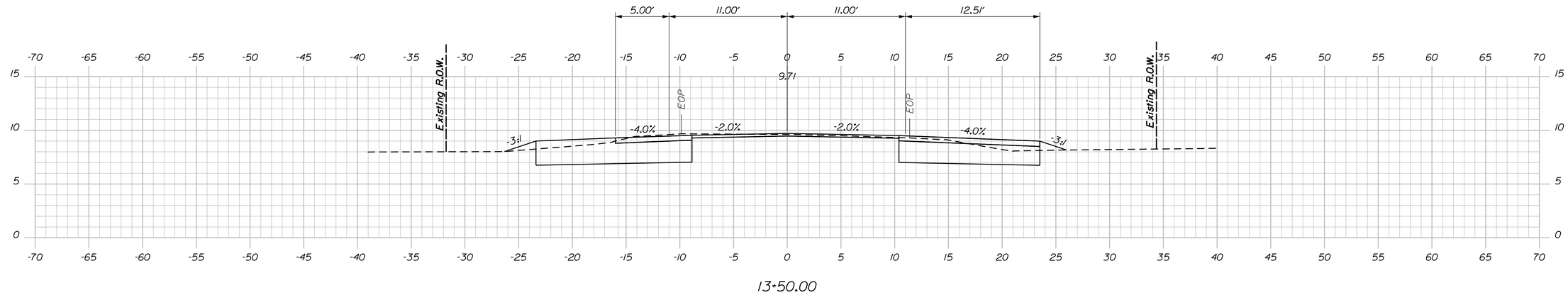
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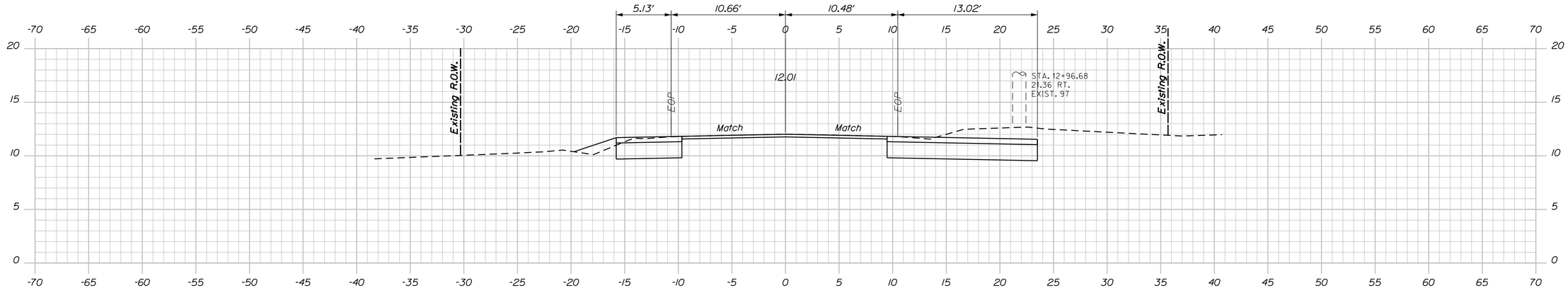
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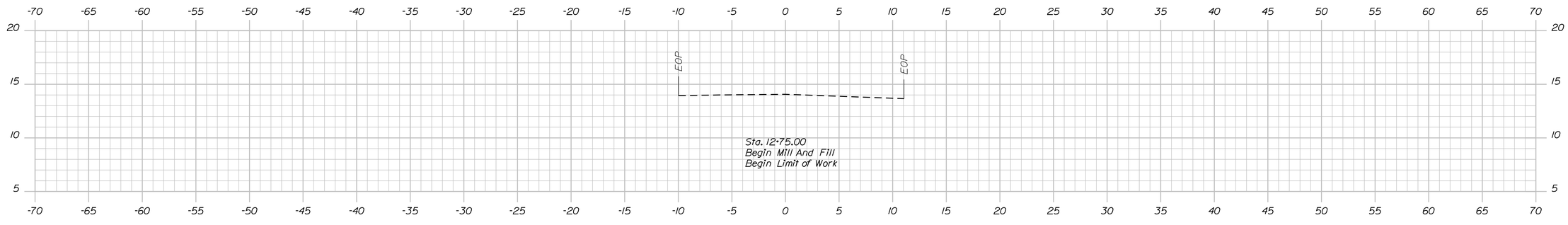
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13+50.00



13+00.00



12+50.00

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2170(200)

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BRIDGE NO. 2511

BRIDGE PLANS

PRELIMINARY
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CONSTRUCTION
08/17/2017

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CHECKED-REVIEWED	DME	RSBLUNT	-------
DESIGN2-DETAILED2			
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REVISIONS 1			
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REVISIONS 4			
FIELD CHANGES			

MARSH BRIDGE
BASS HARBOR ROAD OVER MARSHALL BROOK
TREMONT/SOUTHWEST HARBOR HANCOCK COUNTY

SECTIONS

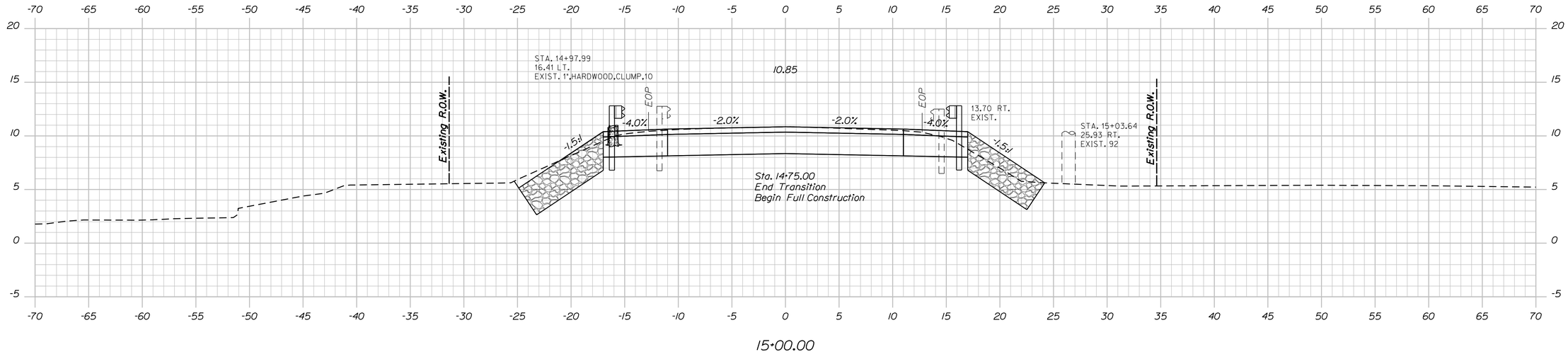
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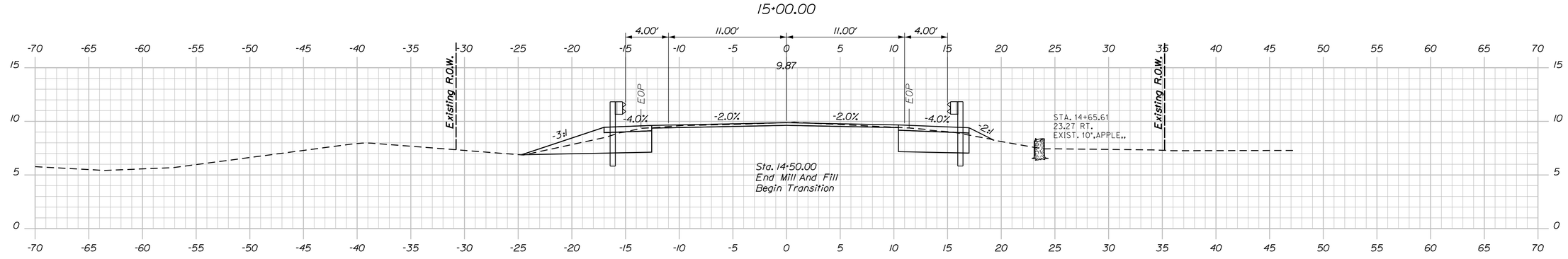
OF 12

Sta. 12+00.00 to Sta. 13+50.00

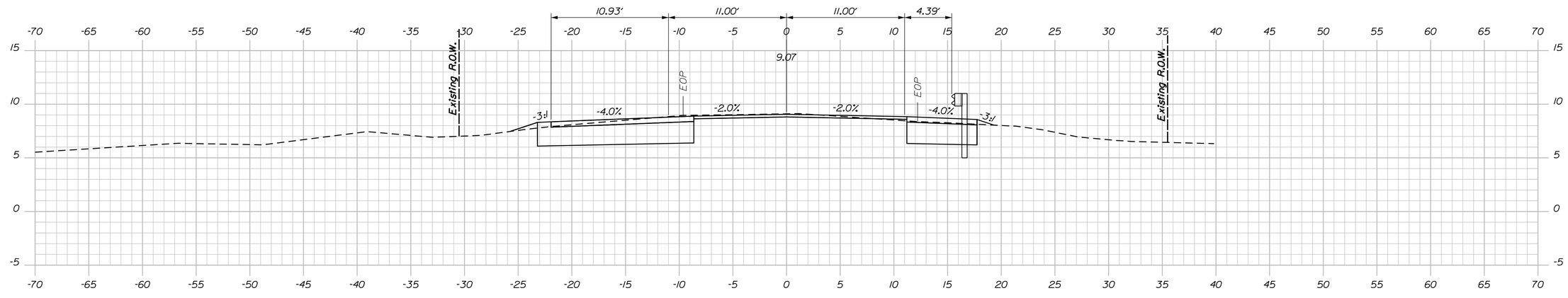
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15+00.00



14+50.00



14+00.00

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2170(200)

WIN 21702.00

BRIDGE NO. 2511

BRIDGE PLANS

PRELIMINARY
NOT FOR
CONSTRUCTION
08/17/2017

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CHECKED-REVIEWED	DME	RSBLUNT	-------
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

MARSH BRIDGE
BASS HARBOR ROAD OVER MARSHALL BROOK
TREMONT/SOUTHWEST HARBOR HANCOCK COUNTY

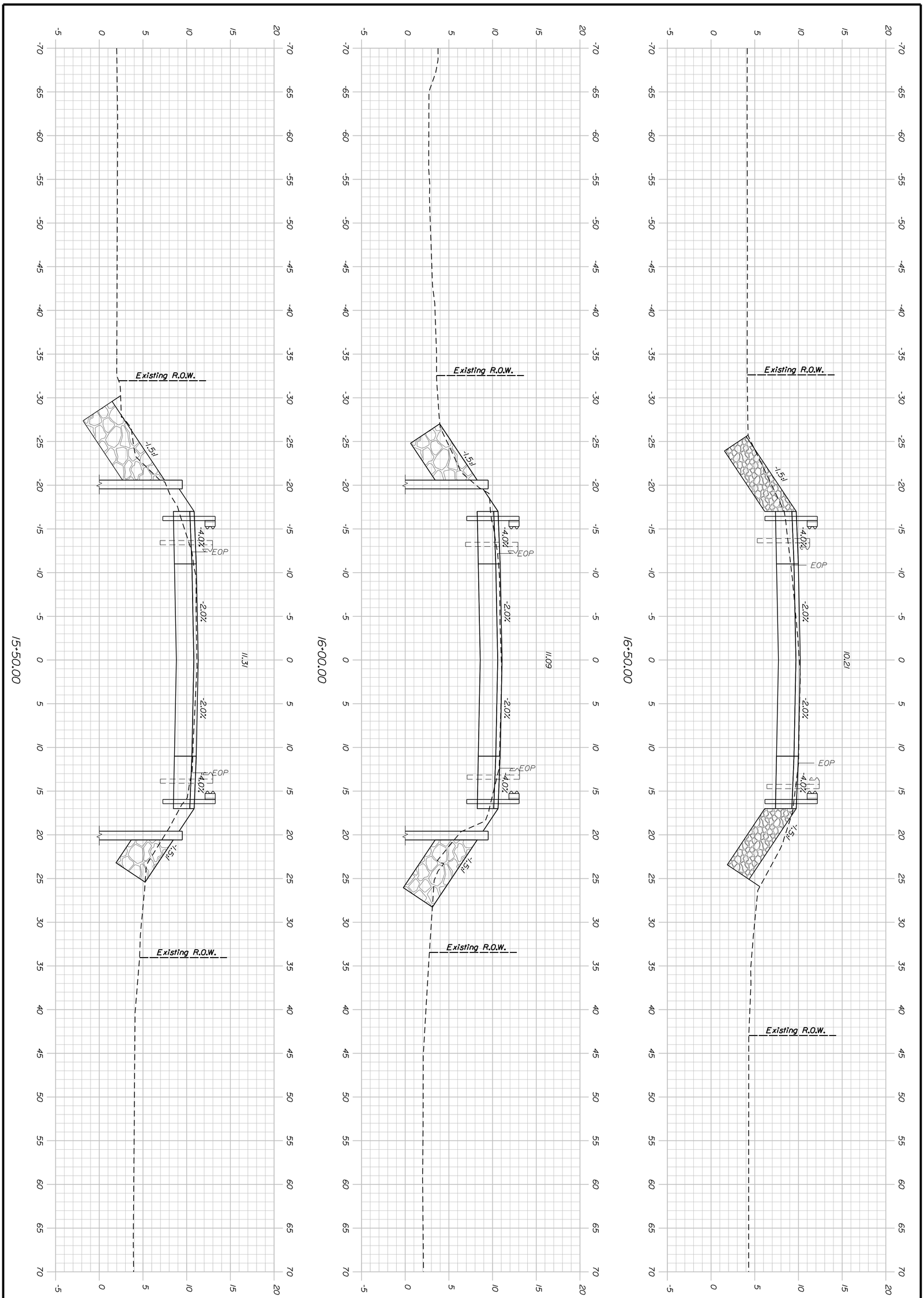
SECTIONS

SHEET NUMBER

8

OF 12

Sta. 14+00.00 to Sta. 15+00.00



Sta. 15+50.00 to Sta. 16+50.00

SHEET NUMBER 9 OF 12

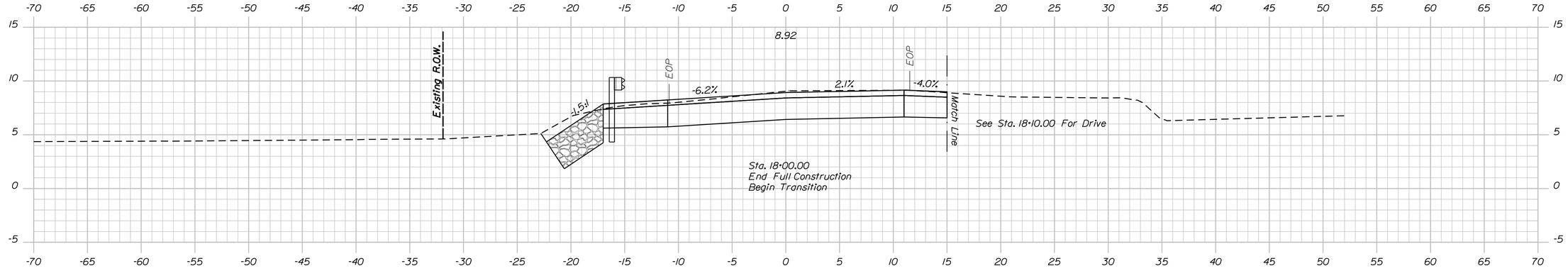
MARSH BRIDGE
BASS HARBOR ROAD OVER MARSHALL BROOK
TREMONT/SOUTHWEST HARBOR HANCOCK COUNTY

SECTIONS

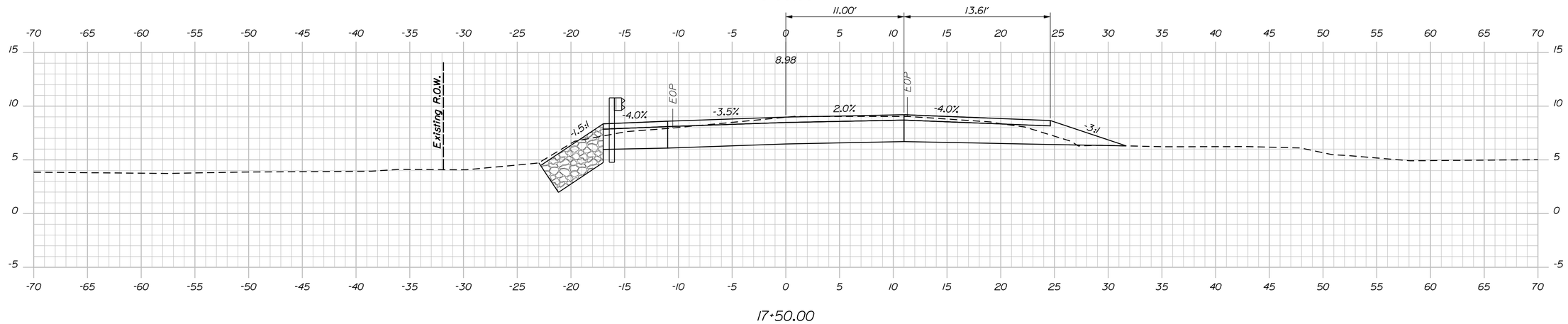
PROJ. MANAGER	L. TIMBERLAKE	BY	DATE
DESIGN-DETAILED	TJC	DPD	--------
CHECKED-REVIEWED	GME	RSBLUNT	--------
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

PRELIMINARY
NOT FOR CONSTRUCTION
08/17/2017

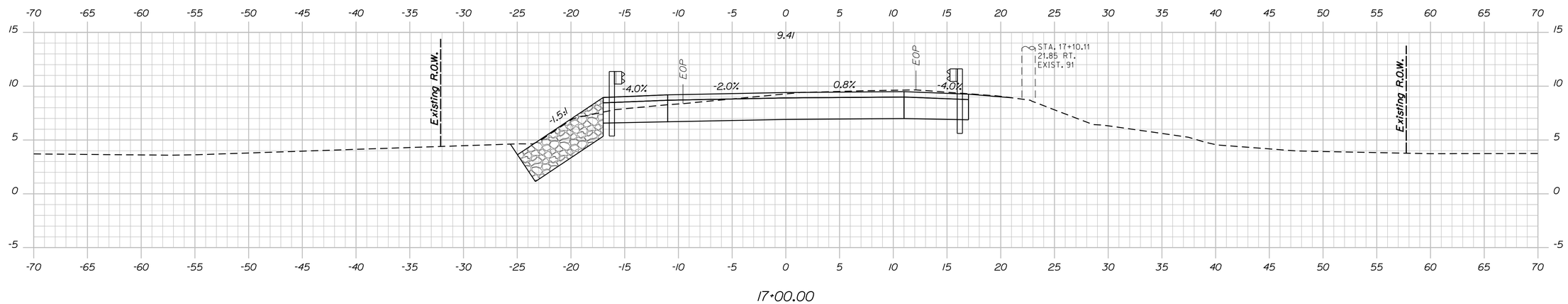
STATE OF MAINE
DEPARTMENT OF TRANSPORTATION
STP-2170(200)
BRIDGE NO. 2511 WIN 21702.00 BRIDGE PLANS



18+00.00



17+50.00



17+00.00

Sta. 17+00.00 to Sta. 18+00.00

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2170(200)

WIN 21702.00

BRIDGE NO. 2511

BRIDGE PLANS

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CHECKED-REVIEWED	OME	RSBLUNT	-------
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

MARSH BRIDGE
BASS HARBOR ROAD OVER MARSHALL BROOK
TREMONT/SOUTHWEST HARBOR HANCOCK COUNTY

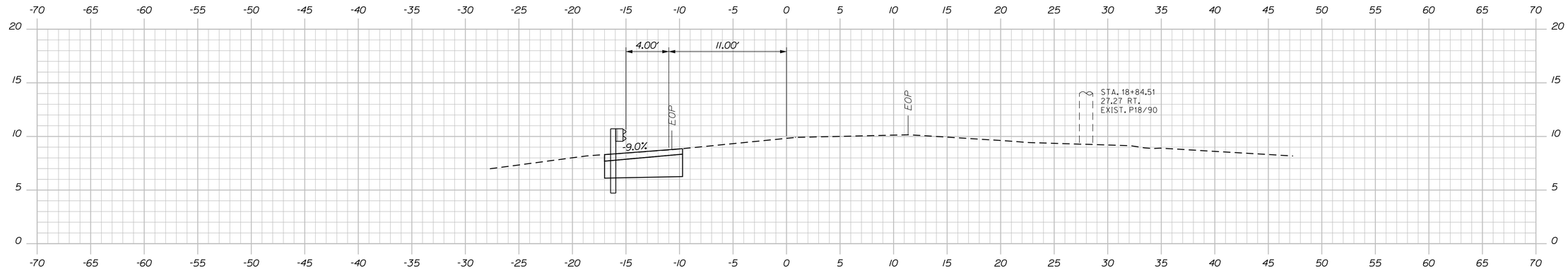
SECTIONS

SHEET NUMBER

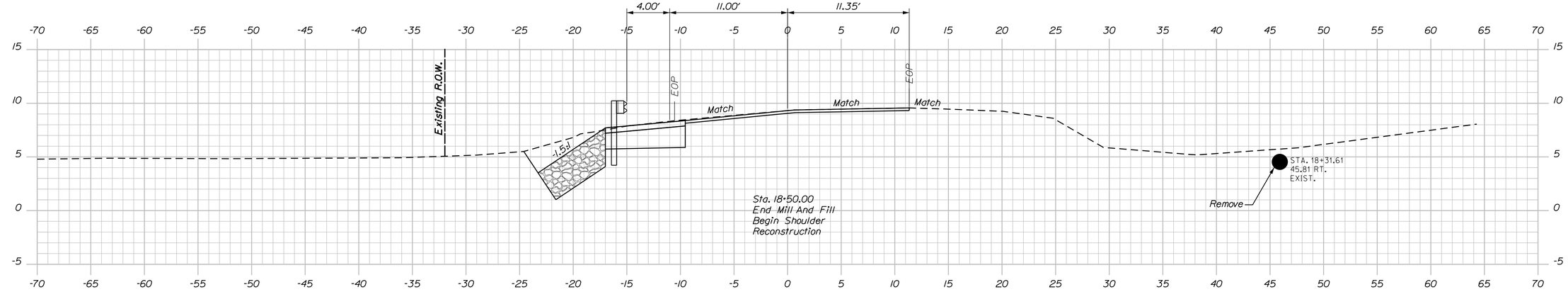
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OF 12

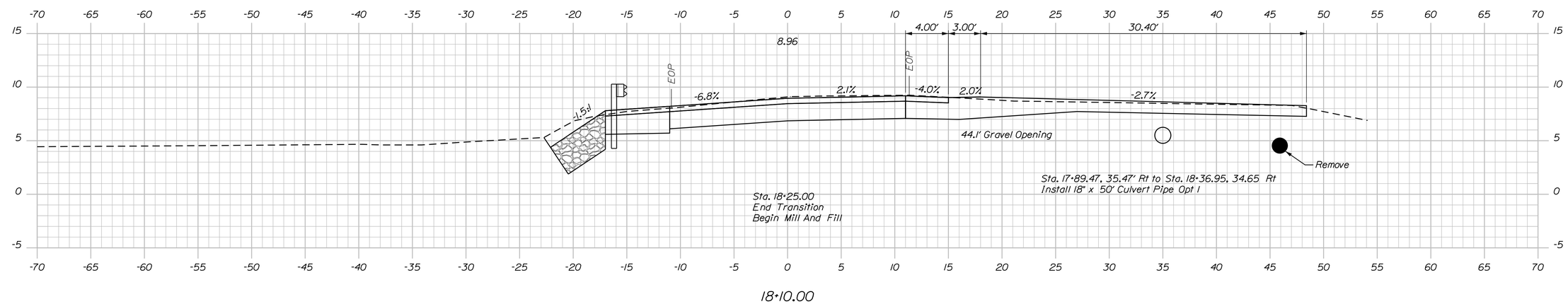
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19+00.00



18+50.00



18+10.00

Sta. 18+10.00 to Sta. 19+00.00

STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2170(200)

WIN 21702.00

BRIDGE NO. 2511

BRIDGE PLANS

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08/17/2017

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CHECKED-REVIEWED	GME	RSBLUNT	-------
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DESIGN3-DETAILED3			
REVISIONS 1			
REVISIONS 2			
REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

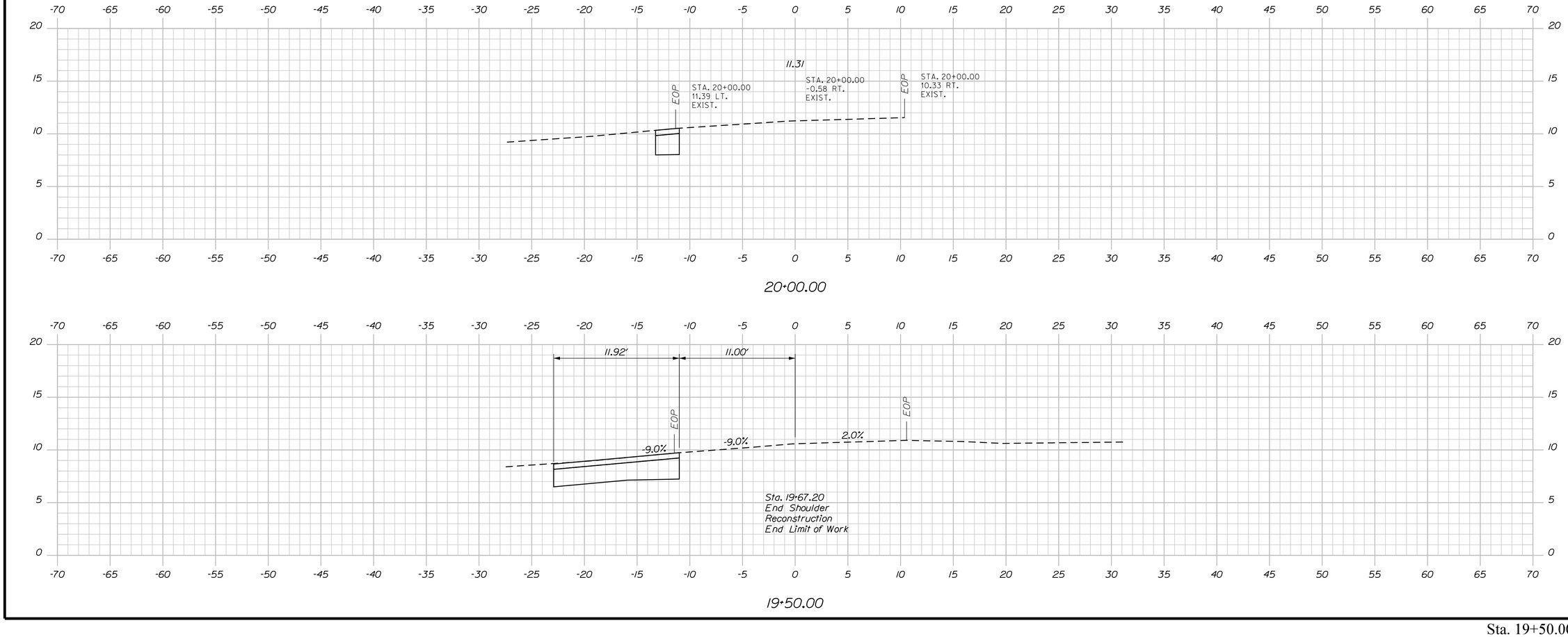
MARSH BRIDGE
BASS HARBOR ROAD OVER MARSHALL BROOK
TREMONT/SOUTHWEST HARBOR HANCOCK COUNTY

SECTIONS

SHEET NUMBER
11
OF 12

Filename: ...\MSTA\011_Xsect_18+10.dgn Division: BRIDGE Username: PDonovan Date:8/16/2017

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STATE OF MAINE
DEPARTMENT OF TRANSPORTATION

STP-2170(200)

WIN 21702.00

BRIDGE NO. 2511

BRIDGE PLANS

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PROJ. MANAGER	L. TIMBERLAKE		
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CHECKED-REVIEWED	OME	RSBLUNT	-------
DESIGN2-DETAILED2			
DESIGN3-DETAILED3			
REVISIONS 1			
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REVISIONS 3			
REVISIONS 4			
FIELD CHANGES			

MARSH BRIDGE
BASS HARBOR ROAD OVER MARSHALL BROOK
TREMONT/SOUTHWEST HARBOR HANCOCK COUNTY

SECTIONS

SHEET NUMBER

12

OF 12

Sta. 19+50.00 to Sta. 20+00.00

APPENDIX G
1931 MARSH BRIDGE DESIGN PLANS

NOTES

Existing Superstructure:-
Five spans with six 4"x 6" stringers and 3" transverse planking in fair but weak condition.

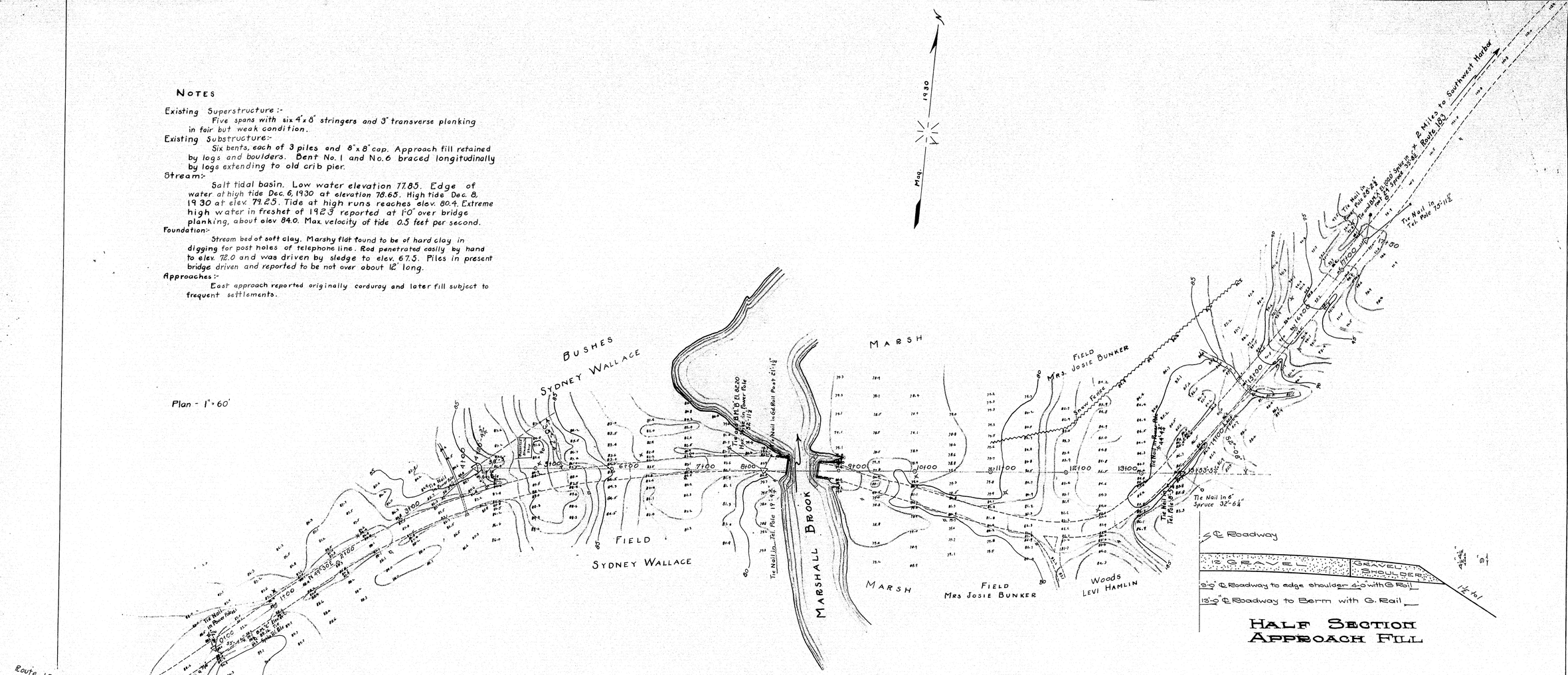
Existing Substructure:-
Six bents, each of 3 piles and 8"x 8" cap. Approach fill retained by logs and boulders. Bent No. 1 and No. 6 braced longitudinally by logs extending to old crib pier.

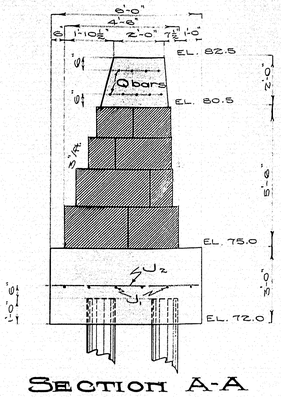
Stream:-
Salt tidal basin. Low water elevation 77.85. Edge of water at high tide Dec. 6, 1930 at elevation 78.65. High tide Dec. 8, 1930 at elev. 79.25. Tide at high runs reaches elev. 80.4. Extreme high water in freshet of 1923 reported at 1'0" over bridge planking, about elev 84.0. Max. velocity of tide 0.5 feet per second.

Foundation:-
Stream bed of soft clay. Marshy flat found to be of hard clay in digging for post holes of telephone line. Rod penetrated easily by hand to elev. 72.0 and was driven by sledge to elev. 67.5. Piles in present bridge driven and reported to be not over about 12" long.

Approaches:-
East approach reported originally corduroy and later fill subject to frequent settlements.

Plan - 1" = 60'

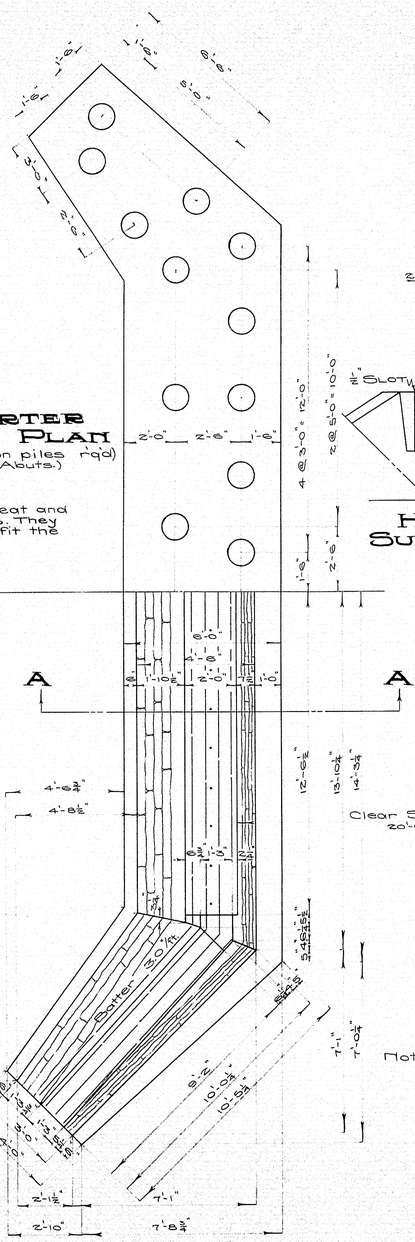

1930
Mag.
Bushes
Sydney Wallace
Marsh
Field
Mrs. Josie Bunker
Brook
Marshall Brook
Woods
Levi Hamlin
Route 183 1¼ Miles to Tremont
1¼ Miles to Pretty Marsh
2 Miles to Southwest Harbor
Tie Nail in 6" Spruce 32°-64'
Tie Nail in Tel. Pole 13°-11'
Tie Nail in Tel. Pole 26°-23'
Tie Nail in 6" Spruce 35°-0'
Tie Nail in Tel. Pole 19'N 42°-30'E
Tie Nail in Tel. Pole 19'N 42°-30'E
Tie Nail in Tel. Pole 19'N 42°-30'E
Tie Nail in Tel. Pole
Tie Nail in Power Pole
Tie Nail in Tel. Pole
Tie Nail in Tel. Pole
Tie Nail in Tel.



Section A-A

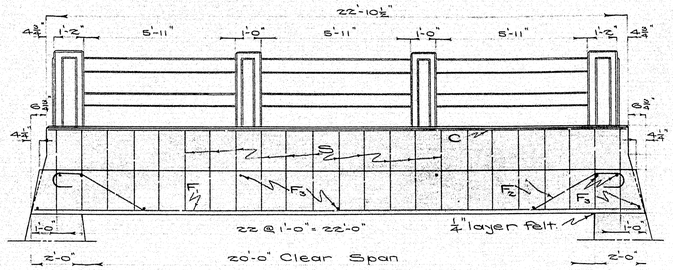
Note — Q-bars to be placed 6" below the bridge seat and 6" above the masonry spaced 6" on centers. They are to be lapped and bent in the field to fit the forms.

Abutments symmetrical about ℄

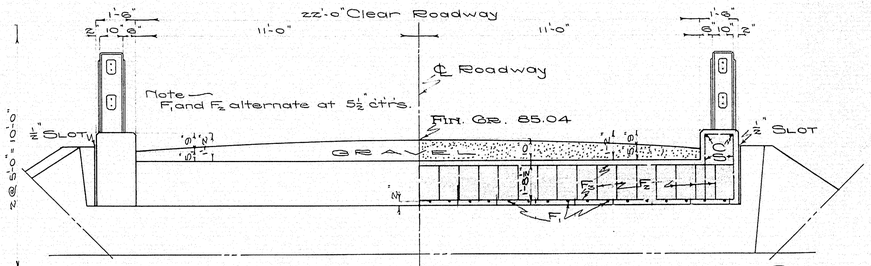


Quarter Pile Plan

(48 — 12 ton piles r'q'd)
(Both Abuts.)



Longitudinal Section

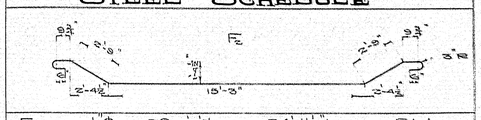


Half End Elev. Superstructure

Half Transverse Sect. Superstructure

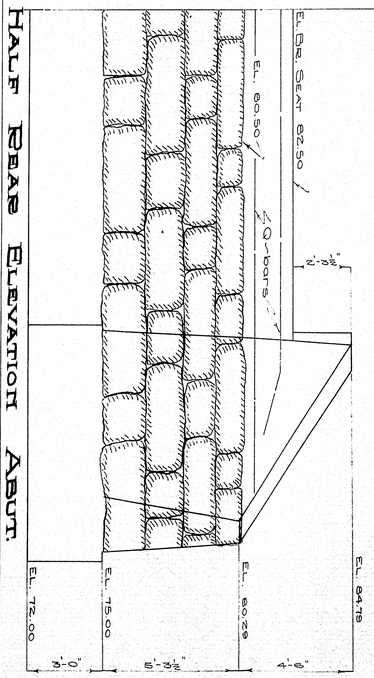
Note — Cover the ½" vertical slots between the superstructure and the wing walls on the back side with two layers of heavy roofing felt. Coat surfaces of concrete and back side of each layer as applied with hot tar or asphalt. Felt to be 10" wide. The area to be covered by the felt is to be recessed ½" by nailing thin strips to the forms before the concrete is placed.

Steel Schedule



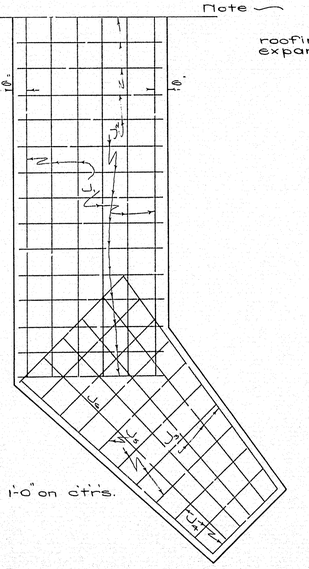
Mark	Size	No. R'q'd.	Length	Location
S	½"φ	46	6'-4"	Curbs
F1	1"φ	27	22'-11¼"	Slab. Str.
F3	¾"φ	11	24'-8"	" "
Q	¾"φ	36	24'-8"	Abuts.
J1	¾"φ	12	28'-0"	Abut Footings
J2	¾"φ	58	5'-8"	" "
J3	¾"φ	24	10'-0"	" "
J4	¾"φ	12	3'-8"	" "
J5	¾"φ	16	4'-6"	" "
J6	¾"φ	16	4'-9"	" "
P	¾"φ	32	4'-0"	Posts
R	⅝"φ	24	6'-2"	Rail Bars
C	½"φ	4	22'-7"	Curbs

Note — All steel to be plain bars of structural grade. All dimensions are to centers of bars.



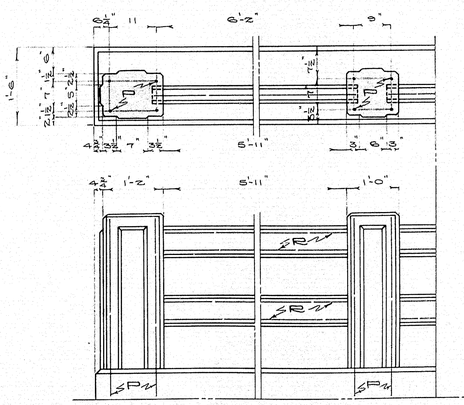
Half Rear Elevation Abut.

Design by
Traced by
Checked by



Quarter Plan Footing Steel

Note — J-bars 1'-0" on ctrs.



Rail Details

Note — Curb and slab to be cast together. Steel for posts to be set in curb. Precast rail bars in lengths of 6'-4". Place rail bars in position with ends projecting into post forms 2½". Wrap end 6" with two thicknesses of light roofing felt. Fold in ends and when post forms are removed cut away all felt exposed. Panels on posts to be ⅝" thick. Chamfer all exposed edges of concrete ½" unless otherwise indicated.

MAINE HIGHWAY COMMISSION
BRIDGE DIVISION

Marsh Bridge

OVER

Marshall Brook

BETWEEN THE TOWNS OF

Southwest Harbor

AND

Tremont

Hancock County

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0 1 2 3 4 5 INCHES